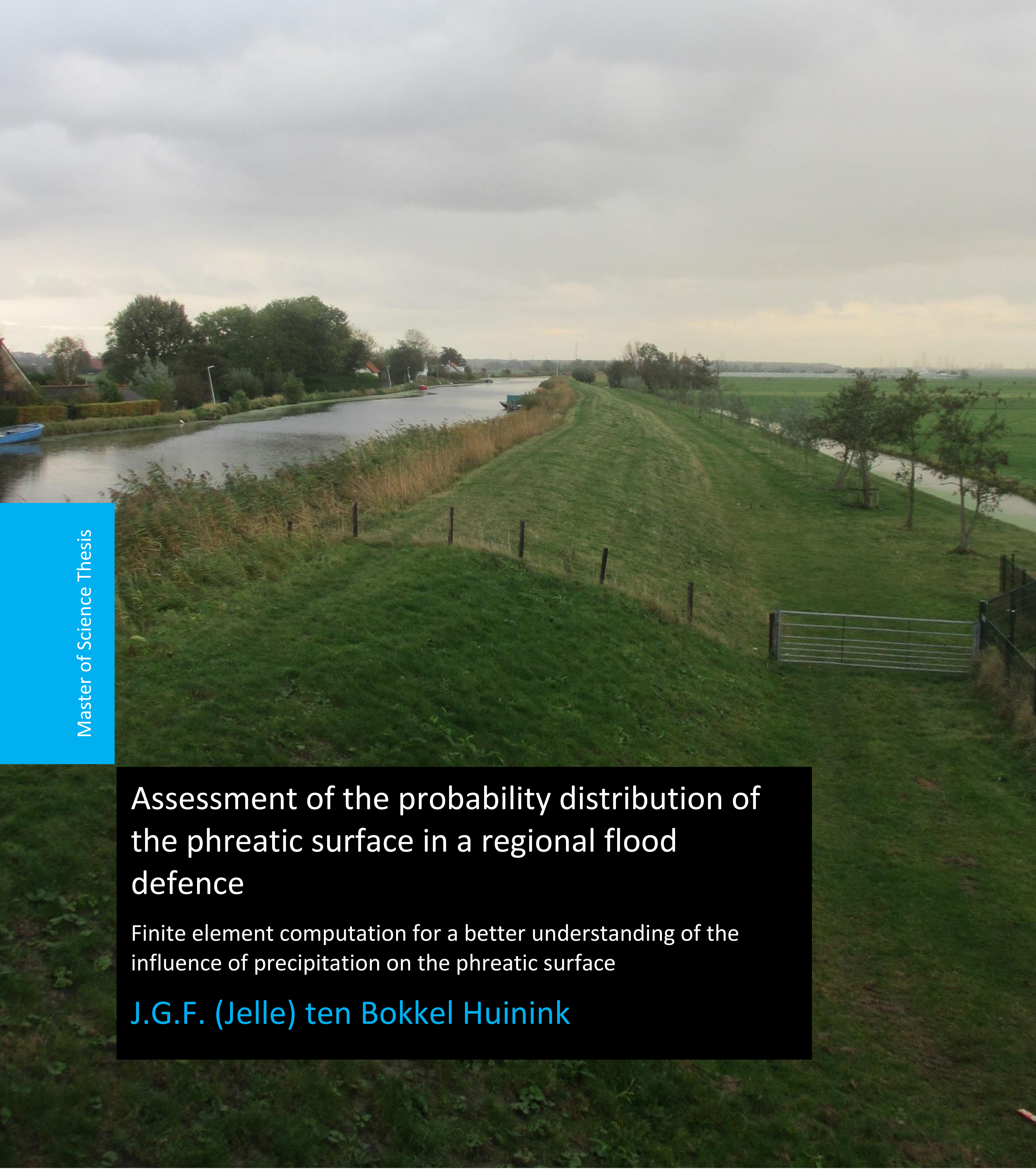

Master of Science Thesis

Assessment of the probability distribution of the phreatic surface in a regional flood defence

Finite element computation for a better understanding of the influence of precipitation on the phreatic surface

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SWECO

The front cover shows a typical Dutch regional flood defence in the area Delfland

Probability density distribution of the phreatic surface in a regional flood defence

Finite element computation for a better understanding of the influence of precipitation on the phreatic surface

By

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An electronic version of this thesis is available at http://repository.tudelft.nl/

Preface

This report is the result of a graduation study for the Master Hydraulic Engineering at the Technical University of Delft. The study is performed with the support of the engineering firm Sweco, section hydraulic engineering. The graduation study is the last step to obtain the degree of Master of Science in the direction of hydraulic engineering.

I would like to thank the head of my graduation committee, prof. dr. ir. M. Kok, for his support and critical vision on the study. Next, I would like to thank my daily supervisor, K.T. Lendering, for all the help in moments of doubt and for the role discussion partner, which provided new insights.
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Last but not least I would like to thank my family for the support during the thesis project.

Jelle ten Bokkel Huinink

Delft, November 2016

Summary

A large part of the Netherlands consists of polder area situated below mean sea level. The inhabitants have to rely on the Dutch flood defence system to protect them. A vast network of drainage canals ensures that the seepage water and the precipitation water is transported from the polders toward the outside water (i.e. the sea or a river). The embankments of the drainage canals are called 'Boezemkaden', in the remainder of the report these will be addressed as 'regional flood defence'.

There are several mechanisms which can cause failure of a regional flood defence. The focus of the study is on one of the failure mechanisms called 'macro instability of the inner slope' also referred to as 'instability'. The stability is mainly determined by the groundwater level, also called phreatic surface, inside the dike. The stability of a dike depends on multiple factors (i.e. outside water level, pore pressures, traffic load, and soil mechanical behaviour). The pore pressures inside the dike cause a reduction in effective stress; the effective stress is necessary to stabilize the dike and therefore the pore pressures influence the stability of the dike.

The objective of the study is *to compose a method to assess the probability distribution of the phreatic surface.* With a probabilistic computation it is possible to assess *how* safe the region behind the dike is instead of only knowing *if* the area is safe enough.

Current schematisation of the pore pressures

The pore pressure varies over the depth of the dike, often the pore pressures are schematized as a phreatic surface with a linear increase of pressure over the depth. Due to precipitation and water level change the pore pressures vary over time. In the current assessment method the effect of a high water level on the phreatic surface is quantified with an empirical formula. The effect of precipitation is however not known in detail, therefore a conservative approach is chosen to schematize the pore pressures: the pore pressures are schematized as a phreatic surface up to a maximum of 0,30 meters below the crest with hydrostatic increasing pore pressures.

In the current probabilistic method (Lendering, Jonkman, & Kok, 2015), a low, average and high phreatic surface are simulated with a probability density of respectively 5%, 85% and 10%.

Effect precipitation on the phreatic surface

The effect of precipitation on the phreatic surface is studied with a data analysis of a field test and a Finite Element Model (Plaxflow). The effect of precipitation depends on the saturated and unsaturated hydraulic soil characteristics; the saturated characteristics (permeability) determines how much pressure head is needed to discharge the infiltrating water; this determines how much the phreatic surface can rise. The unsaturated characteristics determine how fast the precipitation can infiltrate, this influences how fast a stationary situation will develop.

The study shows that the phreatic surface inside a dike can rise with several decimetres for small precipitation events; during extreme events the phreatic surface can rise to surface level. The pore pressures beneath the phreatic surface are non-hydrostatic, contrary to the governing schematization method. The computations show a maximum pore pressure field that is lower than the conservative simple schematization.

Assessment of the distribution of the phreatic surface

The result of this study is a method that describes a lower boundary for the probability density function of the phreatic surface for different soil types. In the method, the distributions of the following parameters is included: precipitation (statistical analysis), soil type (expert judgement) and

permeability (expert judgement or measurement). The report provides the probability density function of a rise of the phreatic surface for 11 combinations of soil type (i.e. sand, peat, clay) and permeability (2,8*10^{-5} m/s, 2,8*10^{-6} m/s, 2,8*10^{-7} m/s, 2,8*10^{-8} m/s) depicted in Figure 1.

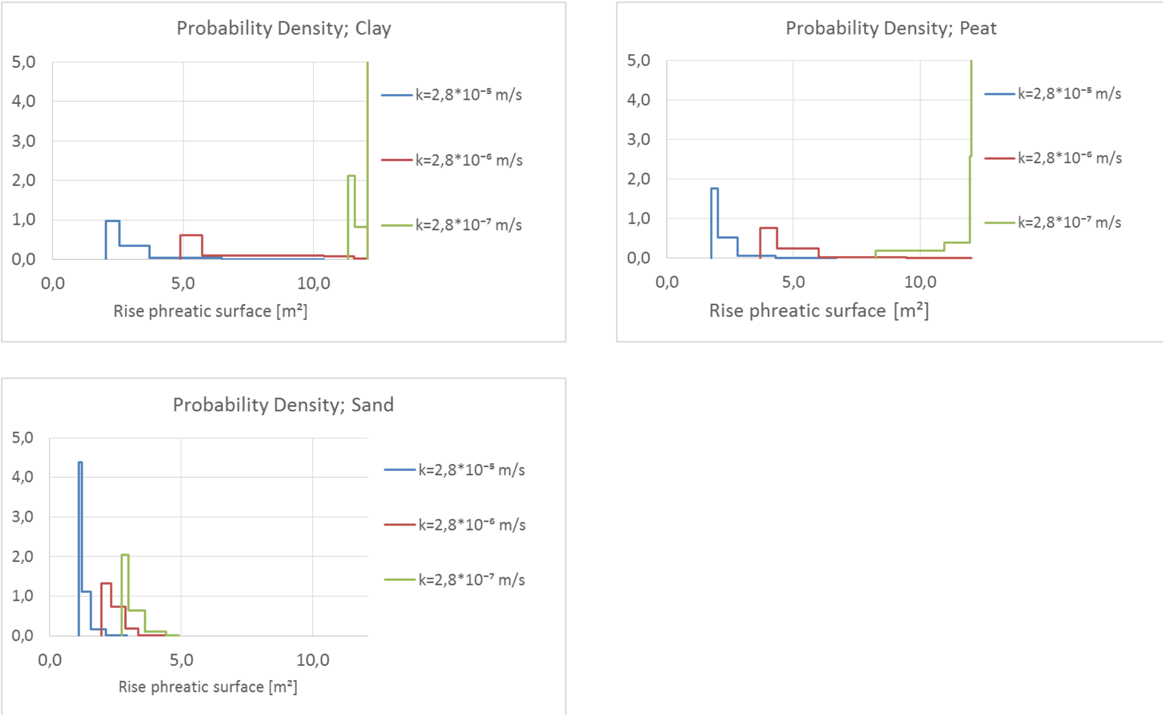


Figure 1 Probability density functions for a dike of clay, peat and sand for different degrees of permeability

A large difference with the current probabilistic method (Lendering, Jonkman, & Kok, 2015), is the probability mass of the different levels of the phreatic surface. In the new method only 4 high levels of the phreatic surface are regarded with a total probability mass of 100%. The average phreatic surface does not influence the probability of failure; the average is exceeded countless times per year. The low phreatic surface should be regarded separately instead of simultaneously with the high phreatic surface.

The probability density functions are based on a single profile, therefor the results must be scaled to fit the geometry. In general, it can be said that the precipitation has a smaller effect on the phreatic surface when the dike is wider or the relative crest height is larger; in those cases the initial phreatic surface is situated lower below the crest, therefor the initial degree of saturation at the crest is smaller which is related to the permeability of the top layer. The effect of precipitation on the phreatic surface can occur simultaneously with a high water level in the canal. It is necessary to compute the combined probability of occurrence through a system analysis of the polder. D-geo Stability is used to determine the failure probability regarding macro instability for all possible combinations of the water level and the level of the phreatic surface with the use of Bishop slip circles. By integration over the possible combinations, the probability of failure can be obtained.

Conclusion

Precipitation can have a large influence on the pore pressures; in this study the distribution of the phreatic surface is computed, based on uniform precipitation events and a homogeneous dike, for a range of soil types and different values for the permeability. The distribution of the phreatic surface can be used in a failure probability analysis. The governing method only includes the effect of precipitation in a conservative, deterministic manner; the new method provides more accurate information on the rise of the phreatic surface with different probabilities of exceedance.

Content

1 Introduction

A large part of the Netherlands is situated below mean sea level. Therefore the inhabitants have to rely on the Dutch flood defence system to protect them against floods, such as occurred in 1953 in a large part of Holland. The protection system consists of two main parts, the primary flood defence system and the regional flood defence system. The primary system protects the inhabitants against outside water such as flood waves on a river or a storm surge at sea. This system has the purpose to prevent large scale flooding and massive loss of life. All together there is approximately 3500 km of primary flood defences in the Netherlands. The regional flood defence system consists of all the non-primary flood defences, which includes 'boezemkaden', embankments of regional rivers, compartment dikes and summer dikes.

The boezem system is a system of drainage canals which transports the inside water towards a river or the sea as is depicted in Figure 2. Without this system, all low laying areas would slowly flood due to precipitation and seepage.

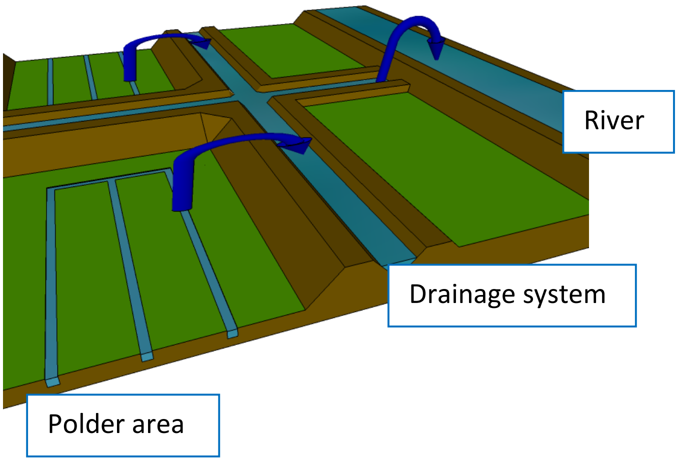


Figure 2 Illustration polder drainage system

In 1953 a major flood disaster occurred in the south west of the Netherlands, which killed about 1800 people. The disaster initiated the development of the Delta plan to protect the Netherlands against future floods. Since the development of the Delta plan much research is conducted to gain insight in the safety of the inhabitants behind the system of flood defences. This covers the entire field of flood defence engineering, from knowledge on safety assessment to knowledge on design, from knowledge on statistics to knowledge on reinforcement methods. As a result, the governing institutions are able to assess the strength and the needed strength of the flood defence systems. This is done by computing and assessing the failure probability regarding different failure mechanisms.

In order to gain insight in the failure probability of the primary flood defences and the dominant mechanisms, the study 'Veiligheid Nederland in Kaart 2' (VNK2) was conducted. In this study only the primary flood defences are considered. The result is the flood risk for different areas in the Netherlands due to the probability of failure of primary flood defences, in which the flood risk is defined as:

$$\text{Flood risk } [€/y] = \text{probability of flooding } [y^{-1}] \text{ x consequence of flooding } [€] \quad (1)$$

The project VNK2 does not include the flood risk due to breaching of a regional flood defence system. The impact of failure of a regional flood defence is less devastating compared to a breach of a primary flood defence. However, on a local scale the breach of a regional flood defence can have serious impact in the protected region. If a regional flood defence breaches, the polder can be flooded with increased water levels varying from several decimetres to several meters in for example the Haarlemmermeerpolder.

“Currently, the safety of regional flood defences is checked in a semi probabilistic assessment every six years. The assessment provides insight in whether or not the flood defence complies with the safety standards, however the assessment doesn’t provide a method to prioritise the required reinforcements based on the risk of flooding” (STOWA, 2015). Neither does the deterministic assessment give insight in how safe the dike actually is. In the report (Lendering, Jonkman, & Kok, 2015) a method is proposed, based on the VNK2, which provides a basis for a more thorough assessment of the regional flood defences. The study shows some knowledge gaps regarding the computation of the failure probability of the regional flood defences, for example the influence of precipitation on the macro stability.

A flood defence can fail through different mechanisms; four main failure mechanisms are macro instability, overtopping/overflow, piping and settlement. Macro instability is the loss of stability of a large part of the slope that can result in severe damage and even a total loss of the water retaining function of the dike. The instability of a large soil body can occur due to a changing groundwater level inside the dike, possibly combined with a load on top of the dike.

To compute the probability that the dike fails due to macro instability, the ‘load’ and the ‘strength’ of the dike are computed; the probability that the load is larger than the strength is the probability that the dike fails. Because a dike is usually an irregular and heterogeneous structure, the features of the dike need to be schematized in order to perform a computation. The schematization is necessary to simplify the complex structure but without the loss of necessary information. To make the schematization as detailed as necessary and as simple as possible, one has to know which parameters influence the result significantly.

Figure 3: Slip circle due to loss of macro stability (GeoDelft, 2007)

To compute the probability for macro instability, the load is compared with the strength regarding macro stability. In the case of macro stability there is no clear line between load and strength in the sense of parameters; some parameters influence both the load and the strength of the dike, for example the weight of the soil; the soil weight is the main driving force of the failure mechanisms. However, when the soil is heavier the effective stress is also larger and it will act as a stabilizing rather than destabilizing factor. In Figure 4 the variables which influence the macro stability are depicted; in this study the focus is on the highlighted parameters.

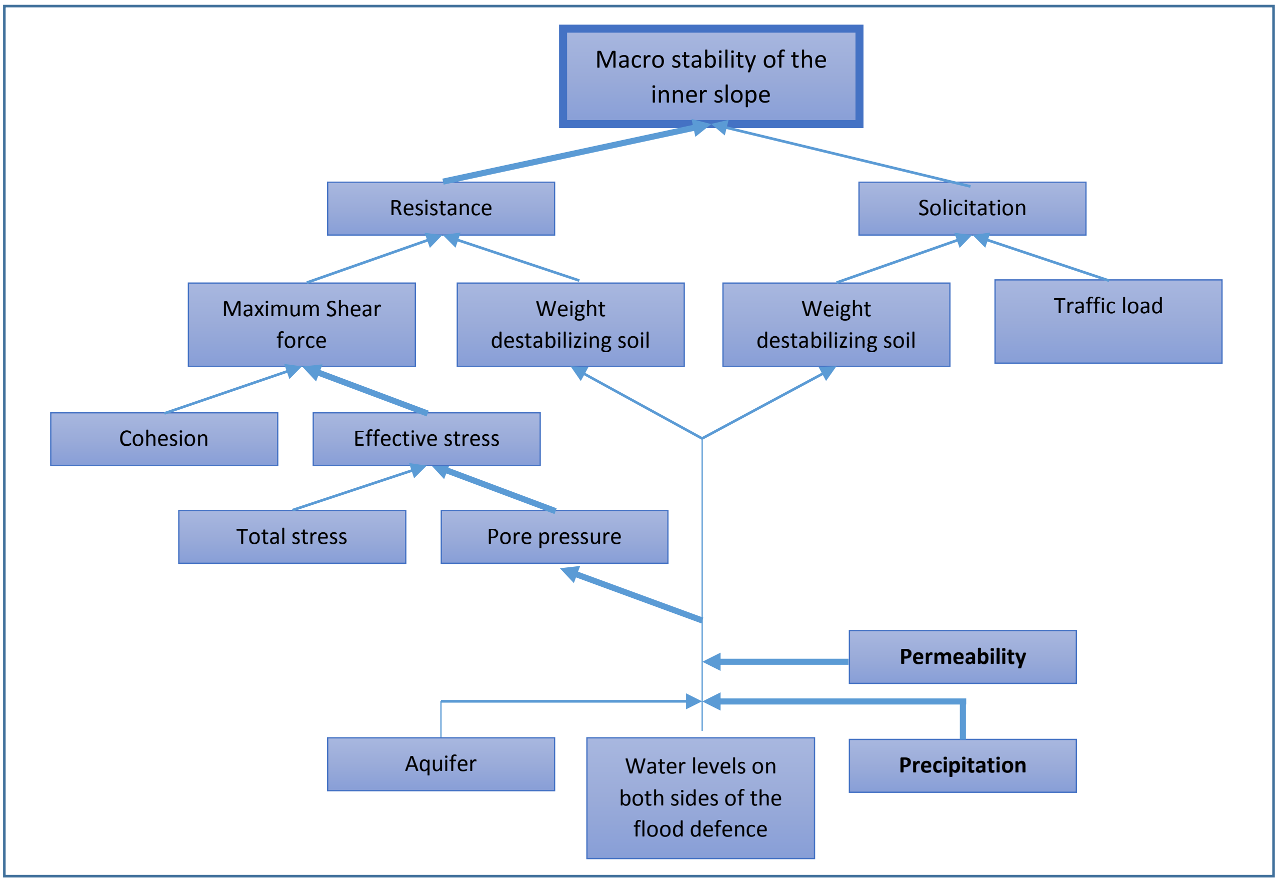


Figure 4 Schematic overview of the variables which influence macro stability; the variables in bold are subject of the study

1.1 Problem description

For macro stability of a regional flood defence one of the important variables are the pore pressures in the dike, which influence the effective stress necessary to create shear resistance.
In a homogeneous cross-section the groundwater table inside a dike can be approached as a straight line between the inside and the outside water level; this line is called the phreatic surface. It represents the location where the pore pressure is equal to the atmospheric pressure.[1]

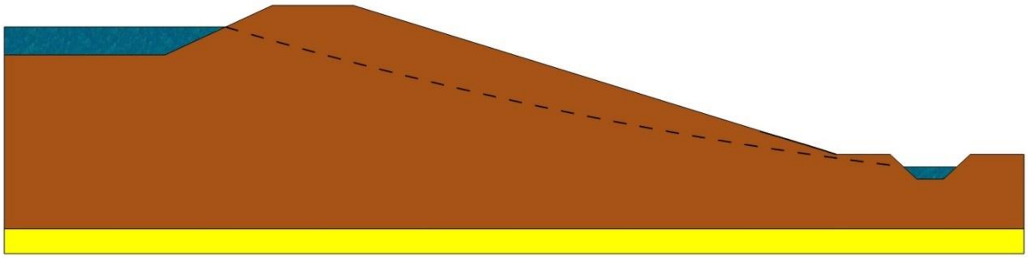


Figure 5 Schematization ground water table inside regional flood defence

The effective stress inside the dike, which generates shear resistance, is influenced by the pore pressure. In order to predict the size of the maximum shear strength, one has to predict the soil characteristics and the size of the pore pressures. As other research states is that 'for the stability of the regional flood defence a high phreatic surface is more dangerous than extreme water levels in or outside the polder, due to the small difference between the average and the extreme water level' (Lendering, Jonkman, & Kok, 2015).

The main source of information on pore pressures for Dutch 'flood defence' engineers is the technical report 'pore pressure in dikes' (TAW, 2004). The technical report provides a couple of basic and a couple of advanced methods to schematize the phreatic surface, based on a homogeneous dike. In the basic methods the dike is represented by a simple and conservative schematization; in the more advanced methods the dike is schematized less conservative and contains more details. Usually in the assessment of a regional flood defence, the simple method is used, unless it does not suffice, for example due to highly irregular features of the dike. The most advanced method which uses a finite element model is only used if it is unavoidable. The reason is that the gathering of the right information is usually more expensive than a conservative and simple approach.

In the stability computation the pore pressures need to be schematized. The current simple approach uses a simple, deterministic and linear schematization based on basal properties of the dike such as geometry and the water level. It is important to know the origin of this approach to understand the problem at hand.

The approach originates from a study conducted in 1977-1978 and 1990 (GeoDelft, 1990) by Geo Delft. The study in 1977-1978 consisted of in-field measurements of pore pressure and precipitation. A numerical continuation of the study is carried out in 1990 with the use of a Finite Element Method (FEM). The results from both studies contradicted each other; the infield study showed a significant response of the phreatic surface on precipitation whereas the phreatic surface in the finite element computation showed no response at all. A probable reason for this contradiction can be found in both studies; the measurements could have been wrong (and/or faulty interpreted) and the FEM techniques in 1990 where far from optimal. Whatever the reason was for the different results, the

[1] In reality the shape phreatic surface is not linear and is influence by the heterogeneity in the dike.

consequence was that there was no concessive answer on the question: 'what is the exact effect of precipitation on the pore pressures'.

The current approach to schematize the pore pressures is only based on the two contradicting results; the method can be summarized as follows: There are two categories, dikes consisting of sand and dikes consisting of clay. For a dike consisting of sand no effect of precipitation is included. For a dike consisting of clay a standard increase of the 'normal' phreatic surface with a maximum of 0,3 meter below the crest is used. A scientific justification for this approach cannot be found in neither the original report TRWD or in other literature. The lack of justification and background information has also been reported in the report 'actual strength of dikes' (Expertise netwerk waterveiligheid, 2009) conducted by a Dutch network of flood defence experts. Concluding, the current method is based on two studies with contradicting results. From these results, based on expert judgement, an addition on the phreatic surface depending on the dimensions of the dike is proposed and implemented. The included effect of precipitation is not directly based on local hydraulic circumstances and the origin of the method is not clear.

In the meantime, the field of flood risk has developed a more probabilistic approach in which uncertainties are included in a more sophisticated manner. A probabilistic approach has the advantage that it takes the uncertainty of all parameters into account instead of standard characteristic values. Furthermore, a deterministic approach only gives information on whether the structure is safe enough, for a statistically defined load, whereas a probabilistic approach gives insight in how safe the structure is.

To implement a probabilistic approach on regional flood defences, more statistical information is needed on the variation of the pore pressures. There is however no scientifically based information available yet on the uncertainty of the phreatic surface. Therefore in the report 'Flood risk of regional flood defences' an approach is chosen with a discrete set of plausible load levels in order to include the uncertainty of the phreatic surface. A set of discrete load levels is defined with a probability of occurrence at a random moment in time. The three levels are: a phreatic surface in drought, an average phreatic surface and a saturated level. Because there is no statistical information on how likely each level is, the probability density is based on an educated guess. The levels are based on the TRWD (TAW, 2004), and expert judgement by the HHNK, it does include precipitation in a simplified manner.

- Drought with a probability density of 5%;
- Average with a probability density of 85%;
- Saturated with a probability density of 10%;

In order to provide a more scientifically based method to include the uncertainty of the phreatic surface further research is needed. The first knowledge gap can be found in the question 'how is the phreatic surface influenced by the precipitation. The second gap is in how to implement the reaction on precipitation into a failure probability analysis.

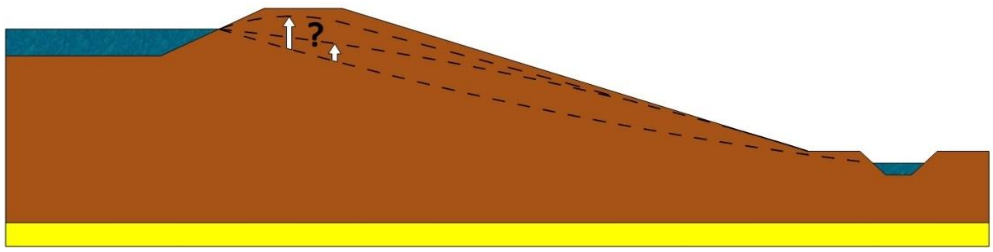


Figure 6 Illustration unknown uncertainty of the phreatic surface

1.2 Objective and research questions

The aim of this report is to provide a better scientific basis on how to schematize the uncertainty of the rise of the phreatic surface, in order to compute the failure probability regarding macro instability in the landward direction.

The scope of this thesis is to develop a method to include the uncertainty of the phreatic surface; special attention is paid on the influence of precipitation on the phreatic surface. The direct uncertainty of the phreatic surface due to a high water level is not studied; the possibility to include the correlation between the water level and the precipitation is studied. The soil mechanical aspects in the stability analysis are out of the scope of the thesis.

In this report the next questions will be answered:

'How to assess the uncertainty of the phreatic surface in a regional flood defence influenced by precipitation and the water level?'

To answer the main research question the following sub-questions will be answered:

- *What is the influence of the phreatic surface on the macro stability of a regional flood defence and why?*
- *How are the pore pressures schematized and included in 1) the current deterministic safety assessment and 2) the current failure probability analysis?*
- *How do the pore pressures develop under the influence of precipitation?*
- *How large is the response of the pore pressures to different combinations of hydraulic loads (i.e. precipitation and water level) and soil types (i.e. (unsaturated) permeability)?*
- *What is the effect of the new schematization of pore pressure uncertainty in a macro stability failure probability analysis?*

Together the answers to the questions form a foundation for a more accurate estimation of the uncertainty of the pore pressures in a regional flood defence. This is necessary in order to compose a method to compute the failure probability regarding macro stability in which the uncertainty of the phreatic surface is related to the uncertainty in precipitation.

1.3 Research approach

The study is focused on the question:

'How to assess the uncertainty of the phreatic surface in a regional flood defence influenced by precipitation and the water level?'

The first step to answer this question is a literature study on three topics. The first topic was a study on the general aspects of regional flood defences and the effect of the phreatic surface on the macro stability. The second topic that is studied is the current method of assessment of a regional flood defence with the goal to identify how precipitation is included. The third studied aspect was on the possibility to implement a probabilistic computation method for regional flood defences.

With the results from the literature study a concept method, to include the uncertainty of the phreatic surface in a failure probability analysis, is presented. In the concept method the knowledge gaps are identified; in order to construct a full method the two most important knowledge gaps are further studied.

The first aspect is the behaviour of pore pressures in a dike under the influence of precipitation. Therefore the data gathered in a field test by the water board 'Hoogheemraadschap Hollands Noorderkwartier' (HHNK) is reanalysed. In this field test the head in a peat dike is monitored in different wells during extreme, artificial, precipitation.

The conclusions from the analysis are used to set up a study in a Finite Element Model (FEM) called Plaxflow. This software package is developed to analyse complex nonstationary situations regarding ground water flow and precipitation; Plaxflow is highly useful for complex projects and research projects (TAW, 2004). The target of the use of a FEM is to gain information on the pore pressure development during extreme conditions to be able to schematize the phreatic surface under extreme conditions. In order to present a method to include the phreatic surface in a failure probability analysis, the dominant parameters are listed and quantitative data is extracted from the finite element method.

The last step was to quantify a relation between the precipitation and the rise of the phreatic surface. With the definition of the relation, the probability distribution of the phreatic surface is determined for different combinations of saturated and unsaturated characteristics. The combination of all studied aspects are combined in one method to include the uncertainty of the phreatic surface in the failure probability analysis regarding macro instability. The results found in this study are compared to both the governing assessment method as the probabilistic method proposed in (Lendering, Jonkman, & Kok, 2015).

1.4 Outline of the report

The structure of the report is schematized in Figure 7. In chapter 2 and 3 background information on flood risk, regional flood defences and the effect of precipitation is given. In chapter 4 the deterministic and probabilistic safety assessment of regional flood defence is elaborated. Chapter 4.3 presents the concept of a method to include the uncertainty of the phreatic surface in a failure probability analysis. In chapter 5 and 6 the question '*How do the pore pressures in different soil types develop under influence of precipitation'* is answered with the use of field test data and a Finite Element Method. In chapter 7 the relation between extreme precipitation and extreme levels of the phreatic surface is discussed and is concluded with the probability density functions of the phreatic surface in a dike for various combinations of saturated and unsaturated permeability. In chapter 8 the method to include the effect of precipitation on the uncertainty of the phreatic surface is presented and compared with the governing methods. The report ends with the conclusion and discussion in chapter 9.

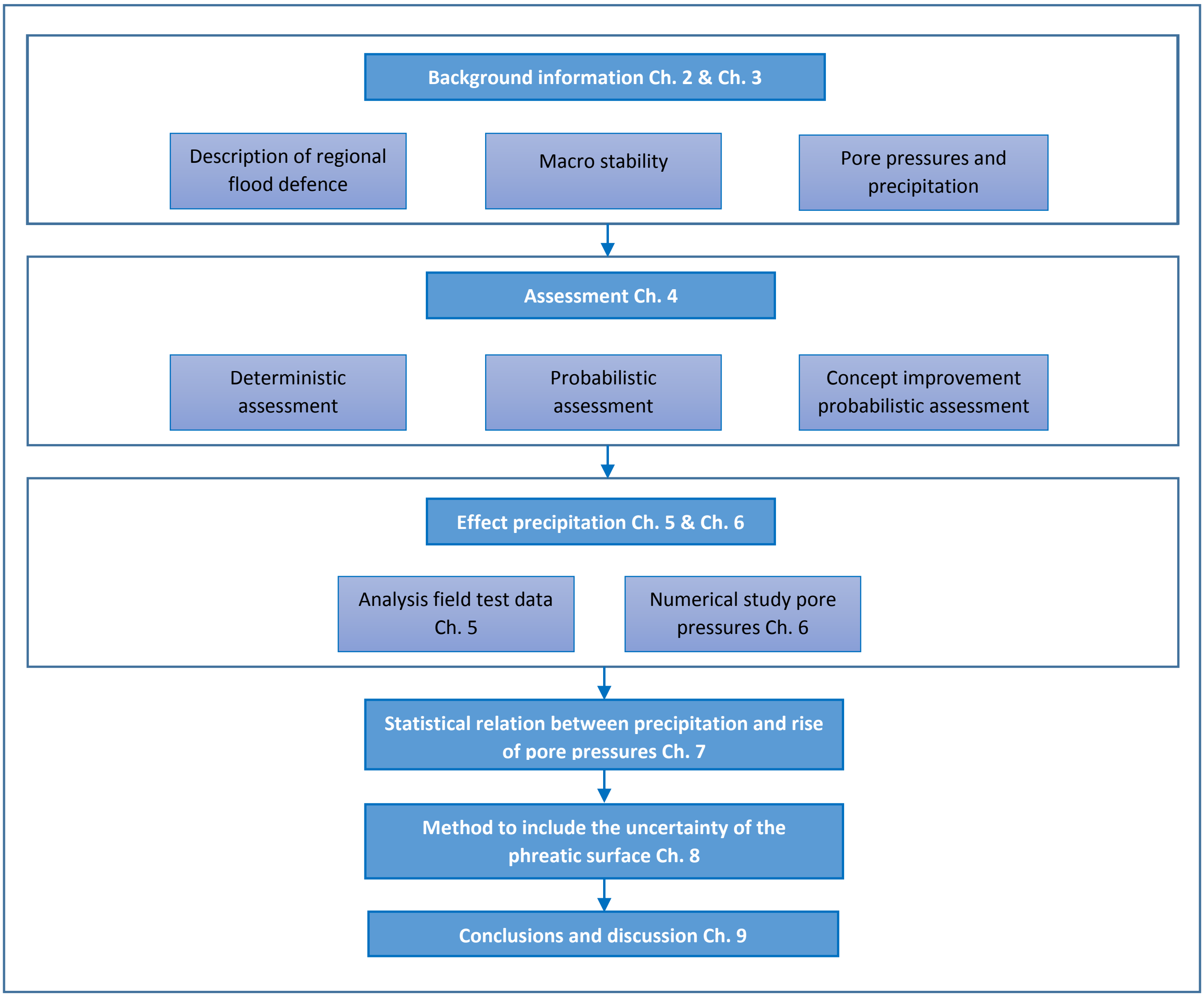


Figure 7 Schematic overview of the outline of the report

2 Background information regional flood defences

A large part of the Netherlands is situated below mean sea level. From as early as around 1000 A.D. the first dikes were built in the Netherlands. With the introduction of dikes it was possible to build safely below the mean sea level and in flood prone areas.
With the introduction of dikes it was possible to keep the rushing water outside. The water which entered the polder through precipitation and seepage could be discharged during low water. Due to the constant threat, people were forced to reinforce the flood defences constantly. Every time a dike breached, it was rebuild to the level of the previous storm. Furthermore, people started to pump water from low laying areas in order to use the land for agriculture. With each innovation, like windmills and steam engines, it became easier to pump dry lower laying areas, called polders. The water in the low laying areas is collected in small drainage canals and is pumped towards a higher drainage canal which usually drains in a river.

In this chapter the subject of the study is placed in the bigger picture; therefore first the flood defence system in the Netherlands is elaborated. Next the role of the regional flood defence system is presented followed by the failure mechanisms of a regional flood defence. The last part of this chapter zooms in on the role of precipitation in the failure mechanism macro instability.

2.1 Flood defence systems in the Netherlands

There are two main categories of water defence systems in the Netherlands, the primary flood defence system and the secondary or regional flood defence system.
The primary flood defence system is managed by the government and protects the Netherlands against ´outside water´. The main threats from outside water in the Netherlands are river floods and the sea. Due to a large storm in 1953 which resulted in several breaches and almost 2000 casualties, a national flood defence system was proposed. This system is called the Delta plan and resulted in national safety standards for the primary flood defence system.

The definition of a regional flood defence is: ‘A regional flood defence is a non-primary water defence which is included in the local rule system of the water board’. A regional flood defence is managed by a water board and protects an area against ‘inside water’. The inside water forms a threat because a large part of the Netherlands consists of polder area. If an area is situated below mean river level the precipitation water cannot flow off freely and for that reason a large scale drainage system is developed over time. Most of the embankments along the main drainage canals are in the category ‘regional flood defences’. The main differences between a primary and a regional flood defence are summed up in Table 1.

	Primary	Regional
Source of the threat	Outside water; sea and river	Inside water; (precipitation) drainage water
Consequences; economic/loss of life	Large	In most cases small compared to the breach of a primary flood defence
Water level difference	Irregular outside water level	Constant and permanent water level difference
Construction material (in general)	Sand core with a protection layer of clay	Local materials such as sand, peat and clay
Safety standards	1/1.250~1/10.000 per year	1/10~1/1.000 per year
Function	Single function: Flood defence. (Sometimes combined with a road)	Multifunctional: Flood defence, road, recreation, mooring spot and meadow

Table 1 Main differences between primary and regional flood defences

2.2 Regional flood defence systems

2.2.1 Description of the regional flood defence system

The regional flood defence system protects a large part of the Netherlands from the inside water. The retaining height of regional flood defences is often in the order of one to several meters. In the past the regional flood defences where constructed when needed, from materials in the area. This resulted in three main types of regional flood defences, depending on the area where they are constructed. The main difference is the type area in which they are constructed which determines the water level difference.

	Flood prone areas	Previous swampy areas	Naturally inundated areas
Depth	Between main and high water level	Just below the mean water level	Depth up to NAP -6m
Type of embankment	Partially surrounded by high ground and partially surrounded by manmade dikes	Surrounded by naturally present embankments and partially heightened by man	Naturally surrounded by 'high' grounds; for example former lakes.
Material of embankment	Partially high ground and partially local material	Peat	Clay and peat
Material of subsoil	Mainly river sediment	Mainly peat and clay	Mainly clay
Specific characteristics	Depending on the location one can find a sandy subsoil	Subject to large land subsidence	Due to its large depth usually divided in compartments with different depths

Table 2 Regional flood defences in three types of areas

Each polder which is situated below the mean water level must be drained continuously, in order to prevent water nuisance and flooding. The water is pumped into a drainage canal, which discharge the water into the so called outside water depicted in Figure 8. The outside water is for example a river or the sea.

The water level in both the polder and in the drainage canals is strictly regulated. Through this regulation the water levels in both the polder as in the drainage canal have a constant level with little variation.

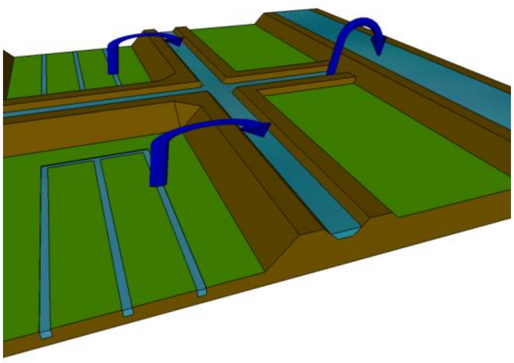


Figure 8 Regional flood defence system

2.2.2 Failure of a regional flood defence

A regional flood defence must be able to guide the (drainage) water towards a river or the sea without failure of the flood defence in which the definition of failure is the loss of the water retaining function. There are several mechanisms which can result in failure; the most important ones are sliding of the inner slope, piping, settlement and overflow which are depicted in Figure 9. From the four main failure mechanisms, the mechanisms 'sliding of the inner slope', called macro instability, is the subject of this study.

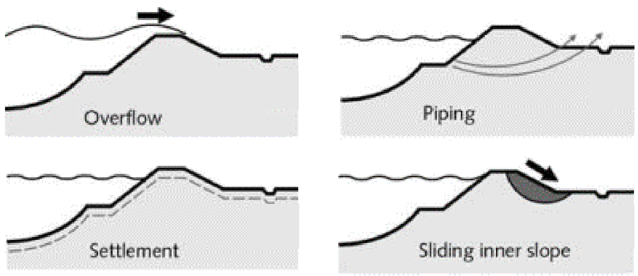


Figure 9 Dominant failure mechanisms regional flood defence system (TAW, 2001)

Overflow

The failure mechanism which is most commonly known is overflow. When the water level rises above the crest, water will overtop and flow into the polder. This immediately results in a (slow) inundation of the polder.

A more dangerous effect of the overflow is erosion of the inner slope. The rushing water suspends soil particles and over time the dike will become weaker and weaker. Eventually the erosion can result in a breach; in this case there is no barrier left between the water in the drainage canal and the polder which results in inundation of the low lying areas behind the dike.

Piping

When the water level in the drainage canal rises, the hydraulic gradient over the dike rises as well. The rise of the hydraulic gradient causes a higher water pressure from the drainage canal and due to the increased pressure more seepage will occur.

When the dike, or the aquifer below the dike has a low permeability, the flow velocity will be very small. However, if the permeability of a layer inside the dike is unfavourably high, the flow velocity will rise. If the velocity becomes high enough, the flow will suspend soil particles at the point of exit. When this situation does not change, more and more particles will be washed out; this creates a channel inside or underneath the dike. When the channel reaches the outside water, the flow velocity will increase even more resulting in more damage and eventually breaching of the dike.

Settlement

Many polders in the Netherlands are subject to settlement; due to dewatering of the polders the soil will settle due to its own weight. Furthermore a large amount of polder soil contains peat; when peat is exposed to the air it will slowly disintegrate and cause the soil to settle even more. The settling speed of the Dutch polder areas- is in the order of 0,01 to 0,1 meter per year. Due to the settlement, the height of the dike slowly decreases which increases the probability of flooding.

2.3 Macro instability; the mechanism explained

The failure mechanism macro instability can be best described as a soil body which slides along an arbitrary slip plane due to a loss of stability. The proces is depicted in Figure 10. The definition of macro stability is:

> *Macro stability: 'the ability to withstand loads without the loss of its water retaining function due to large deformations'.* (Deltares, 2013)

The reason that instability can occur during an extreme event, is the change in forces in the soil body. To comprehend the cause of this failure mechanism it is important to understand each contributing factor. Therefore the factors which contribute to the balance of forces are described in this chapter. Macro instability can occur when the driving forces become larger than the resisting forces; a simplification of all parameters which influence macro stability is presented in Figure 11. The bold variables are the subject of the study.

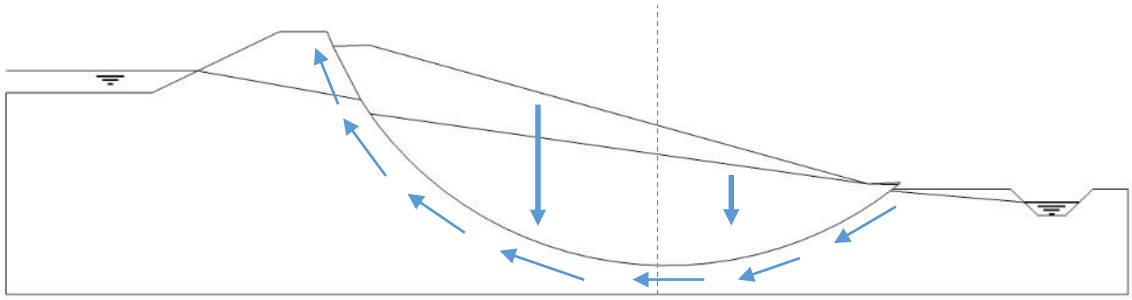


Figure 10 Schematization mechanical aspects macro instability

Self-weight of the soil

The self-weight of the dike is the main driving force for macro instability. The self-weight of the embankment depends on the geometry and the present soil materials together with the amount of water inside the dike. Important to notice is that the self-weight of the dike contributes as well to the resisting forces by its effect on the effective stress; with a larger self-weight the driving moment will be larger but the effective stress and thus the maximum shear force is larger as well.

Pore pressure and groundwater head

The water inside the soil is called groundwater; due to the presence of water inside the pores pressure is exerted on the soil particles. The size of the pore pressure depends on three factors:

- Unit weight of water: in general the unit weight of water is 9,81 kN/m^3; in order to simplify the calculation sometimes the unit weight of water is estimated to be 10 kN/m^3;
- Depth: the pore pressures depend on the depth below the groundwater table; the pressure increases linearly with the depth;
- Ground water flow: when there is ground water flow there must be a pressure difference; the pressure difference results in a changed pore pressure field. This effect is elaborated in chapter 6;

In a stationary situation, as is depicted in Figure 12, the pore pressures beneath the phreatic surface increase linearly. The groundwater head is the sum of the pore pressure ($\frac{u}{\rho_w g}$) plus the height above the reference level (z), in which 'u' is the pore pressure, 'ρ_w' is the density of water and 'g' is the gravitational constant ; in a stationary situation without flow the groundwater head ($\frac{u}{\rho_w g} + z$) is constant.

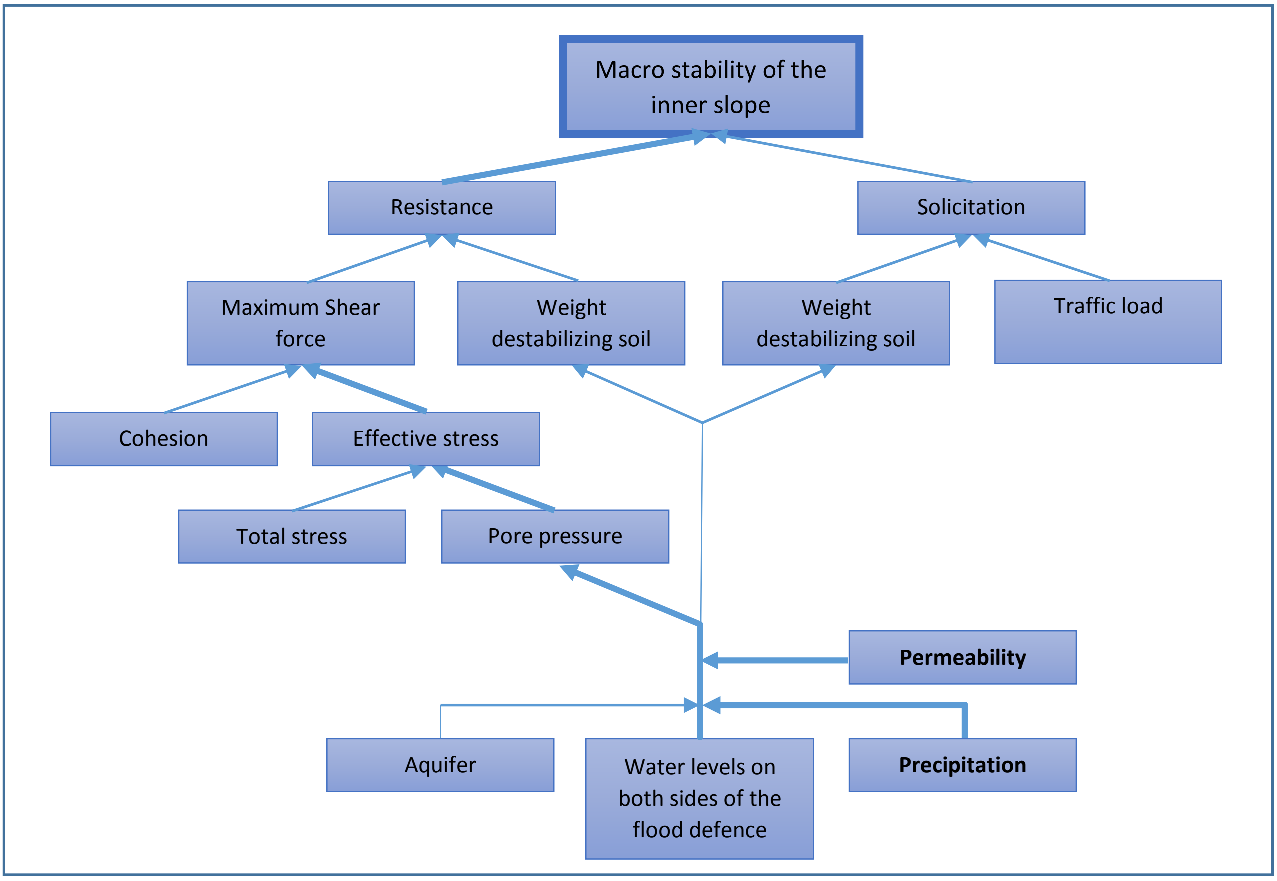


Figure 11 Schematic overview of the variables which influence macro stability; the variables in bold are subject of the study.

Effective stress and total stress

Everywhere in a soil body the soil particles and water particles will exert stress on one another. The stress, called total stress (σ), is caused by the self-weight of the (wet) soil. The pore pressures (u) increase with the depth; the effective stress increases linearly over the depth with the self-weight of the soil minus the pore pressure. When the ground water table rises, the effective stress will decrease over the entire depth. Therefore an increase of the ground water table results in a lower factor of stability. In Figure 12 the effect of the pore pressure on the effective stress is illustrated.

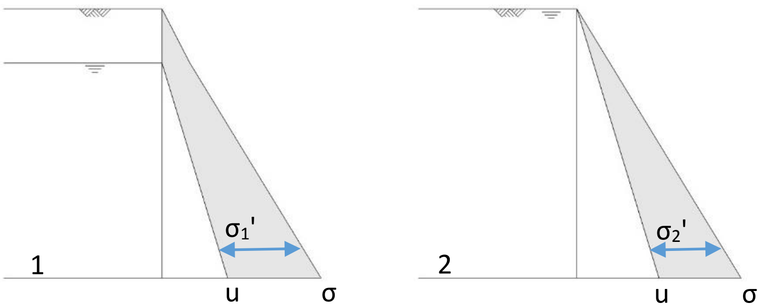


Figure 12 Illustration of the effect of the pore pressure on the effective stress; $\sigma_1' > \sigma_2'$ due to the increased groundwater level in situation 2.

Shear strength

The shear force along a possible slip plane is the most dominant force resisting macro instability. The shear strength is created by two material characteristics; the first characteristic is the ability of the soil to withstand shear forces; usually the ability to withstand shear forces of a soil body is expressed as the angle of internal friction (φ [deg]). The second characteristic is the cohesion of the soil (c [kPa]).

Besides the two material characteristics, the maximum shear strength is influenced by the (local) effective (normal) stress (σ_n' [kPa]); the effective normal stress is called 'effective stress' in the remainder of the report. When the soil particles are pressed firmly together, the maximum shear strength is large and when the soil particles are loosely packed the maximum shear strength is small.

$$\tau = c + \sigma'\tan(\phi) \qquad (2)$$

Top load

The last factor which can initiate macro instability is a (temporary) top load on the crest or the upperpart of the slope of the dike. It is self-explanatory that an object placed on the dike will exert a force which is unfavourable for the stability of the dike. The force on top of the dike caused by for example a standing truck is a local and usually temporary force. The advantage is that the dike segment can draw resistance from the surrounding dike body on which no top load is present.

2.4 Ground water and pore pressures in a dike

The pore pressures in a dike are one of the most important factors for the stability of the dike. Therefore the effects of the hydraulic conditions in- and aside the dikes are presented in this chapter.

When a typical regional flood defence is regarded, such as depicted in Figure 13, there is always a water level difference over the dike. Inside the dike the two water levels are connected through the ground water 'level'. The ground water level is called the phreatic surface and is defined as the collection of points where the pore pressure equals the atmospheric pressure. Underneath the phreatic surface the soil is fully saturated; above the phreatic surface the soil is partially saturated due to the capillary suction of the soil.

The pore pressures inside the dike will change due to changing boundary conditions such as water level differences and precipitation on the dike.

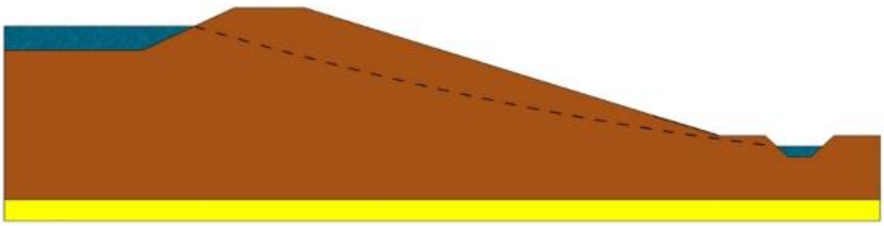


Figure 13 Water level difference regional flood defence

2.5 Conclusions

The Netherlands are protected against the outside water with a primary flood defence system and against the inside water with a secondary or regional flood defence system. There are several types of regional flood defence systems like the dikes along a drainage canal and summer dikes along a river. In this study the dikes along the drainage canal are referred to as regional flood defence; the other types of regional flood defences are not regarded in this study.

A regional flood defence is usually constructed from local materials; in the past, the reinforcements where not recorded and as a result there is a lot of uncertainty in the composition of the subsoil. Furthermore, the reinforcements where often based on a local scale which results in high variety over the length of the dike. A regional flood defence system can fail due to macro instability, this is influenced by the soil strength parameters and the (hydraulic) load on the dike.

The pore pressures influence the level of the effective stress inside a dike; the effective stress determines the shear strength of the dike. Therefor the pore pressures are the main variable which influences the resistance against macro instability. Precipitation causes the pore pressures to rise and therefor precipitation influences the failure probability due to macro instability. However, there is not yet a general method available to describe the effect of precipitation on the slope stability.

3 Precipitation on a dike

The pore pressures inside a regional flood defence are influenced by changing hydraulic conditions; precipitation is one of the hydraulic boundary conditions which can influence the height of the pore pressures inside a dike. First the effects of precipitation on the pore pressures are elaborated, this includes physical relations and different possible effects of a precipitation event. In the past multiple studies are performed to capture the effect of precipitation on the phreatic surface, one of the goals of the studies was to quantify a phreatic surface due to precipitation in order to include the effect in the assessment methods.

The pore pressures inside a dike have an important role in the stability of a dike; the higher the pore pressures are the less stable the dike becomes due to loss of effective stress. Therefor it is necessary to understand the role of precipitation in the variation of the phreatic surface.

3.1 Flow through a dike

The fundamental process which is important when regarding the effect of precipitation is the flow of water through a porous medium; in this case the dike is a porous medium which consists of soil particles and voids. The voids are partially connected and thus water can flow through the voids of a dike. The flow through a porous medium is described by Darcy's equation (11(3). The flow through a soil body is influenced by the pressure gradient ($\underline{\nabla} p_w$), the larger the pressure gradient the larger the flow velocity. The discharge through the soil is linearly related to the permeability of the soil; the discharge will be larger through a soil body with a higher permeability, and the same head difference than in a soil with a lower permeability.

$$\underline{q} = -\frac{\underline{\underline{k}}}{\rho_w g}(\underline{\nabla} p - \rho_w \underline{g}) \tag{3}$$

With:

$\underline{q}$= specific discharge;

$\underline{\underline{k}}$= tensor of the permeability;

$\underline{g}$= gravitational acceleration vector;

$\underline{\nabla} p$= pressure gradient;

ρ_w= density of water;

A dike can be roughly divided into two area's which is illustrated in Figure 14: the first area is the area below the groundwater level (phreatic surface); in this area the soil is fully saturated and is called the saturated zone. The second area is the soil above the phreatic surface; in a steady state situation, without precipitation, the area above the phreatic surface is partially saturated due to capillary rise. This area is called the unsaturated zone.

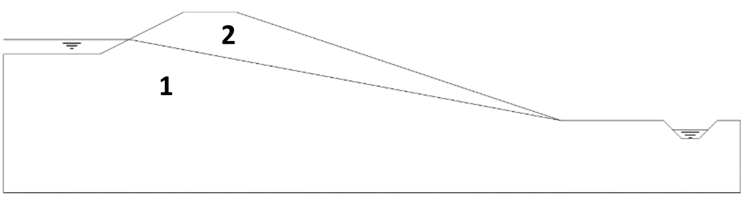


Figure 14 Illustration saturated and unsaturated zones in a dike

3.1.1 Flow through the saturated zone

The flow through the saturated zone is driven by the pressure difference between different locations. When regarding a regional flood defence there is a water level difference over the dike which causes a pressure difference over the dike, this pressure difference induces a flow through the dike from the drainage canal (left) to the polder area (right) which is illustrated in Figure 15. The discharge through the dike is influenced by the pressure difference, the width and the permeability of the dike.

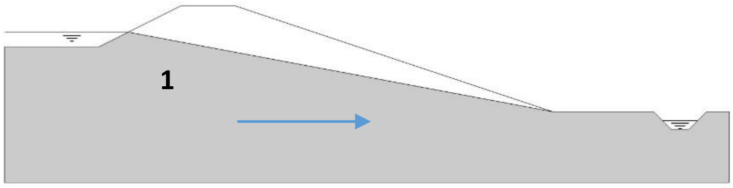


Figure 15 Illustration saturated zone

3.1.2 Flow through the unsaturated area

When there is no precipitation, no evaporation and no fluctuation in the water levels, the flow through the unsaturated area is minimal. A change in water level, or infiltration or exfiltration of water due to precipitation or evaporation, will cause an increased flow through the unsaturated zone. The flow through the unsaturated zone is based on the Darcy equation. There is one big difference with the flow through the saturated zone; the permeability in the unsaturated zone is not constant over the height and depends on the degree of saturation.

The permeability depends on the degree of saturation; simultaneously with the flow computation, the degree of saturation for each element is computed with the continuity equation. The continuity equation used in Plaxflow is based on (Song, 1990) and is simplified by assuming 1) no particle movement and 2) no gradients in the density of water (Boussinesq's approximation) (Plaxis, 2016):

$$\nabla^T \left(\frac{k_{rel}}{\rho_w g} \underline{\underline{k}}^{sat} \left(\underline{\nabla} \rho_w \underline{g}\right) - n \left(\frac{S}{K_w} - \frac{\partial S}{\partial \rho_w}\right) \frac{\partial p_w}{\partial t} = 0 \qquad (4)$$

With:

n=porosity
S=saturation
K_w=Elastic bulk modulus of water

The last step to compute the infiltration after each time step is the soil water characteristic curve (SWCC); the curves describe the degree of saturation depending on the negative pore pressure. The permeability of an unsaturated soil element depends on the degree of saturation; for multiple soil types the relation between the degree of saturation and the permeability is described in literature; for now it is important to know the following:

Each soil type has different unsaturated characteristics;
there is a relation between the degree of saturation and the permeability of the soil above the phreatic surface;

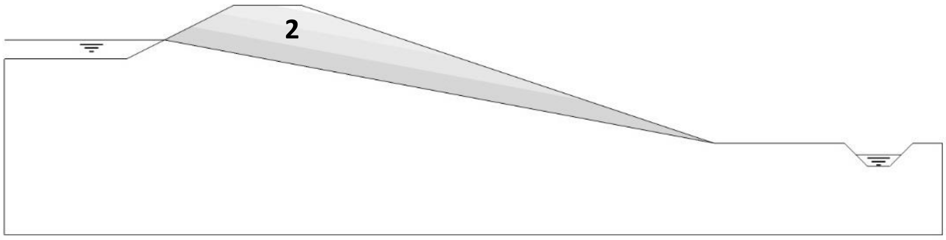


Figure 16 Illustration unsaturated zone above the phreatic surface

The permeability of the layer above the phreatic surface is expressed as a percentage of the saturated permeability and depends on the height above the phreatic surface. For different soil types the relation between the height above the phreatic surface and the permeability is different. This holds that, even if the saturated permeability is equal, there are differences in the unsaturated permeability.

In Figure 17 three typical curves for the unsaturated permeability of three soil types (sand peat and clay) are presented. The relations are based on the Staring data series; these show clear differences between soil types regarding the unsaturated permeability above the phreatic surface.

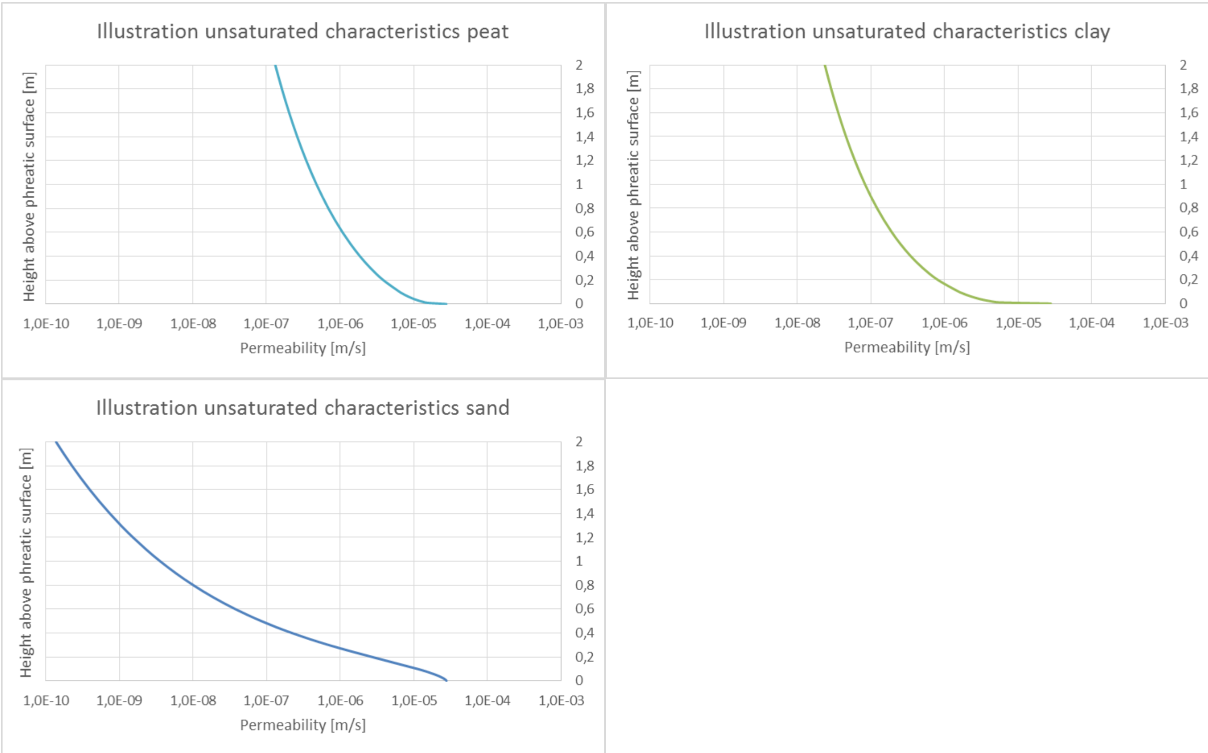


Figure 17 Illustration unsaturated characteristics peat (top left) clay (top right) and sand (bottom)

3.2 The effect of precipitation on the pore pressures

A precipitation event causes an inflow through the top layer of the dike. The infiltrating precipitation will cause the dike to become wetter, so the degree of saturation of the dike will increase during a precipitation event. The water will infiltrate and will flow through the toe of the dike towards the polder drainage system; pumps in the polder will take care that the water level inside the polder will not rise (too much). There are two main aspects which determine how the phreatic surface will respond to the inflow of water.

3.2.1 Discharge capacity

The water which infiltrates in the dike has to flow through the toe of the dike, towards the polder. The flow through the dike is driven by the pressure gradient inside the dike as is described in Darcy's law. In order to obtain a discharge capacity which is equal to the infiltration capacity the phreatic surface inside the dike will rise; a rise of phreatic surface results in a larger pressure gradient at the toe of the dike which facilitates a larger discharge. The phreatic surface will rise during the precipitation event, until the pressure gradient is large enough to facilitate a discharge that is as high as the inflow. This process is depicted in Figure 18.

In practice the process of the rise of the phreatic surface due to precipitation can be divided into two groups; either the phreatic surface has reached an equilibrium state at the end of the precipitation event or the phreatic surface was still rising and did not reach an equilibrium state.

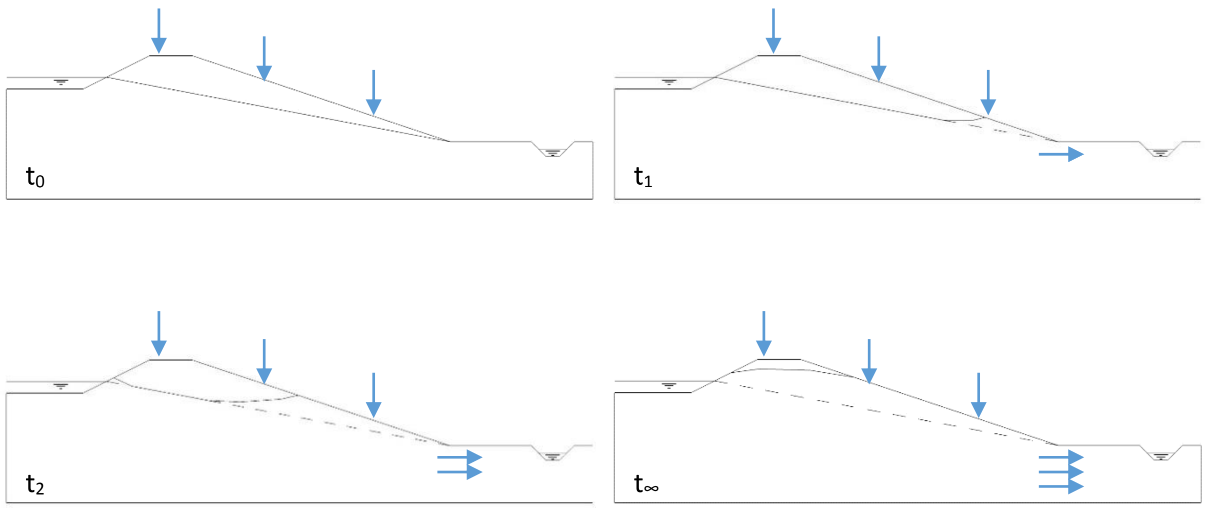


Figure 18 Illustration process of the effect of precipitation in the phreatic surface over time until an equilibrium is reached

3.2.2 Infiltration capacity

The second aspect which determines the behaviour of the phreatic surface is the infiltration capacity of the dike; the dike has a maximum infiltration capacity depending on the (un)saturated permeability of the dike. When the infiltration capacity is larger than the precipitation intensity, all the precipitation water will infiltrate in the dike. When the infiltration capacity of the dike is smaller than the precipitation intensity only a part of the precipitation will infiltrate; the first part of the water which cannot infiltrate will form puddles on the dike. In a later stage, the excess water will flow over the surface of the dike into the polder area, this water will not infiltrate in the dike and will not influence the phreatic surface directly.

The infiltration capacity is determined by the permeability of the soil; the permeability above the phreatic surface does depend on the saturated permeability but also on the degree of saturation. During a precipitation event the infiltrating water will cause the soil to become more saturated; thereby the soil becomes more permeable in the course of the precipitation event. The top layer does thus not have 1 infiltration capacity but has an infiltration function, with an increase of the permeability during a precipitation event.

3.3 Literature study on the effect of precipitation

In the past, multiple studies are performed, with the goal to quantify the effects of precipitation on the pore pressures. The reason for the quantification is that the effect of precipitation on the phreatic surface must be included in the assessment methods for macro stability. In this paragraph the two studies which are used as a basis for governing guidelines are elaborated.

Study on the effect of extreme precipitation on pore pressures in a dike; in-situ (GeoDelft, 1980)

The first documented study on this subject was conducted in 1978 by Grondmechanica Delft in Nieuw-Lekkerland and is documented in (GeoDelft, 1980). The goal of the study was to find a relation between the precipitation and the rise of the phreatic surface in order to use this information in a stability calculation.

Therefore, pore pressure measurement devices were installed in 7 cross-sections, each with 2 section lines. At each section line multiple devices were installed at different depths. Together with the pore pressures, the precipitation intensity and duration were measured as well as the outside water level.

For each week the influence on the hydraulic head due to the precipitation was determined with correction for the tidal movement and other effects of the outside water level. Each 7-day data point was plotted against the precipitation depth in Figure 19.

For the design of the dike reinforcement the phreatic surface was extrapolated to a design condition. The described design condition was a precipitation event with a duration of 7 days with a probability of exceedance of 1/4000 per year. The corresponding precipitation depth was 0,155 meters; therefore, the measurements are extrapolated to 0,155 meters in 7 days. The resulting increase of the phreatic surface in design condition, would be between 0,8 and 2,1 meters for the 7 sections.

The amount of run-off was not measured and thus not included in the extrapolation of the phreatic surface. This made it impossible to compose a water balance, which is necessary to compare the results.

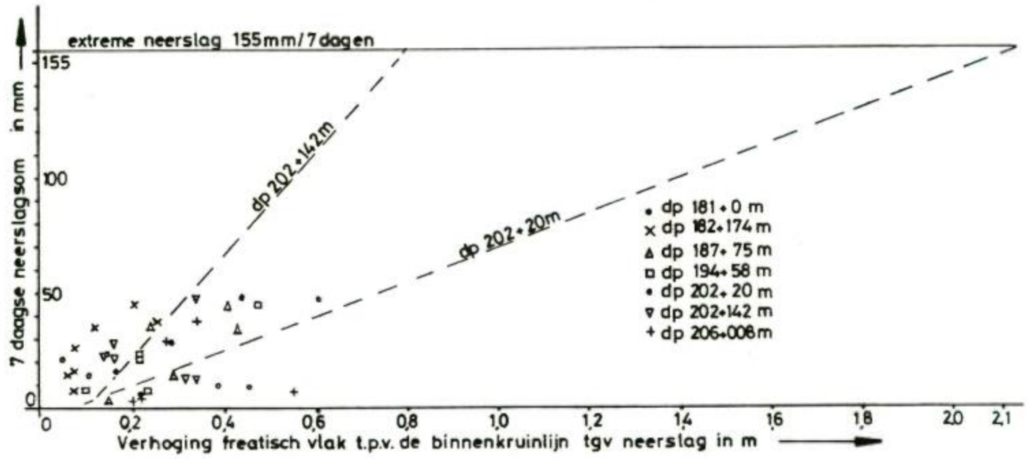


Figure 19 Relation precipitation-increase phreatic surface at the location of the inner crest line (GeoDelft, 1990)

Study on the effect of extreme precipitation on pore pressures in a dike; numerical (GeoDelft, 1990)

Twelve years after the in-situ study of the effect of precipitation a numerical study is performed on the same cross-sections described in (GeoDelft, 1990). The study is performed in the software package SWANFLOW. In the study one profile is schematized and used as input for the model; to reduce the complexity of the computation the left and right boundary of the model are assumed to be impermeable. The phreatic surface underneath the crest was situated more than 3 m below surface level; underneath the toe the phreatic surface was at surface level -1,8 m.

There are two precipitation events that were simulated:

1) an event of 1 mm/h for 2 weeks;
2) after event 1 an event of 16 mm/h for 1 day;

After event 1 the phreatic surface underneath the toe has risen 1 m; this was then compensated for the fact that the simulated right and left boundaries where impermeable which resulted in an expected rise of 0,5 m. During event 2 the phreatic surface underneath the toe rose to surface level.

In both simulated events the phreatic surface underneath the crest did not respond to precipitation.

The results from the numerical research showed completely different results than the in situ research 10 years previously; whereas the in-situ research showed a clear response of the phreatic surface due to precipitation, the numerical research showed no response underneath the inner crest line at all.

In a normal situation the pore pressure is hydrostatic over the depth; if the measurements in a normal situation (1) are extrapolated, the pore pressure line looks as in Figure 20. The same principle is used for the pore pressures in a dike during a precipitation event; so linear extrapolation of the measuring points (2); the phreatic surface would be at point a. However the simulated phreatic surface was at point b. The explanation given for the difference is that there is upward flow due to the fact there is no stable situation yet.

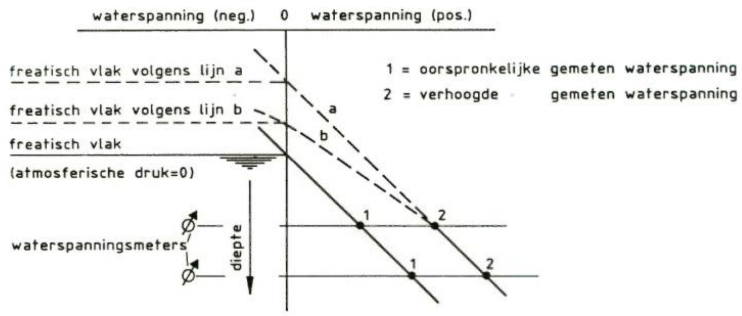


Figure 20 Pore pressure over the depth (GeoDelft, 1990)

Effect of precipitation on the phreatic surface in a dike consisting of peat (2013)

In 2013 a study is conducted for the water board 'Hoogheemraadschap Hollands Noorderkwartier' in which the pore pressures where monitored during an extreme artificial precipitation event inside two dikes; the entire study is documented in (Alterra, 2013). A more detailed description can also be found in chapter 5 and Appendix: F.

During the artificial precipitation event the pore pressures in different section lines are measured together with the actual precipitation intensity and the run off; the outside water level was kept at a constant level.

The next step in the study was a numerical reconstruction in Hydrus-2D, of the measured data series with the use of a finite element model. The model simulated a 2D flow through a soil body with a predefined relation between the saturated and unsaturated permeability; therefore the unsaturated characteristics of the soil (samples) are measured.

The simulated cross-section was subjected to the same precipitation event as the real dike in order to compare the results; this was an event with a duration between 1 and 7 days with a return period of 100 years.

The results from the numerical study are compared to the in-situ study and the following is concluded:

- The phreatic surface in the numerical model due to precipitation was higher than in the real situation; in the report by Alterra there is no explanation for this difference. A possible cause for the difference is that the pore pressures from the field test are linearly extrapolated, with the assumption of hydrostatic pore pressures whereas the computer model also simulates the non-hydrostatic effects.
- There are large differences in reaction between the two studied sites; in 1 test site the 1/100 year event resulted in no significant response and in the other test site a critical situation arose; a possible cause is a difference in material inside the dike;

3.4 Conclusions

The effect of precipitation on the phreatic surface depends on multiple factors; a part of the precipitation will infiltrate in the dike, this depends on the permeability of the (unsaturated) top layer. The water will flow through the toe of the dike towards the polder area. The flow is driven by a pressure difference, in order to create the pressure difference the phreatic surface will rise under influence of precipitation.

In order to quantify the relation between the rise of the phreatic surface and the amount of precipitation 2 studies are conducted in 1980 and in 1990; the studies consisted of a field test (1980) in which the pore pressures where measured in multiple cross-sections during several months together with the precipitation intensity. The result was an indication that the precipitation influenced the phreatic surface; it was not yet possible to provide a quantification of the relation between precipitation and the phreatic surface.

In order to provide more insight and quantitative information for an assessment method, a finite element (FE) study was performed in 1990 based on the study a decade before. The results from the finite element study contradicted the field study; in the FE study the phreatic surface did not respond significantly on precipitation events.

4 Macro stability assessment of a regional flood defence

In the previous chapter all the important aspects on regional flood defences, pore pressures and macro stability (further referred to as stability) are set apart. The theory of a stability calculation is straight forward, in order for the dike to fulfil a water retaining function, the resisting force must be larger than the driving force.

In practice, there is however uncertainty in all the parameters used in the macro stability equation. The background of the construction of regional flood defences makes that there is a lot of variation in materials that are used; many dikes are heightened many times without a (detailed) record of which materials are used and even which sections are reinforced. Therefore the soil structure is not known in advance; measurements must be performed and expert judgement is necessary to interpret the measurements. The measurements are often point measurements, this means the soil structure is only investigated at one or a couple of vertical lines (with a small diameter in the order of a decimetre). The point measurements are performed with an interval of tens to hundreds of meters. So even with (point) measurements and expert judgement there is still a lot of uncertainty in the soil structure. In engineering practice assumptions must be made for all the unknowns and uncertainties, data must be interpreted and a simplification of the reality is made; this is illustrated in Figure 21.

In chapter 4.1 the current deterministic assessment method for regional flood defences regarding stability is described and discussed. For primary flood defences a probabilistic method is developed in the project VNK2; this method is used to develop a probabilistic method for regional flood defences in (Lendering, Jonkman, & Kok, 2015). The probabilistic method is shortly described in chapter 4.2. In chapter 4.3 a concept method is proposed to improve the probabilistic method regarding the uncertainty of the phreatic surface.

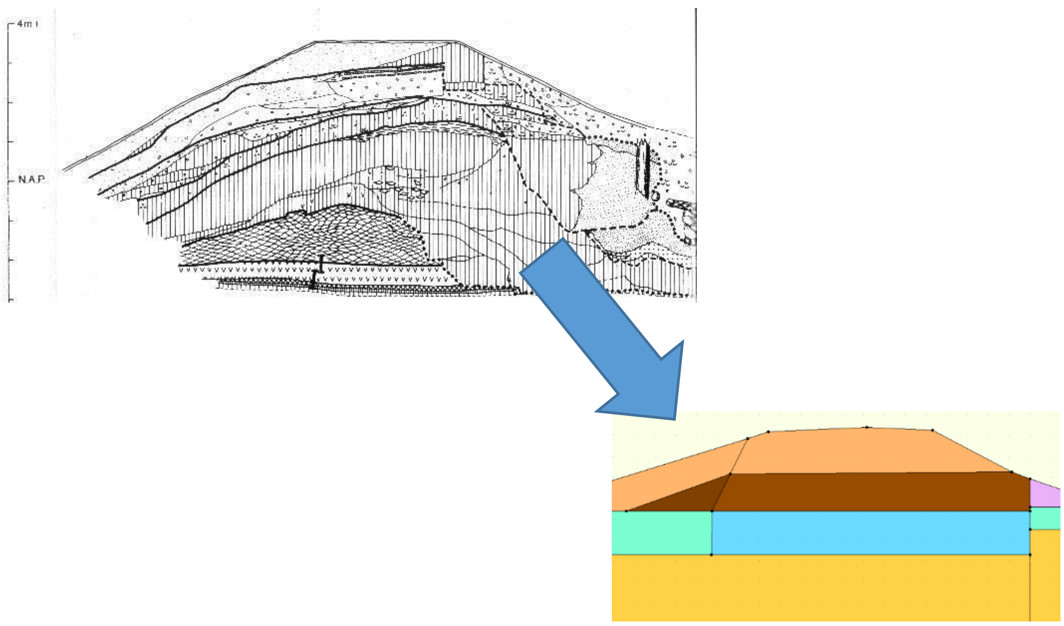


Figure 21 Illustration: from reality (left) (van Geel et al, 1993) to schematization (right)

4.1 Governing assessment macro stability; deterministic

In this chapter the current assessment method regarding macro stability is described. The basis for the national assessment method stems from December 1998, when the fourth policy plan of water management is formulated. This was the first time a nationwide approach is proposed for the assessment of the regional flood defence system (Ministerie van Verkeer en Waterstaat, 1998). From this moment onwards the water boards collaborate, in order to provide a national guideline to assess regional flood defences. This collaboration has resulted in the 'guideline for assessment of regional flood defences' (STOWA, 2015).

The guideline prescribes which aspects should be assessed and which types of schematization are necessary for all the regional flood defences. The water boards are obligated to assess the regional flood defences every 6 years with the use of the guideline.

4.1.1 Governing assessment approach for macro stability of regional flood defences

To understand why the method is as it is, it is important to look at the safety philosophy which was followed in the previous decades. The government and the water boards both wanted the Netherlands to be safe against flooding. The definition of safe is however a point of discussion, because 100% safe does not exist.

To determine how safe the hinterland must be, the government chose to evaluate all regions in the Netherlands on economic activity and the amount of inhabitants in each separate polder/region. According to the evaluation the polders are ranked in one out of 5 IPO safety classes. The IPO class gives information on the minimum level of safety which must be reached for that particular dike. The corresponding damage factor is used in the stability computation. The level of safety is related to the probability density function of the water level in the canals. In Table 3 information on the IPO classes is listed.

IPO class	Probability of exceedance water level [1/year]	Damage factor
I	1/10	0,8
II	1/30	0,85
III	1/100	0,9
IV	1/300	0,95
V	1/1000	1,0

Table 3 IPO classes regional flood defence (STOWA, Materiaalfactoren boezemkaden, 2009)

Each regional flood defence must be able to withstand a water level which is exceeded on average once in x years. The average exceedance frequency varies from 0,1 per year to 0,001 per year. The national government has made an assessment on the allowable risks considering economic damage and (a monetary value for) loss of life; the required level of safety is thus a political choice.

The safety approach is based on statistical characteristics of the water level. Because the water levels in the Dutch polders are strictly regulated and measured, there is information available on the probability of exceedance of different water levels. A measure for the spread around the mean water level is the decimation height. The decimation height is the water level difference with a ten times smaller probability of exceedance, which is for most Dutch polders in the order of 0,05 to 0,10 m.

All the dikes are assessed every 6 years to check if they are able to withstand the water level corresponding with the safety target. To execute the assessment as efficient as possible there are three levels of assessment. The first level is conservative and simple, if the dike passes the first level it is safe enough; if not a more detailed and less conservative assessment is performed in the second level. If the dike does not pass the second level, a third even more detailed and les conservative assessment is performed.

For each level of assessment the dike is divided into sections which have similar strength characteristics, both in height and dike material. Each section is then individually assessed. A section could be in the order over 25 meters to several hundreds of meters, depending on the spatial variation.

4.1.2 Level 1) simple assessment

The simple assessment exists of two possible paths which are described below.

Comparison with the past

The first step in the assessment process is the simple assessment. In this step the dike is assessed on crude aspects of the dike. The target is to prevent labour intensive calculations for sections which are definitely strong enough.

For each section the first step is a comparison with the previous assessment of 6 years ago. The hydraulic conditions are compared together with the physical condition of the dike. If all loads and physical aspects have not changed for the worst the section will be marked safe enough. Note must be given that the above only holds if the assessment method has not changed in the past years.

Robust profile

If one of the conditions has changed the next simple assessment is performed. The profile of the section is compared with a profile with conservative dimensions. The conservative profile includes minimum dimensions of the crest width, gradient of the slopes and the presence of objects in or on the dike. The dike is then compared with the conservative profile; if the dike fulfils all the minimum dimensions the dike can be marked as safe enough.

To compare the dimensions, the height and width of the dike should be mapped; this is possible through an analysis of the AHN2 or AHN3 files. These files visualize satellite measurements of the current height of a part of the Netherlands. Another more costly option is to conduct a land survey to map the profile of the dike sections.

If a dike is robust enough to fulfil the ‘safe profile’ requirements the dike is considered sufficiently safe. If the dike does not meet the requirements above an advanced method should be applied to check if the dike is safe enough. When it is obvious the dike will not pass the advanced assessment, the advanced assessment is not performed to conclude the dike is not safe enough. Advanced computations are performed in order to decide on the necessary level of reinforcement.

4.1.3 Level 2) Advanced assessment

A method which is used a lot in the Netherlands is the advanced assessment. The main goal of this step is to schematize a cross-section of a dike and next to compute the safety factor, based on Table 3, of the schematized cross-section. The first step in the advanced assessment is the (re)determination of the dike sections. The dike is divided in a limited amount of sections, which are chosen so that the differences inside 1 section are small enough (i.e. profile, geology) but the amount of sections isn’t too big.

For each section a stability computation is performed with the software package D-geo stability. This software is developed by Deltares and is used for most of the stability computations in the advanced assessment. In the stability calculation the safety factor of the dike is computed; if the safety factor is larger than the required safety factor corresponding to the IPO class (presented in Table 3) the dike can be marked ‘safe’. For the computation the section of a dike must be schematized according to schematization rules and expert judgement.

Profile schematization

For each section a representative profile is be determined, this could be the minimum envelope of the entire section, the cross section of the weakest part of the section or any other representation seemed fit by the engineer. The cross-section is the basis for the stability computation.

Soil

The next step is to determine the layering of the soil and the soil parameters. This is usually done by performing cone penetration or drilling tests. The location of the tests is determined by the engineer and is based on (expected) differences in the profile. In 1 cross-section two or three measurements are conducted to compose the course of the soil layers. From the CPTs ‘individual’ soil layers are identified. In this way the soil profile perpendicular to the dike is schematized.

For each soil layer the strength characteristics is be determined. There are two main options to determine the strength parameters, the first is to test the drill samples and to determine the strength characteristics. The second option is only possible if the water board has a large database of the characteristics of soil samples. From a large set of samples a representative value can be chosen which belongs to the soil types which are found in the CPT. The advantage of the use of a large data set is that the spread in soil characteristics of a specific soil is accounted for; which makes is statistically more reliable.

The strength characteristics which are used in the computation are based on a semi probabilistic level 1 method. The uncertainty of the parameters is accounted for with the use of 1 characteristic value which describes the lower boundary for the strength; the lower boundary depends on the acquired level of safety prescribed in the guideline ‘material factors for drainage canals’ (STOWA, Materiaalfactoren boezemkaden, 2009). For each IPO class a material factor is determined for the effective angle of internal friction (φ') and for the effective cohesion (c').
The value for φ' and c' which is found from either the drill samples or the databased is divided by the safety factor (>1).

Hydraulic loads

One of the most important aspects in the computation are the hydraulic conditions inside the dike. The pore pressures determine if the dike is or isn’t stable enough. The pore pressures depend on the outer water level, inner water level, seepage from the aquifer and infiltration by precipitation. For each of the boundaries a value is be determined.

The design water level is determined by the hydraulic system and the IPO class of the system. For the inside water level usually the winter water level of the polder or the polder surface level is used.

Pore pressure schematization

The most important step is the translation from the boundary conditions to a field of pore pressures inside the dike. Therefore multiple tools are available like finite element software like Plaxflow or M-Seep. The use of finite element software is however labour intensive and is not often used for relatively simple regional flood defences. Instead the guideline ‘technical report pore pressures in dikes’ is recommended by STOWA. In this guideline the translation from boundary conditions to the schematization of pore pressures is described.

If no head measurements are available the following approach is advised for the schematization of pore pressures in a regional flood defence regarding macro stability. In the schematization distinction is made between four material configurations:

- Material of the dike itself: sand or clay;
- Material of the base of the dike: sand or clay;

To illustrate the assessment approach the method for a clay dike on a sand base is described below. The example includes all the important schematization aspects regarding the pore pressure. For other configurations the method is slightly different. The total description can be found in the TRWD (TAW, 2004).

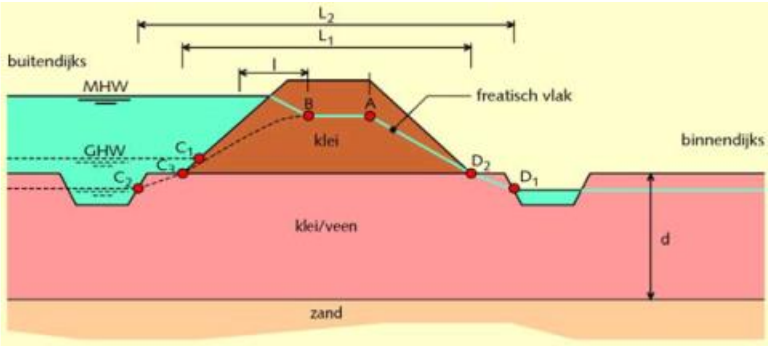


Figure 22 Schematization guideline phreatic surface for a clay dike on a sand base (TAW, 2004)

The schematization consists of four points:

A. Point A represents the influence of precipitation. The height of point A is determined by the width of the dike divided by a factor plus the lowest value of C and D. The height of the phreatic surface is for a clay dike only depending on the width of the dike. The maximum height is 0,3 m below the crest;
B. Point B represents the heightening of the phreatic surface due to a high outside water level. In the computation of the horizontal position of point B is determined by the penetration depth of the high water level; this depends on the duration, the average permeability of the dike and the water level increase in the canal;
C. Point C is determined by the outside water level rise in case of an extreme event;
D. Point D is determined by the water level inside the polder;

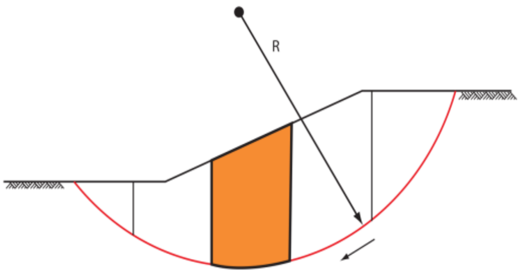
What can be concluded from the schematization rules is that the (in-stationary) effect of a high water level is included. The effect of precipitation is included in the rule for point ‘A’ and does not depend on the permeability of the dike or the base. (For a dike consisting of sand the precipitation is not taken into account, no explanation is given)

In an addition to this method head measurements in the dike can be used to determine the daily average head level inside the dike.[2]

[2] In practice daily stable phreatic surface does not excists; the head measurements give however a good indication if the standard schematization is valid or not.

Stability computation

When all the characteristics are schematized the stability computation can be performed in D-geo stability (abr. D-stab). For the stability computations the method of Bishop is used which determines the safety factor along circular slip circles with the method of slices. The soil body is divided into a number of vertical slices, for each slice characteristic values (i.e. load and resistance) are computed which are used as representative value for that slice. With the use of the characteristic values for the soil strength parameters and the hydraulic condition, the driving moments (i.e. water and soil) are compared with the resisting forces (soil). For each possible slip circle the factor of safety is computed according to the following relation between the driving and the resisting forces which are described in detail in chapter 2.
Another option is the method Uplift-Van; this method uses two circular slip circles in combination with a horizontal slip surface. For cross sections with a weak layer in combination with a drainage ditch near the dike the horizontal slip surface could occur. The Uplift-Van method requires however more computation time; in practice the Bishop method is mainly used when possible and the Uplift-Van method is used when required.



$$F_s = \frac{M_{R;soil} + M_{R;end\ section}}{M_{D;soil} + M_{D;water} + M_{D;load}} \quad (5)$$

Figure 23 Slip circle Bishop (Deltares, 2015)

4.1.4 Level 3) Detailed assessment

The detailed assessment is even less conservative as the level 2. A level 3 method can consist of several improvements on the level 2 method, for example the use of more detailed data or the use of a more advance software package for the stability computation. The advanced assessment can be performed if either of the following two situations occur:

- The dike has irregular characteristics which cannot be simulated in the advanced assessment;
- The dike has failed the advanced assessment but has high potential to pass the detailed assessment;

In practice many regional flood defences are relatively simple structures which protect areas with a relatively low economic value. Therefore a level 3 assessment is usually too expensive for the project; instead the dikes which are not safe enough are heightened up to the level required by the level 2 assessment. Because the probabilistic method which is studied in this thesis is based on a level 2 method no further information on the level 3 method is provided in this report. For more information on the level 3 method is referred to the governing guidelines (STOWA, 2015).

4.2 Probabilistic assessment

The main advantage of the probabilistic method is that one can determine how safe the dike is instead of only 'if' the dike is safe.

A method which determines the failure probability of a dike is first composed for the primary flood defences and is called 'Veiligheid Nederland in Kaart 2' (VNK2) which translates to 'View on the Dutch Safety 2' (VNK project office, 2012). The VNK2 project is founded by the Dutch government and the water boards in order to map the strength of the Dutch primary flood defences. In this assessment the combination of probabilities and consequences of failure are investigated and combined in one final report. By combining these two a more valuable prioritization can be made for the reinforcement of the dikes which have the 'highest' risk (which is the combination of failure probability and consequences).

The probabilistic method used for primary flood defences was applied to a system of regional flood defences and documented in: 'Flood risk regional flood defences' (Lendering, Jonkman, & Kok, 2015).

The basis for the failure probability analysis is the limit state formula:

$$Z = R - S \tag{6}$$

In which 'R' is the resistance of the dike and 'S' the solicitation/load on the dike. Both the load and the resistance are a probabilistic distribution with a mean value and a spread around the mean. The probability that the difference between the resistance and the load is smaller than zero ($Z<0$) describes the probability the dike will fail to retain the water.

In order to compute the probability of failure statistical information is needed on all the influencing factors, from the distribution of the hydraulic load to the distribution of the soil strength parameters.

In the next chapter a short description of the entire method to determine the failure probability of a regional flood defence is given. Because this study is focussed on the effect of the hydraulic load more attention will be given on that topic.

The core of the probabilistic method is to determine the probability density function of the limit state formula. In principle the same method and parameters are used as in the deterministic assessment. So in the probabilistic calculation the stability of multiple possible slip circles is calculated by comparing the driving forces with the resisting forces.

The main difference is that to use the limit state formula statistical information on the load and strength characteristics is needed. That means that each parameter now has a mean value and a spread around the mean instead of a characteristic value. For each parameter a distribution is defined based on either measurements or expert judgement. The type of distribution per parameter is listed in Table 4.

Variable	Distribution type
Water level in canal	Empirical
Phreatic level inside flood defence	Empirical
Water level in polder	Normal
Top load	Empirical
Retaining level flood defence	Normal
Level of toe flood defence	Normal
Width of flood defence	Normal
Angle of inner slope	Normal
Angle of outer slope	Normal

Table 4 Distribution type parameters macro stability (Lendering, Jonkman, & Kok, 2015)

4.2.1 Hydraulic load

The hydraulic load is the main driving factor for macro instability; the hydraulic load consists of two aspects in this study: the outside water level and the phreatic surface. Both influence the safety against macro instability.

Water level

The variation of the outside water level is well known for most drainage canals in the Netherlands. Therefore the main values and probabilities for annual exceedance are relatively easy to compose with a peak over threshold method or the annual maxima method. An important aspect described in 'Flood risk regional flood defences' (Lendering, Jonkman, & Kok, 2015) is the drain stop level.

Many Dutch polders have a policy to stop pumping water into the drainage canal if the water level exceeds a predefined level. This is to prevent failure of the flood defences along the drainage canals. Therefore the probability density function of the water level has a peak around the drain-stop level as showed in Figure 24.

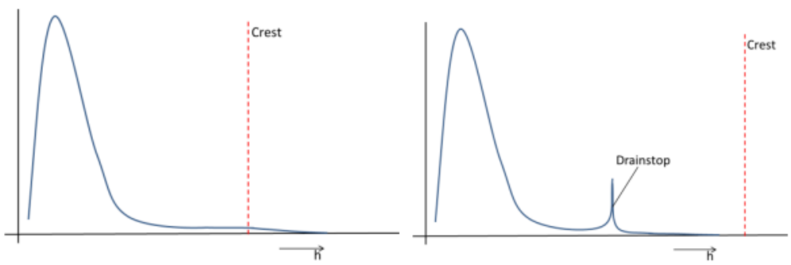


Figure 24 Probability distribution of water levels primary flood defences (left) and regional flood defences (right) (Lendering, Jonkman, & Kok, 2015)

In the computation for the failure probability the uncertainty of the water level is taken into account by regarding 4 discretized water levels which describe the entire distribution of the water level. The four levels are al low level, the average level, the drain stop level and the highest measured water level for the polder system. For the 4 water levels the conditional failure probability of the dike is computed; by integration of the conditional failure probabilities the unconditional failure probability is computed.

Phreatic surface

In a regional flood defence the phreatic surface is near to the surface in a daily situation. Therefore the possibility of macro instability is much larger for a regional flood defence than for a primary flood defence. It is important to include the uncertainty of the phreatic surface in the macro stability calculation.

There is however no statistical information of the phreatic surface available. Therefore an empirical relation must be compose which describes the relation between the probability of exceedance of the rise of the phreatic surface and the variables which influence the phreatic surface (i.e. precipitation, water level). The following approach is used: the phreatic surface inside a dike can either be low, normal or high. The normal phreatic surface is the surface one could expect (or measure) in daily circumstances. The high phreatic surface is based on the schematization prescribed in the TRWD possibly extended with field measurements. The low phreatic surface corresponds with a dry situation described in the TRWD possibly extended with field measurements.

For each level the conditional probability of failure is computed. In order to integrate over the conditional probabilities of failure, the probability of occurrence of the different levels of the phreatic surface is determined by expert judgement. The following values are chosen which are also depicted in Figure 25:

- *Drought with a probability density of 5%;*
- *Average with a probability density of 85%;*
- *Saturate with a probability density of 10%;*

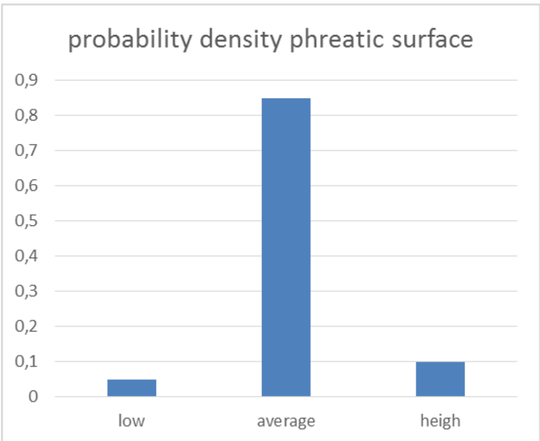


Figure 25 probability density level phreatic surface

4.3 Concept framework to assess the uncertainty of the phreatic surface

The main target of this report is to provide information on the uncertainty of the phreatic surface to use in the probabilistic method. There is great uncertainty in how to schematize the pore pressures in a representative way. Therefore in this report a more scientific based method is developed to schematize the uncertainty of the phreatic surface. In this chapter the concept of a method to include the uncertainty of the phreatic surface is presented. The concept provides a short framework which describes the steps necessary to give an answer to the main research question.

'How to assess the uncertainty of the phreatic surface in a regional flood defence influenced by precipitation and the water level?'

The framework is based on the current probabilistic method; in the method the software package D-Geo Stability is used. This package provides a reliability tool in which the conditional failure probability can be computed for several combinations of water level and level of the phreatic surface.

The scope of the method includes the precipitation (duration and intensity) the geometry of the dike, the relation between the precipitation and the water level and the relation between the phreatic surface and the precipitation. The following aspects are not included in the scope: influence of high water level on the phreatic surface, heterogeneity of the dike material and the shape of the precipitation events. The reason for this limitation is to focus on the effect of precipitation on the phreatic surface. It is recommended that in a later stage the combined effect on the phreatic surface of a water level change and precipitation is studied.

4.3.1 Schematization pore pressures

For a regional flood defence the mean water level is near to the crest, therefore the neutral phreatic surface is always near to the surface. Because the water level difference between a normal and extreme situation is small and the phreatic surface is situated so high, the effect of precipitation on the phreatic surface can be significant compared to the effect of a water level change. Therefore it is important to include the effect of precipitation in the uncertainty of the phreatic surface.

Schematization of the soil

The effect of the precipitation depends for a large part on the hydraulic soil characteristics. In very permeable soil types the precipitation water will infiltrate and dissipate faster than in less permeable soil types. Therefore the first step in the method is a schematization which describes the soil characteristics. How the schematization must be executed depends on the variability of the effects of precipitation between soil types and degrees of permeability. In chapter 5 and 6 two studies are performed on the effect of precipitation on the pore pressures.

Influence precipitation

Because the TRWD does not describe the effect of precipitation on the phreatic surface, the first step towards determining the uncertainty of the phreatic surface is to determine what the effect of precipitation can be. Precipitation events vary in length, intensity and shape; it is thus important to know what the effect can be of events with different intensity-duration ratios. When the effects are known it is possible to propose a method to schematize the influence of precipitation.
To determine the uncertainty of the phreatic surface the first step is to isolate the effect of the precipitation on the uncertainty. Therefore a method must be developed to relate the uncertainty of precipitation to the uncertainty of the phreatic surface; this aspect is elaborated in chapter 7.

Influence high water
The influence of the water level on the phreatic surface is already included in the current probabilistic method. However the uncertainty in the phreatic surface due to a combination of a high water level and extreme precipitation is not yet included. Information must be gathered on the combined effect; an approach to do so must be developed, this is out of the scope of this study. This study does include an approach to describe the correlation between both loads; this is necessary because the water level is influenced by the precipitation.

4.3.2 Failure probability computation

As is mentioned in the introduction, the computation is performed in the reliability analysis module of d-geo stability. The reliability module gives the option to enter 4 different levels of the phreatic surface. For each level the conditional probability of failure is calculated; by integration of the conditional failure probabilities, the total failure probability regarding macro stability is computed.

When the uncertainty of the phreatic surface is determined 4 levels should be selected to be simulated in the failure probability analysis. Therefore a justified method must be developed to choose a maximum of 4 levels of the phreatic surface (with corresponding conditional probability of occurrence).

4.4 Conclusions

The current assessment method is based on a level 1 probabilistic method. The uncertainty in soil strength parameters is taken into account with the use of design parameters and applying safety factors. The effect of the outside water level is based on the required level of safety and is acquired from a series of water level measurements. The effect of precipitation is not described but is (sometimes) taken into account with a conservative schematization; the height of the phreatic surface in the computation is increased up to 0,30 m below the crest. The end result of the assessment is an answer on the question ‘is the dike safe enough?’

The probabilistic method takes into account the uncertainty of all the parameters in order to provide information on how safe the dike is. Therefore statistical information must be gathered on all the parameters. The uncertainty in the water levels can be gathered from the water level measurement series. The uncertainty in the phreatic surface is based on the TRWD combined with expert judgement on the conditional probability of occurrence. The end result of the assessment is ‘how safe is the dike?’

The way to provide a more scientific base for the uncertainty of the phreatic surface consists of three steps. Firstly the effect of precipitation must be studied and identified; secondly the relation between the uncertainty of the precipitation and the uncertainty of the phreatic surface must be described; the third and last step, is to compose a method to integrate the conditional probability of failure for different levels of the phreatic surface.

5 Analysis field test artificial precipitation on a peat dike

In order to include the uncertainty of the phreatic surface in a failure probability analysis the influencing factors should be quantified. The main hydraulic aspects which influence the phreatic surface are the precipitation and the water level (difference). The effect of the water level difference is already included in the deterministic and probabilistic assessment. For that reason no extra attention is given to this subject in this study.

Because the exact influence of precipitation on the pore pressures is unknown, the first step is a study on the kind of effect of precipitation on the phreatic surface in a real dike.

A field test with artificial precipitation on a dike consisting of peat is conducted by the water board 'Hoogheemraadschap Hollands Noorderkwartier', further referred to as 'HHNK'. To gain insight in the effects of precipitation on the pore pressures the data is the field test is reanalysed. The analysis of the test data gives valuable insights in which aspects should be further investigated to complete the computation method.

The target of the analysis of the field test is:

Target: *A description of the effect of precipitation on the development of pore pressures inside a dike, to be able to determine which aspects should be included in a computation method.*

In this chapter a short description is presented of the field test together with a brief analysis; the detailed analysis of the results can be found in Appendix: F.

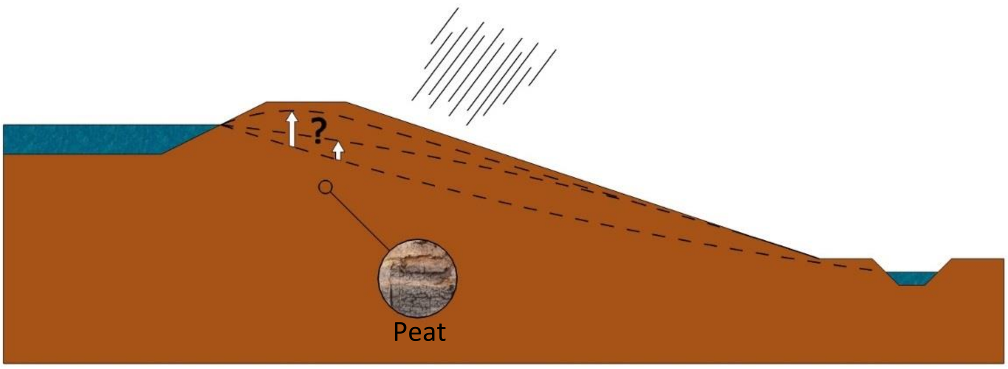


Figure 26 research target Ch. 4 : influence of precipitation on the pore phreatic surface inside a peat dike

5.1 Background information field test artificial precipitation

The study by the HHNK was conducted in 2012 and is documented in the report 'The effects of artificial precipitation on the phreatic surface in peat dikes' (Alterra, 2013). The goal of the study was similar to this one, namely to gain information on the effect of precipitation on a peat dike. Because the report did not present the necessary details, the original data of the test is studied.

In the study by the HHNK artificial precipitation is used to simulate an extreme precipitation event. During the test, the pore pressures on different locations in two cross section were measured; one location received irrigation water and the second section served as a reference. A full description of the test is included in Appendix: F.

The field study is conducted near the city of Purmerend on two dikes on opposite sides of a drainage canal showed in Figure 27. Both dikes consist largely of clayey and peaty materials and are founded on a sand layer at roughly NAP -6 meters.

To schematize the geometric profile of the dike, a land survey is conducted. Cone penetration tests are performed in 2 section lines with 12 CPTs per line. At the points of the cone penetration tests, head monitoring wells are installed to measure the effect of precipitation. The filters at the bottom of the monitoring wells are situated in clay or peat; some of the filters are partially situated in the sand layer underneath the clay and peat which can be seen in Figure 28.

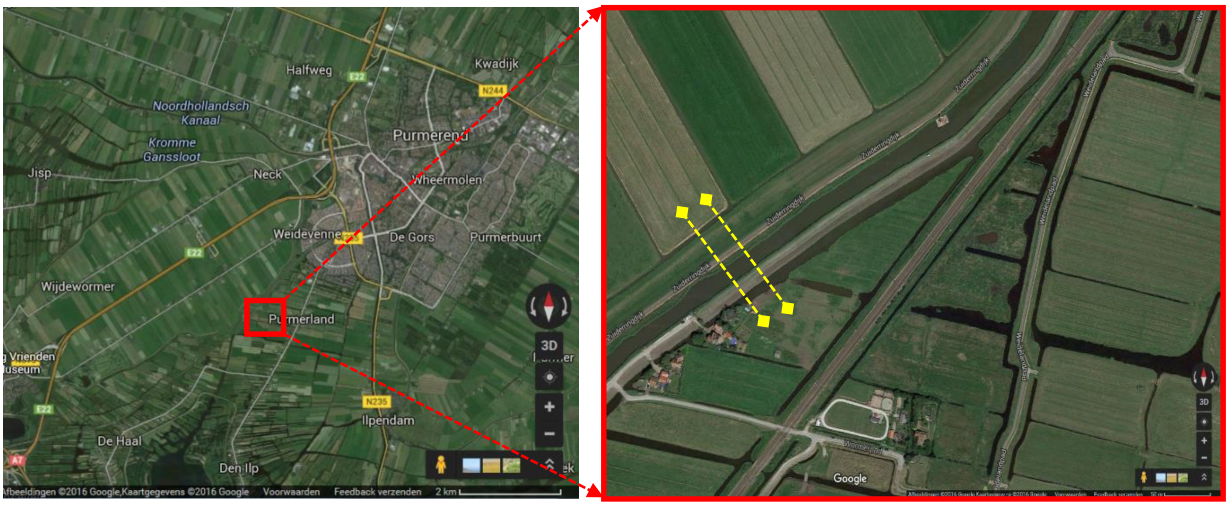


Figure 27 Location field test HHNK (Alterra, 2013)

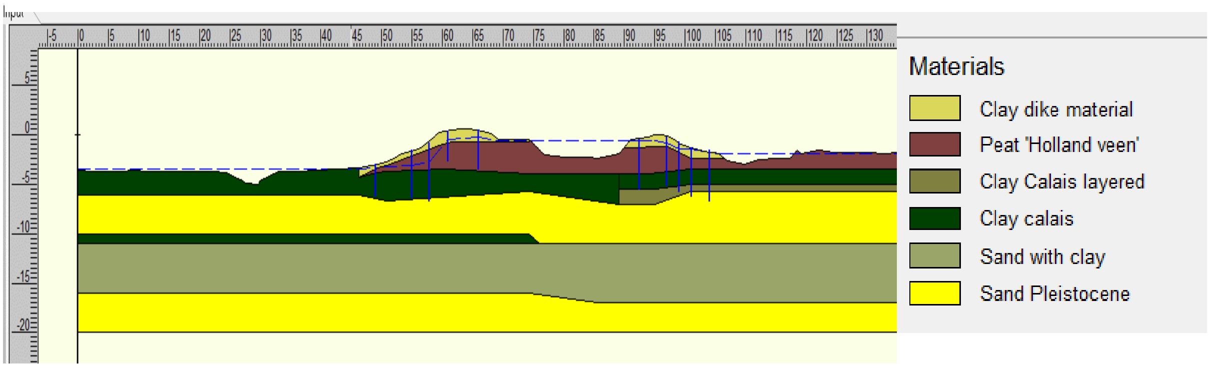


Figure 28 Schematization test site (Alterra, 2013)

The main part of the test is a series of events which should simulate a 1/100 per year event. The duration and intensity of the event is based on statistical analysis of the precipitation in the Netherlands by STOWA; in (STOWA, 2004) the extreme precipitation is presented as values for the depth-duration frequency (DDF) curve. The DDF curve is available for durations between 4 hours and 9 days.

In advance, it was not known what type of precipitation event would cause the pore pressures to rise; a permeable soil will react on a short and intense event and a less permeable soil will react more on a long and less intense effect. Therefore, each soil type or combination of soil types has a critical depth duration combination. To simulate both the short and intense events and the long and less intense events in 1 test, the following approach is chosen: on the first test day, the depth duration ratio of a 24 hour event is used and simulates a 1/100 year event of 24 hours. On the second day, the amount of irrigation equals the difference between a two day and a one day event. This is repeated for 9 days in total. The values used as irrigation gifts are listed in Table 5. The actual amount of irrigation was measured and is included in Appendix: F. With the use of this method, the event that is simulated, is an event with a peak in the beginning and a low tail.

Duration [days]	1	2	3	4	5	6	7	8	9
1/100 event [mm]	79	92	102	109	118	125	131	137	143
Difference [mm] (irrigation gift)	79	13	10	7	7	7	6	6	6

Table 5 Depth/duration 1/100 per year

One of the goals of the study was to determine a relation between precipitation intensity, duration and the rise of the phreatic surface; therefor it is necessary to know how much water does really infiltrate in order to compose a water balance. Therefor a large runoff is prevented by testing the infiltration capacity prior to the main test. Before the main test, the dike was irrigated with an intensity of around 9mm/h. The intensity was slowly diminished until the run-off was minimal. The intensity which fitted the infiltration capacity was 5,4mm/h (with a runoff of 0,1mm/h).

The test is performed such that the precipitation intensity is just under the maximum intensity for half an hour. The next half an hour the irrigation is stopped. This sequence is repeated until the desired total amount of irrigation is applied.

During the test, natural precipitation occurred as well; the total amount of precipitation is measured and documented in Appendix: F.

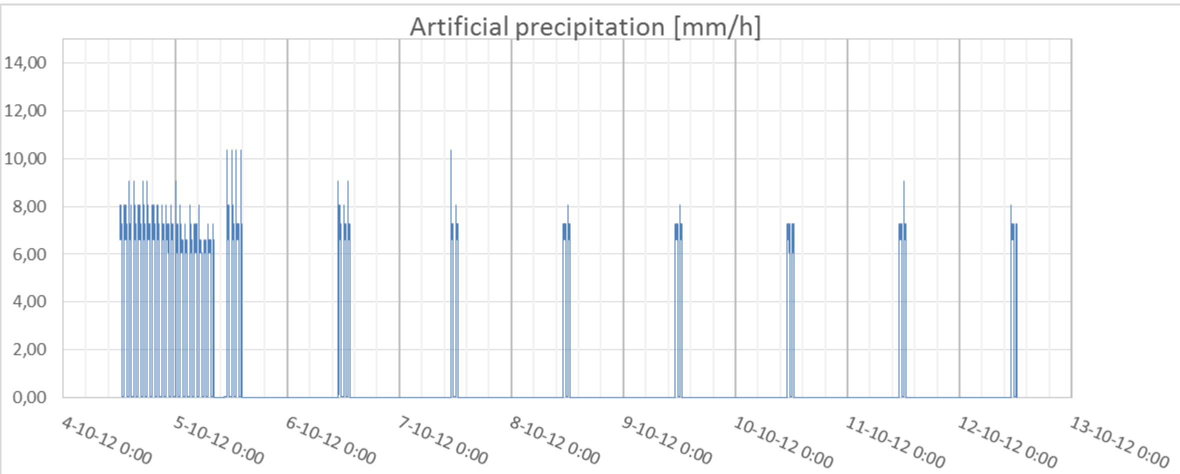


Figure 29 Artificial precipitation Oudelandsdijkje

5.2 Results of the field test

During the artificial precipitation events the actual precipitation is measured every hour; this includes the intensity of the irrigation plus the natural precipitation. Furthermore the head in the monitoring wells is measured every 15 minutes. To gain insight in the effect of precipitation on the head in the dike, both data series from the ‘Oudelandsdijkje’ are visualised in Figure 31.[3] A full analysis of the results can be found in Appendix: F.

In the analysis of the results special attention is paid to the reaction speed and size on precipitation events with a different duration and intensity.

- With small events only the head in the toe of the dike reacts;
- It takes 2 to 4 hours for all the head monitoring wells to respond to the 1/100 year water gift on day 1;
- With a precipitation of 93mm (artificial plus natural precipitation) on the first day, the rise of the pore pressures in the head monitoring wells was 0,2 m to 0,4m;
- The short irrigation events on day 2 to day 9 resulted only in a head rise of 0,05m to 0,01m. These events do not last long enough (only 1,5 to 2 hours) to cause significant change;
- A natural event of 2mm/h for 4 hours caused a head rise of 0,1 to 0,3m;
- At the Zuiderringdijk there are two filters at only 2m below the surface, these respond almost instantaneously on precipitation;
- The control field of the Zuiderringdijk shows a quick response on natural precipitation with a head rise of 0,5m due to an event of 6 hours with a maximum intensity of 4,2mm/h and an average of 1,5mm/h. The reaction is different than in the three other locations;
- Due to a small road on the Zuiderringdijk the response in the head monitoring well on the outer crest line is negligible;

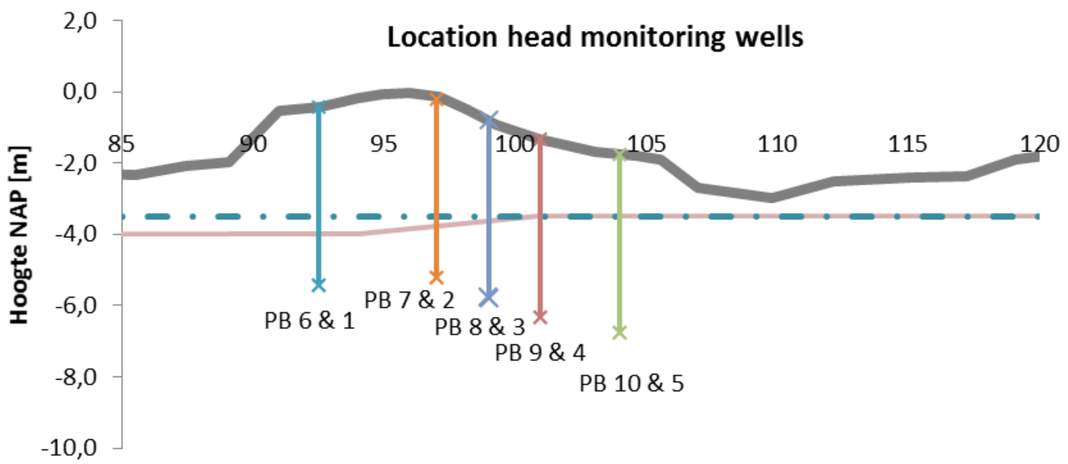


Figure 30 cross section and head monitoring wells ‘Oudelandsdijkje’

[3] Note that the scale of both graphs is different; because the measurements of Pb5 and Pb6 are not correctly recorded it is possible to zoom in, and show more detail of the remainin three data series.

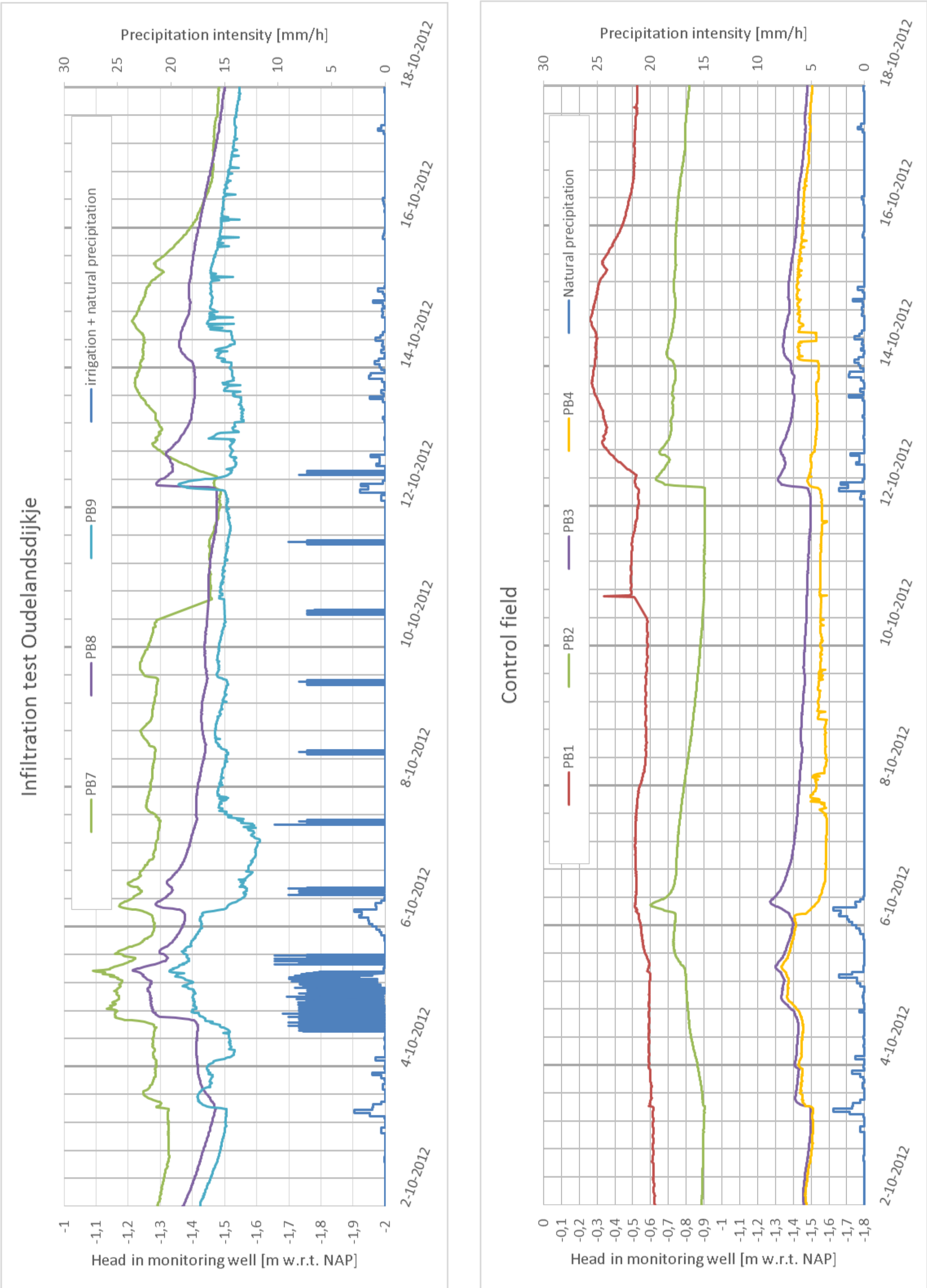


Figure 31 Head measurement series infiltration field (left) and reference field (right) 'Oudelandsdijkje'

5.3 Analysis

Regarding the situation with only natural precipitation, a rise of the pore pressures of 0,3m underneath the crest and 0,1m underneath the toe of the dike is observed. The 'Zuiderringdijk' shows similar results. The results indicate that the influence of a normal precipitation event, with an average return period larger than once per year, can cause the pore pressures to rise notably.

In the 4 test locations (2 infiltration sites and 2 reference fields) the reaction speed and the reaction size differed; the size of the reaction in the reference field on the 'Zuiderringdijk' was larger than expected; the rise of the phreatic was up to NAP-0,49m. Because the rise was larger than the predefined limit the test was stopped for safety reasons. Evidently, the soil characteristics at this location differed from the other 3 locations. This means that it is highly possible that 'weak' or 'wet' spots are present between measurement points. In this case the 'wet spot' was probably caused by rubble inside the dike; rubble is much more permeable so the water can infiltrate faster.

For most precipitation events in this test series, the time the pore pressures needed to respond on the precipitation is in the order of several hours. The reaction of the pore pressures on shorter, but more intense precipitation events was less. Furthermore, events with a higher intensity seem to influence the pore pressures faster than events with a low intensity; this means that in normal circumstances the permeability of the dike was high enough to let all the precipitation water infiltrate.

5.4 Discussion

When the test itself is regarded there are a couple of noteworthy aspects:

- Precipitation pattern: the chosen pattern of the precipitation event is with a peak at the start of the event and a large and low tail. In reality, there are 7 different precipitation patterns identified including the event with 1 high peak (STOWA, MeteoBase, 2016), therefore the probability of exceedance of the simulated precipitation event is less than the intended 1/100 per year.
- The measured pore pressures in the head monitoring wells is linearly translated to the phreatic surface without taking into account the head loss due to the downward infiltration in the dike. This means that the phreatic surface is probably higher than computed.
- The irrigation intensity is based on the maximum infiltration capacity in the dike; prior to the test a test is performed to check at which precipitation intensity run-off is starting to occur. Because the initial infiltration capacity was relatively high, the initial amount of precipitation was set very high. Therefor the simulated precipitation events where very short
- This resulted in a short gift of a large amount of water followed by a long dry period (23hrs). The test shows that the influence of the short irrigation periods is minimal. This finding should be taken into account when repeating similar tests in the future.

5.5 Conclusions

The results from the test show a variety in results, even between the 4 sections which are located close to on another; therefor it is not possible to, based on the results from this test only, develop a general method to describe the influence of precipitation on the phreatic surface. Nonetheless, the test does give insight in which aspects should be taken into account and to which extend the precipitation influences the pore pressure.

The first question which can be asked is: Is it necessary to include precipitation in the computation method? The answer to this question is positive. The pore pressures at several meters below the surface react significantly on both natural (daily) as artificial precipitation events. The results show that the size of the reaction of natural precipitation in this case is 0,1 to 0,3 meters. The reaction to the extreme artificial precipitation was in the order of 0,2 to 0,4 meters, which can have a significant influence on the stability.

Another aspect that is important to include, is the uncertainty of the permeability of the soil. The reference field of the Zuiderringdijk had a higher permeability, probably caused by rubble inside the dike, which caused the pore pressures to react sharper on a precipitation event. Therefore, the permeability influences the extent of the effect and this should be taken into account.

Two studies on the effect of precipitation on the pore pressures ((Hamdhan & Schweiger, 2013) & (Arnold & Hicks, 2011) concluded that it takes a day or more for the precipitation to influence the pore pressures; however, this study shows that the pore pressures react on the precipitation in a matter of hours. This difference shows the influence of different soil configurations on the reaction speed of the phreatic surface. There is thus a wide range possible in the reaction speed of the pore pressure, this should be accounted for in future schematization.

6 Numerical study effect precipitation on pore pressures

The next step is to find an answer to the following question:

'*How do the pore pressures, in different types of soil, develop under the influence of precipitation?'*

In literature there is some knowledge available on the effect of precipitation on the pore pressure inside a dike. Unfortunately the knowledge is limited to case studies that focus on the representation of measured pore pressures in a finite element model instead of a description of the effect of precipitation on pore pressures in general. In order to provide a general method, which takes precipitation into account, it is necessary to know something of the basic behaviour of the pore pressures due to precipitation. To develop a general understanding on the effect of precipitation on a dike, that can consist of different soil types, the following target is reached at the end of this chapter:

> ***Target:*** *A qualitative description of the differences of the phreatic surface between soil type (i.e. sand, clay and peat) in the size of the pore pressure change and the corresponding time scale.*

The method that is used to investigate the general behaviour of the pore pressures is a study with a Finite Element Method (abr. FEM) called Plaxflow. In the previous chapter, valuable information is gained on which aspects should be studied in the FEM (i.e. reaction speed of the pore pressures and the effect of permeability).

The link between the phreatic surface and the precipitation is studied in order to qualify the relation. The aspects that are studied, are the effects (i.e. size and speed) of precipitation on three soil types with different hydraulic characteristics. In chapter 6.1 the general and technical information of the Plaxflow model is described. In chapter 6.2 the test set up is elaborated and in chapter 0 the results of the computations are presented. In chapter 6.6 an answer to the research question of this chapter is provided.

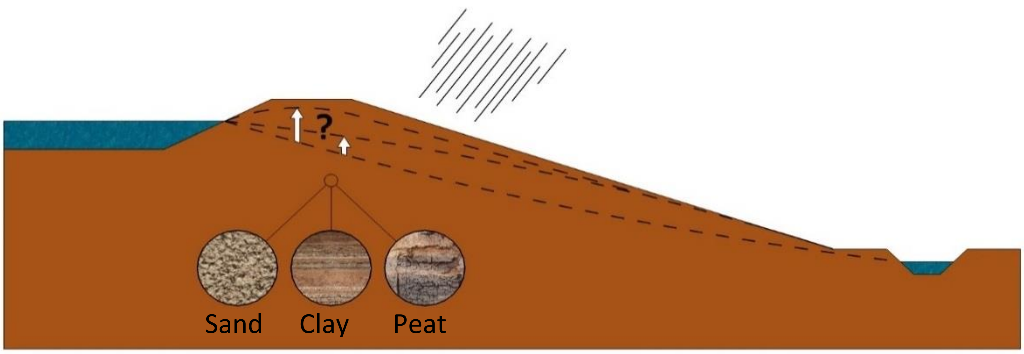


Figure 32 Research goal Chapter 6: general information development pore pressures due to precipitation

6.1 General information on a finite element model

In order to determine the general behaviour of the pore pressures inside a dike, a hydraulic model is developed in a 2D Finite Element Method (FEM) called Plaxflow. The advantage of Plaxflow is that it is highly useful to study the basic behaviour of ground water flow through both saturated as well as unsaturated soils for discrete time intervals. The use of Plaxflow is recommended by the 'Technische Adviescommissie Waterkeringen' (TAW) in the Technical Report Pore Pressures in Dikes (TAW, 2004).

The first modelling step is the schematization of a cross-section of a dike. In reality, a regional flood defences consist of multiple materials and a complex shape of layers. The cross section in Plaxflow can be composed of multiple soil layers, but for this study a homogeneous dike is chosen. The reason that the dike is modelled as a homogeneous soil body is to find typical differences between soil types. In future research the typical behaviour can be used as a basis for more detailed schematization.

The finite element model divides the cross-section in many small element. Plaxflow this grid consists of many small triangles, each with varying properties depending on the location and the prescribed soil characteristics. The user can determine the amount of triangles, which determines the accuracy of the final result. The grid size also determines the calculation time; the smaller the grid cells are the longer it takes to compute the end result. Therefore a grid is chosen such that it small enough to present useful results but not so small that the computation time is too long.

For each triangle, basic hydraulic computations are performed. The basic for each computation is the continuity equation; together with Darcy's law and a formulation of the SWCC the state of each element is computed per time step. The flow is forced by the hydraulic boundary conditions such as the water level (difference) and precipitation. The boundary conditions are user defined and can simulate different situations like a stationary water level or a changing inflow or outflow in the model.

There are two computation possibilities that can be used; a stationary and a transient computation. The stationary computation is not time dependent and computes an equilibrium situation given stationary boundary conditions. The transient computation is time dependent and can include changing boundary conditions. The transient computation module is ideal because it can simulate a precipitation event with a limited duration.

With the transient module different precipitation events can be simulated. The event is divided in many small time steps, for each time step the infiltration is computed; the amount of infiltration determines the degree of saturation, and thus the permeability for the next computation step.

The output of the model provides many possibilities. The most important one is the possibility to visualise the development of the pore pressure during a precipitation event on any point in the dike.

6.2 Test set up

The main goal is to determine how the pore pressures in a dike respond to precipitation. Therefor a model is composed in order to represent a typical Dutch regional flood defence. Various test are performed with the model. First a general description of the tests, and the different purposed is provided. In the next paragraphs a detailed description in the test variables is described.

The target of the simulations is to obtain information on the following topics, all combined with precipitation:

- difference between soil types;
- effect of saturated and unsaturated characteristics;
- development of the pore pressures over the depth of the dike;
- development of pore pressures over the cross-section of the dike;

To obtain information on the 4 topics, for simulations are composed which are listed in Table 6. Each test has its own goal and computation characteristics. Table 6 provides a schematic overview of the test characteristics, in the remainder of this chapter the reasoning behind the test set-up is provided.

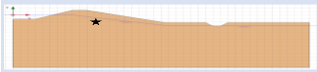
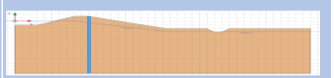
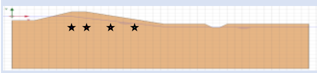
	Goal	Computation	
1a	Gain insight in typical differences between the three soil types.	• 3 soil types with default parameters; • Precipitation intensity: 0,25mm/h; Graphical output of the pore pressures in the entire dike	
1b	Insight in the differences between soil types; influence of permeability and the soil water characteristic curve (SWCC)	• Permeability of all soil types equal; • Precipitation intensity: 0,25mm/h; Output: visual comparison of pore pressure development at 1 point inside the dike	
2	Insight in development of pore pressures over the depth	• 3 soil types with default parameters; • Precipitation intensity: 0,25mm/h and 20mm/h; Output: ground water head underneath the inner crest line	
3	Insight in development of pore pressures over the length of the dike	• 3 soil types with default parameters; • Precipitation intensity: 0,25mm/h; 4 measuring points over the length of the profile.	

Table 6 List of computations behaviour phreatic surface

6.2.1 Geometry

The profile is composed in such way that it represents a typical Dutch regional flood defence; it is based on the minimum requirements of a regional flood defence according to the 'guideline assessment for regional flood defences' (STOWA, 2015). An important aspect of a regional flood defence is that there is a permanent water level difference over the dike; this is included in the boundary conditions. The dimensions of the simulated dike can be found in Table 7 and in Figure 33.

Outer slope	Crest width	Inner slope	Water level difference	Water depth drainage canal
1:2	2 meter	1:3	3 meter	1,2 meter

Table 7 Dimensions dike in FEM

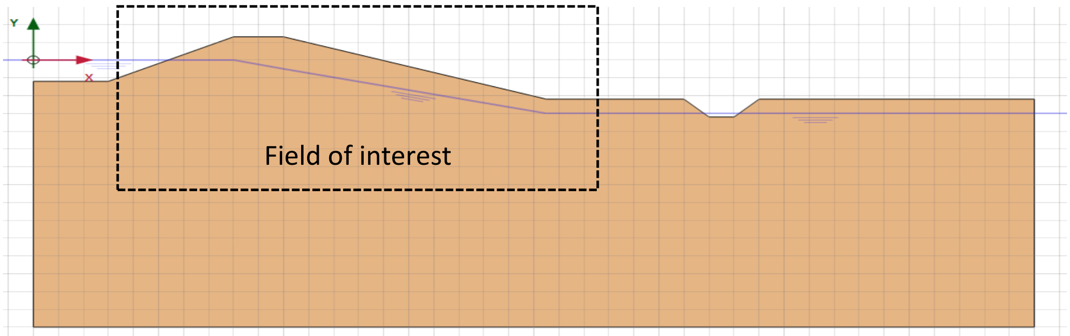


Figure 33 Cross section dike used in FEM

The reference point is the intersection of the x- and y-axis in Figure 33; it coincides with the left edge of the model in x direction and with the water level in the drainage canal in y direction.

The crest height is 1,2 meters above the water level in the drainage canal and the polder surface is at 2,2 meter below the water level in the drainage canal.[4] The water level inside the polder is kept at 3 meter below the outside water level. To keep this water level difference at a fixed value, a small drainage ditch is simulated in the model. The boundary condition in the drainage ditch is set as a fixed water level. The drainage ditch is situated at 6 meter from the inner toe. This distance is long enough to prevent extra drainage of the dike, in order to simulate a dike without extra drainage facilities.

The main goal of the study is the effect of precipitation on the pore pressures inside the dike, therefore the field of interest is the dike itself. To ensure the ground water flow in the dike is not affected by the boundaries, they are chosen at a sufficient distance from the dike.

[4] Note: *Plaxflow generates a soil body which does not change over time. This mean that the computation is only representative if the situation which is regarded, is a stable situation. For the development of the pore pressures due to precipitation the dike can be assumed to be non-changing if the cover layer of the dike does not erode. Therefore this study is only valid for-dikes with a sufficient good cover layer and situations without piping.*

The left boundary is situated at 3 meter from the outer toe, the right boundary is situated at 20 meters from the inner toe. The lower boundary is set at -15 meter below reference level. With a pre-run of the model it is verified that the outcome does not change if the 3 boundaries are moved further outwards.

6.2.2 Soil characteristics; permeability

One of the goals for the Finite Element analysis is to capture the differences between a precipitation event on a dike consisting of either peat, sand or clay. Therefore, tests are composed for three different soil types with different hydraulic characteristics.

An important concept is that the permeability of a dike is divided into two parts; the permeability of the soil below the phreatic surface is called the (saturated) permeability. The permeability of the soil above the phreatic surface is a percentage of the saturated permeability and depends on the soil type as is explained in detail in chapter 3.1

Plaxflow contains 5 sets of default parameters which describe the hydraulic characteristics of the soil: Standard, Hypres, USDA, Staring and User defined. The principle for each dataset is similar; based on a large amount of measurements, multiple soil types are defined. For each soil-type-classification the representative values for the hydraulic characteristics are chosen. In Table 8 the 5 different options for default data sets are listed.

	Type	Soil classification	Hydraulic model	Notes
Standard	Simplified selection of common soil types	Based on Hypres topsoil classification With soil texture triangle	-Van Genuchten	Organic layer available
Hypres	International soil classification system	i.e. coarse, medium, fine Can be depicted in a soil texture triangle	-Van Genuchten -Approximate van Genuchten	Organic layer available
USDA	International soil classification system	Sand, clay and loam (and variations) A different division of the soil texture triangle than with Hypres	-Van Genuchten -Approximate van Genuchten	No option for peat
Staring	Soil classification system mainly used in the Netherlands	Sand, clay, loam and peat (and variations)	-Van Genuchten -Approximate van Genuchten	‘Peat layer’ available
User defined	User defined saturated and unsaturated properties	User defined	-Van Genuchten -Spline -Saturated	

Table 8 Available hydraulic parameter data sets Plaxflow

Unsaturated characteristics

For this study the data set “Staring” is used. Staring is a soil classification data set based on Dutch soil types conducted mainly for agricultural use. Because the dataset represents Dutch soil gradations, it makes the closest fit to the materials used in Dutch dikes. The default hydraulic parameters that are used in the four described tests, can be found in Appendix: C. A visual representation of the permeability in the top layer for the three different materials is depicted in Figure 34; this figure depicts the permeability above the phreatic surface.

A known disadvantage from the Staring data set is that the hydraulic parameters for clay are different from measurements in Dutch dikes. In the report ‘assessment of geophysical material models used in agricultural research’ (Alterra, 2001) is concluded that the unsaturated permeability used in the Staring data set is most likely an underestimation of the real hydraulic conductivity in a clay dike. The underestimation is possibly caused by the difference of land use between a dike and agricultural land. On agricultural land heavy machinery is used to sow and harvest the crops. This machinery compacts the soil which causes the decrease of the hydraulic conductivity. Therefore the default hydraulic conductivity for clay in the Staring data set is an underestimation. For this part of the study we are interested in general behaviour of the pore pressures and this disadvantage of the Staring data set is negligible. However, for further research it must be kept in mind.

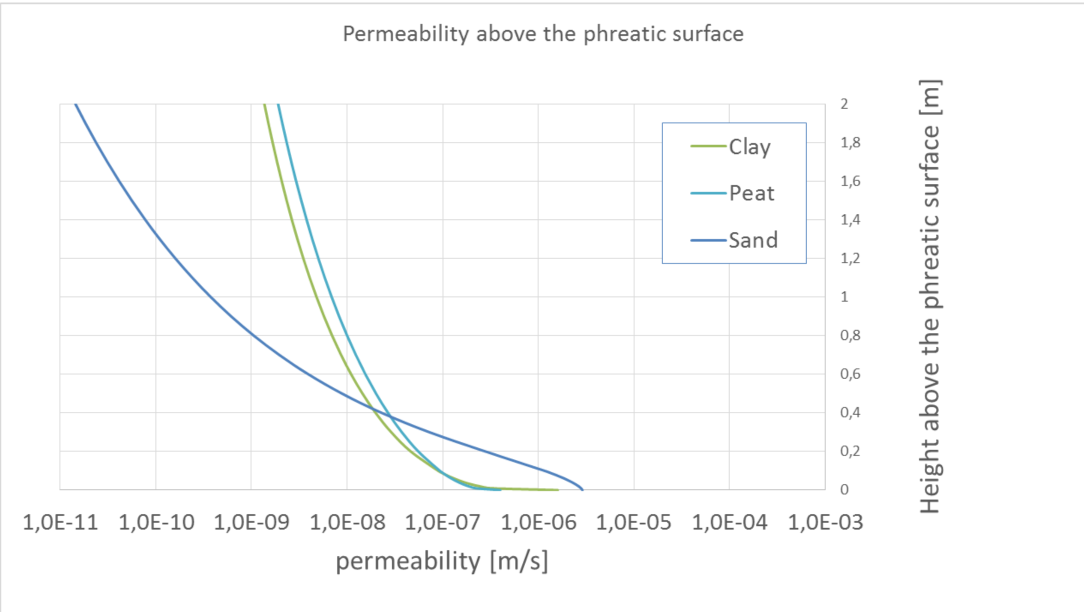


Figure 34 Permeability of the materials sand, clay and peat as used in the computation

6.2.3 Mesh

The grid of triangles that is used to perform the computation in Plaxflow is based on a 'medium' grid. Because the goal of the study is to describe the effects inside the dike, the grid in the field of interest is refined. The total grid consists of 1990 elements and 16313 nodes. The grid size varies from elements of 0,1m at the top to 1,5m at the bottom. This is shown in Figure 35.

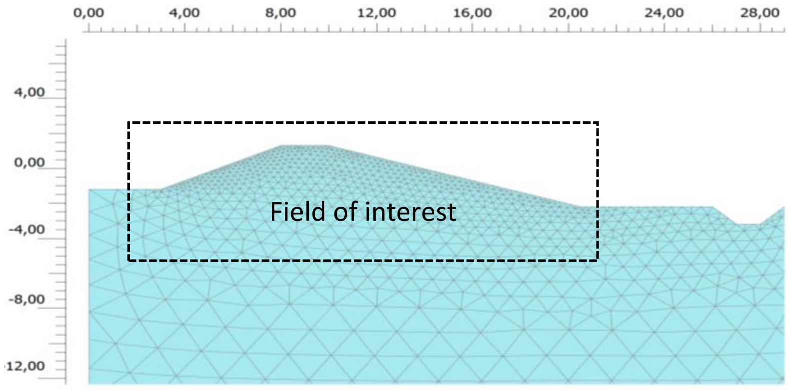


Figure 35 Computational grid

6.2.4 Hydraulic conditions

Saturation

An important aspect for the infiltration capacity of the soil is the soil moisture content or the degree of saturation of the soil. The precipitation on the dike will slowly infiltrate in the dike. The top layer of the dike consists of unsaturated soil and the permeability of the top layer depends on the moisture content; the permeability of a soil reaches its maximum when the soil is fully saturated. This is the case below the phreatic surface. Plaxflow uses the assumption that the moisture content decreases linearly with the height above the phreatic surface. This situation is used as the initial situation for each computation.

The degree of saturation, and thus the permeability, during a precipitation event will change due to the infiltrating water; Plaxflow uses iteration in each computation step to recalculate the permeability and the saturation.

Hydraulic Load

The only hydraulic load which is varied is the precipitation. The main precipitation event that is used in the 4 tests, is an event with an infinite duration and an intensity of 0,25mm/h. This value is chosen arbitrary and is in the order of magnitude of an event which occurs between once every 10 and once every 100 years. The main purpose of the simulated event is to show the differences between the three soil types. The water level in both the drainage canal and the drainage ditch are kept constant. In future studies the information can be combined with a water level rise in the drainage ditch and or the drainage canal.

In Plaxflow the precipitation intensity is defined as m/h per running meter; the precipitation intensity on a sloped surface is reduced depending on the angle of the slope. The precipitation is modelled as a constant influx on the top layer of the soil profile. If the precipitation cannot fully infiltrate due to a low infiltration capacity of the soil, the surplus of water will form a thin (0,01 m) layer of water on the slope. The surplus of the water will disappear from the model. The initial permeability depends on the unsaturated characteristics and the surface height above the phreatic surface.

6.3 Results numerical analysis

In this chapter the qualitative results of the four tests are described; per computation the main goal is described followed by the results and an evaluation of the results.

6.3.1 1a) Insight in the differences between soil types

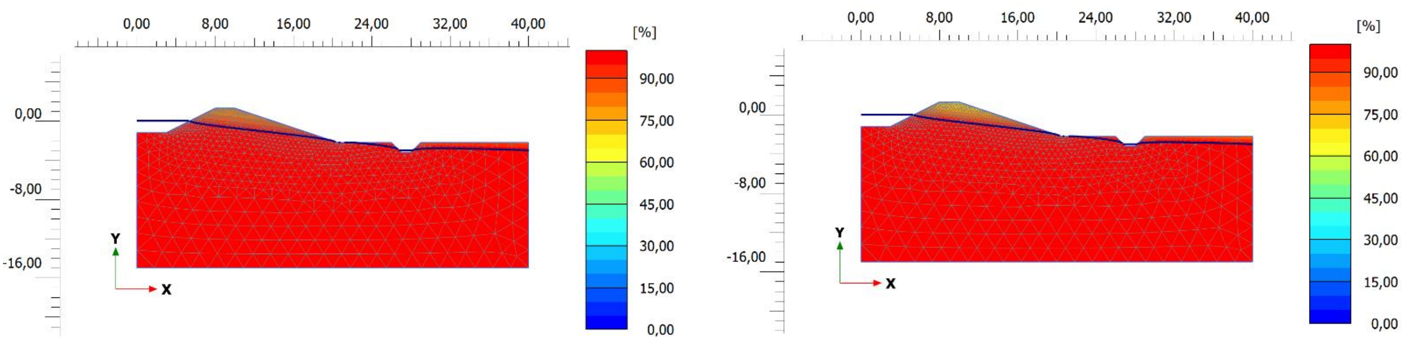
The first goal was to gain general understanding of the effect of precipitation on dikes and to study the differences between the rise of the phreatic surface in different types of soil. Therefore, three computations are performed, each with a dike consisting of a different type of soil material. In the three computations a precipitation event of 0,25 mm/h is simulated.

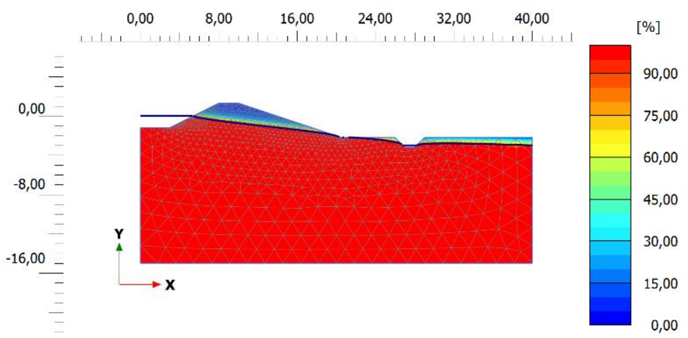
Moisture content

The first difference can be seen before the start of the precipitation event. The moisture content of the dike in the initial situation differs per soil type. The dikes consisting of clay and peat have a much higher moisture content in the top than the dike consisting of sand.

This can be explained by the ability of the different soil types to keep water under stress above the phreatic surface. In practice, this is referred to as the capillary rise. For each soil type the initial degree of saturation is shown in Figure 36. It can be seen that the effect of capillary rise on the soil moisture content is much larger in peat and clay than in sand; this is in line with common knowledge. The difference in moisture content has a large impact on the permeability of the soil, which in its turn influences the infiltration capacity. The initial permeability of the soil is plotted in Figure 37. The focus of the graph is on the top layer of the dike. It's clearly visible that in this case, peat has the highest permeability in the top layer and sand the lowest permeability.

It can be concluded that the initial moisture content combined with the unsaturated characteristics of the soil determine the initial permeability of the top layer. The soil with the lowest saturated permeability has the highest unsaturated permeability at the top of the dike.





The minimum moisture content at the crest is:

Clay: 79,4%

Peat: 68,6%

Sand: 2,4%

Figure 36 Initial moisture content clay (top left), peat (top right) and sand (bottom left)

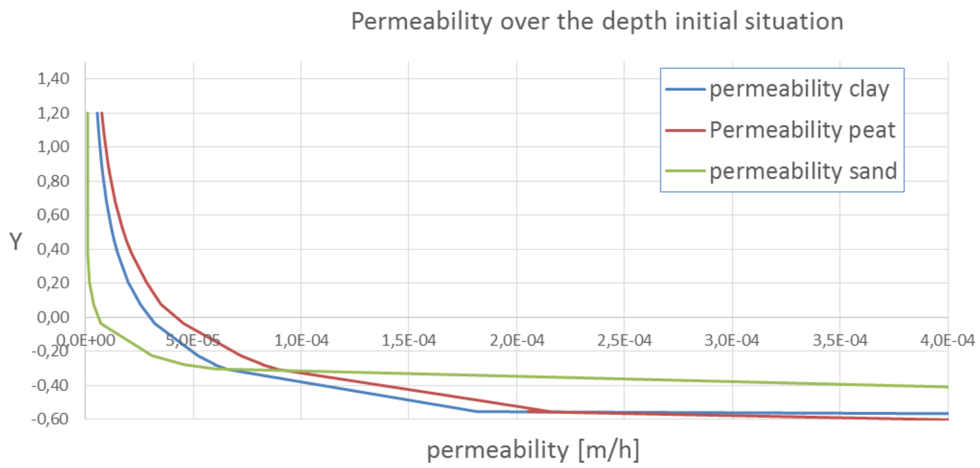


Figure 37 Unsaturated permeability in the top layer of the dike in the initial situation

Reaction of pore pressures to precipitation

To study the differences between the three selected soil types, all three types are subjected to an infinite precipitation event. To visualize the effect of the precipitation, the pore pressure development in a single point is depicted in Figure 38.The velocity of the increase of the pore pressures is the highest in peat and the lowest in sand. This is remarkable, because the (saturated) permeability of peat is the smallest ($4,0*10^{-7}$ m/s) and the (saturated) permeability of sand is the largest ($2,9*10^{-6}$ m/s). One would expect that the most permeable material will respond fastest on precipitation.

The reason for the pore pressures to rise so fast in the simulated dike of peat, is due to the unsaturated permeability characteristics; the capillary rise in the dike of peat is high and therefore the unsaturated permeability is relatively high compared to sand. For the dike of peat, the combination of the unsaturated permeability together with the initial moisture content causes a fast rise of the pore pressures. A dike of sand, on the contrary, has a very low capillary rise which causes the permeability of the top layer to be much lower and this results in a slower rise of the pore pressures.

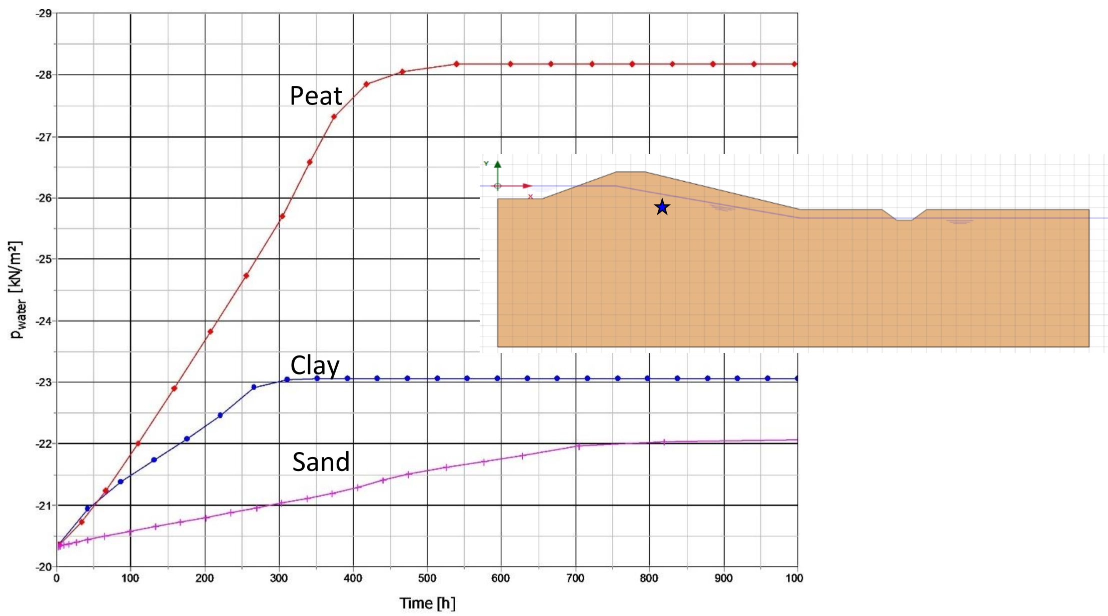


Figure 38 Development pore pressures of clay, peat and sand; precipitation intensity of 0,25mm/h

The effect of the saturated permeability can be noticed when regarding the final situation at t>900 hours. For a dike of sand, which has in general a larger saturated permeability, the final pore pressures are much lower than in peat due to the low capillary rise.

The saturated permeability seems to determine the discharge capacity of the dike and therefore it determines the maximum rise of the phreatic surface. This is in line with Darcy's law of flow through a porous medium; in order to discharge the same amount of water through a medium with a lower permeability the head difference must be larger and therefore the rise of the phreatic surface must be larger. This results in higher pore pressures in a dike consisting of clay or peat than in a dike consisting of sand.

6.3.2 1b) Insight in the differences between soil types; influence of permeability and the SWCC

In the previous test predefined representative values for the permeability are used for each soil type, in order to gain insight in the typical differences. In this test the influence of the unsaturated characteristics on the pore pressure rise is tested. Therefore the saturated permeability of each soil type is set to the same value (1,38*10^{-6} m/s) to isolate the effect of the unsaturated characteristics.

In the previous test it seemed that the unsaturated permeability was the reason for the slow rise of the pore pressures in sand compared to peat and clay. The results from this test are in line with the hypotheses; the development of the pore pressures in sand is much slower due to the low capillary rise and thus a low permeability in the top layer. The result is showed in Figure 39.

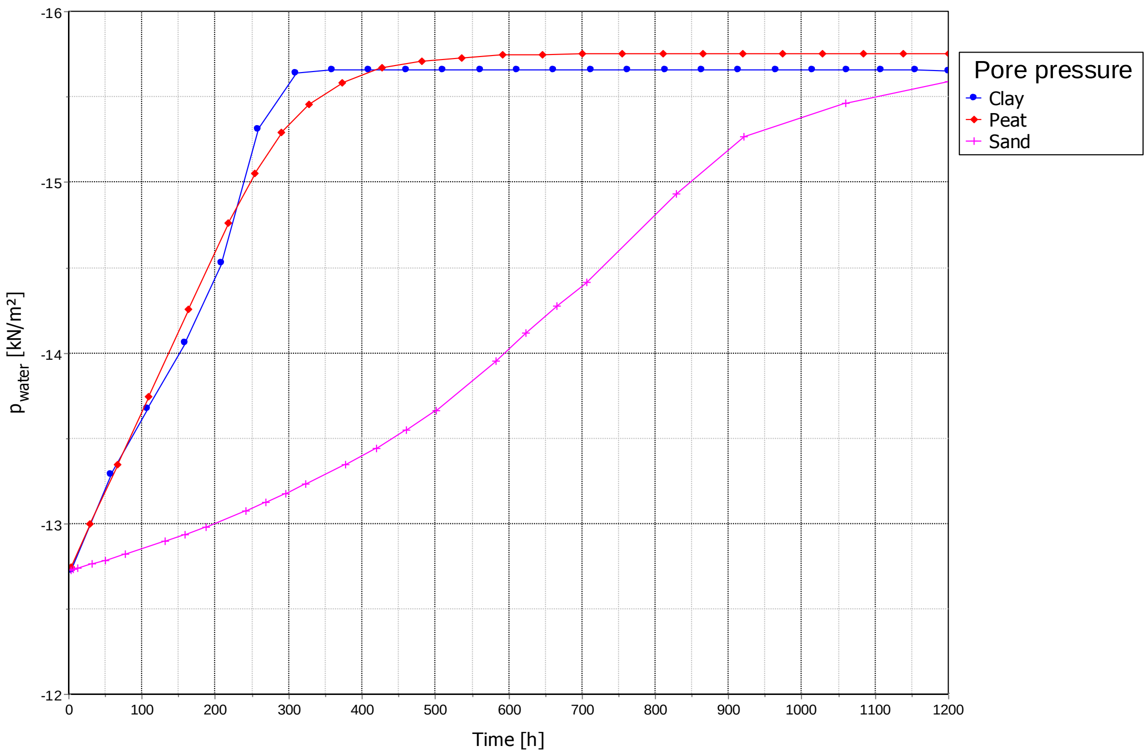


*Figure 39 Pore pressure development for clay, peat and sand with a permeability of 1,4*10^{-6} m/s for an infinite precipitation event of 0,25mm/h*

Conclusion of test 1a and 1b

From test 1a and 1b can be concluded that the unsaturated permeability determines the (initial) infiltration rate; the saturated permeability determines the height of the phreatic surface. The relation between saturated permeability, unsaturated permeability, pore pressure rise and precipitation duration can now be categorized as presented in Table 9. The effect on the pore pressure of different events is marked between '0' (no significant responds) and '++' (significant response).

	Low saturated permeability	High saturated permeability
Low unsaturated permeability	Long event low intensity: + Short event high intensity: 0	Long event low intensity: 0/+ Short event high intensity: 0
High unsaturated permeability	Long event low intensity: + Short event high intensity: ++	Long event low intensity: 0/+ Short event high intensity: +

Table 9 Response on a long and a short event for different combinations of saturated and unsaturated permeability

6.3.3 2) Development of pore pressures over the depth

The second test is performed to determine the head difference in a vertical section of the dike. The reason this study is conducted is because in the governing guidelines the pore pressures are assumed to be hydrostatic.

However in the case of a dike there is always water flowing from the drainage canal through the dike into the polder (seepage). The flow vector contains a vertical component, which indicates there should be a head difference inside each vertical section; a head difference implies a non-hydrostatic pore pressure field.

On top of the stationary seepage, the infiltrating water from precipitation will have a large vertical component which will add to the extent of non-hydrostatic pore pressures.

To visualize the results, the hydraulic head on a vertical cross-section of the dike is presented; the hydraulic head is the sum of the pressure head, elevation and velocity head. Due to the low flow velocity the velocity head is in the order of 10^{-10}m/s or smaller, therefore this term can be neglected. The two remaining terms are the pressure and the elevation head; in a situation without flow, and thus without head loss, the head over a vertical is constant.

$$h = \frac{u}{\rho_w g} + z + \left(\frac{v^2}{2g}\right) \tag{7}$$

Ground water head difference; initial situation

In the initial situation inside the dike, the groundwater head difference over the entire vertical is computed and is in the order of 0,1 to 0,15 meter, this is shown in Figure 40. The head difference is so small that it can be schematized as a hydrostatic increase of pore pressures.

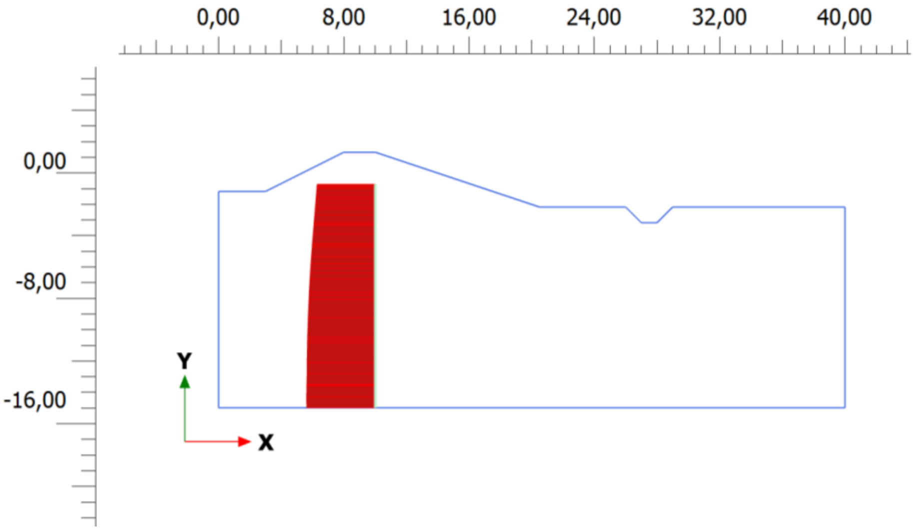


Figure 40 Ground water head over the depth in neutral situation: Maximum: -0,97m and minimum -1,06m

Ground water head difference; precipitation

When the dike is subjected to a precipitation event, the groundwater head rises due to the infiltrating water. The infiltrating water will flow through the soil, this causes friction which results in pressure loss. To know the size of the head loss is important for the extrapolation of field measurements.

The size of the head loss after the precipitation event is presented in Table 10; the ground water head over the depth in 1 vertical is visualised in Figure 41. It's clearly visible that the assumption of a hydrostatic pore pressure field is not always valid when regarding a precipitation event.

For the dike consisting of sand, the head loss is only 5 centimetre larger than in the initial situation. The head loss for clay increased with 0,1 meter and for peat the head loss increased with 0,92 meter. In this scenario with a precipitation intensity of 0,25mm/h, the assumption of a hydrostatic pressure gradient only holds for sand. For clay it is disputable and for peat a hydrostatic pore pressure field does not hold. Remarkable is that the larger the rise of the phreatic surface, the larger the percentage of head loss becomes, relative to the head rise.

	Ground water head in the crest [m]	Rise phreatic surface [m]	Ground water head at the bottom [m]	Rise ground water head at the bottom [m]	Head loss	Rise head loss w.r.t. head rise
Clay	-0,42	0,55	-0,75	0,31	0,24	43%
Peat	0,50	**1,47**	-0,57	0,49	**1,07**	72%
Sand	-0,55	0,42	-0,79	0,27	0,15	0%

Table 10 Head loss over vertical X=10 forced by precipitation event 0,25mm/h at t=∞

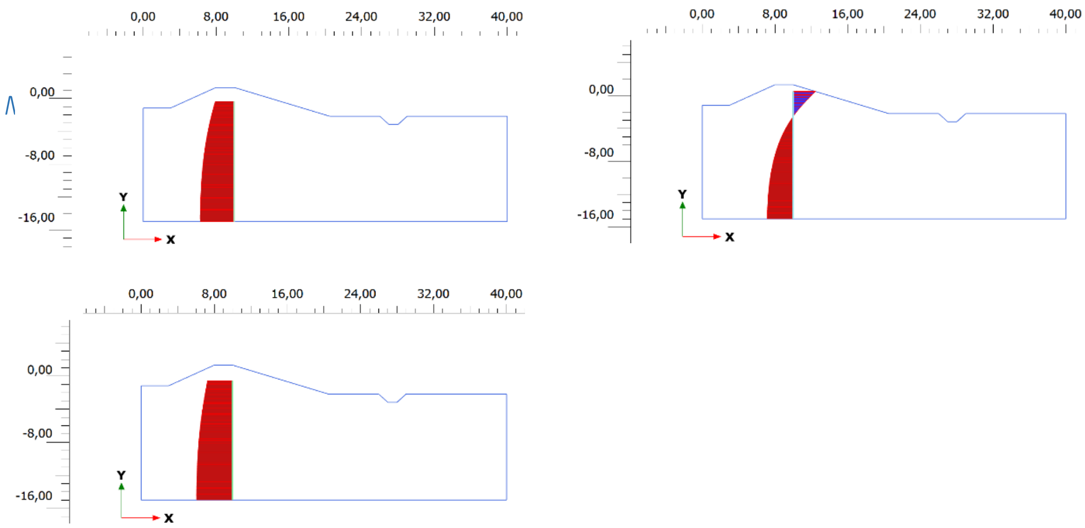


Figure 41 Equilibrium ground water head for clay (top left), peat (top right) and sand (bottom) with a precipitation intensity of 0,25 mm/h at the inner crest line (x=10)

The limit of the head loss is reached when the dike is fully saturated due to an extreme precipitation event. The dike will become fully saturated if a precipitation event is simulated with an intensity so high that the dike is not able to discharge all the water. Therefore, a precipitation event of 20 mm/h is imposed on all three types of soil material. The results of all three cases are denoted in Table 11; remarkably, the maximum head loss is the same for each soil type and thus does not depend on the permeability. The maximum head loss for this geometric profile is 1,82 meter for a 2,26 meter rise of the phreatic surface.

The explanation for the fact that the head loss only depends on the rise of the phreatic surface is as follows: the head loss is determined by the friction loss in the soil. The friction loss has a positive relation with the flow velocity and a positive relation with the permeability. Because the flow velocity and the permeability have a negative relation, the permeability does not influence the head loss. If the dike is not fully saturated, the permeability implicitly influences the head loss due to the fact that the permeability determines the rise of the phreatic surface.

	Ground water head in the crest [m]	Rise phreatic surface [m]	Ground water head at the bottom [m]	Rise ground water head at the bottom [m]	Head loss [m]	Rise head loss w.r.t. head rise
Clay/peat/sand	1,29	2,26	-0,53	0,53	1,82	80,5%

Table 11 Head loss over the vertical X=10 forced by precipitation event 20mm/h at t=∞

6.3.4 3) Development of pore pressures over the cross-section of the dike

When precipitation water infiltrates in a dike, the pore pressures over the length of the dike change. Therefore, 4 points are chosen at 1 height and different x coordinates, which are depicted in Figure 41. It shows that the pore pressures underneath the outer crest line react differently on precipitation than the pore pressures underneath the slope do.

- The measuring point at the toe is the first to respond on the precipitation event; in a later stage the pore pressure increases underneath the outer crest and develop to the centre of the dike;
- Underneath the inner crest line, the distance between the neutral phreatic surface and the surface level is maximal. Therefore it takes longer for the pore pressures to respond to the precipitation;
- The total rise of the pore pressure underneath the inner slope is smaller than beneath the crest; it is limited to the surface height of the dike;
- The shape of the increase in pore pressures underneath the crest is mostly concave and the increase underneath the toe and a part of the slope shows a convex shape at the start of the event;

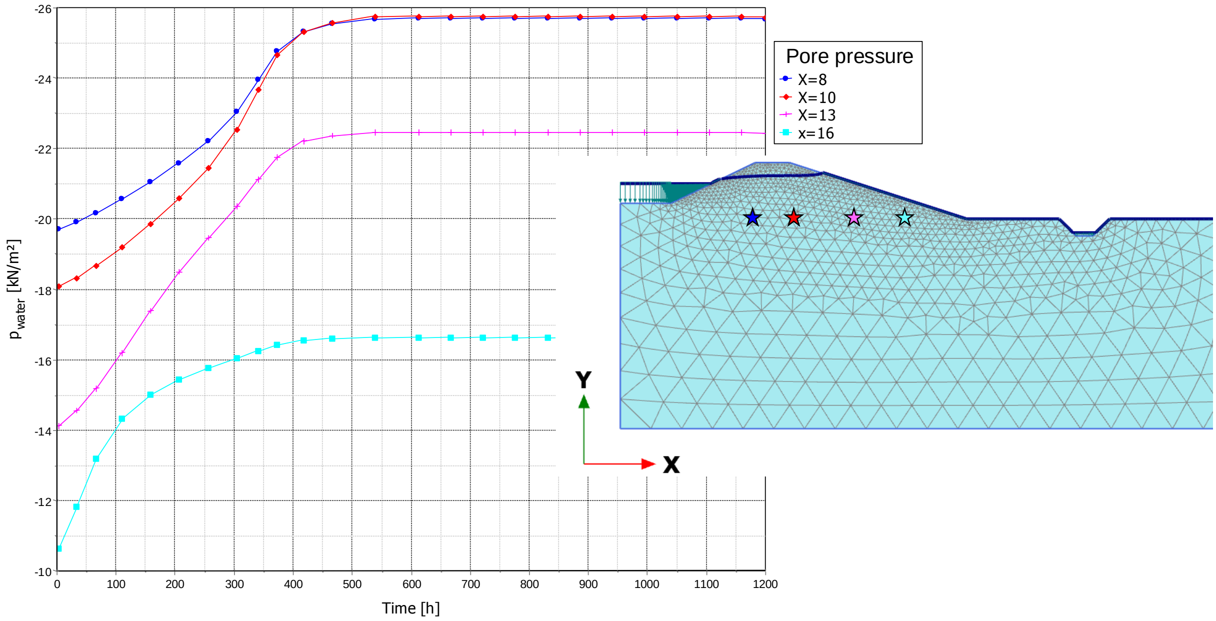


Figure 42 Pore pressure rise at underneath the crest and the slope of a dike of peat for an infinite event of 0,25mm/h (left) Phreatic surface at t=∞ for a precipitation event of 0,25mm/h on a peat dike (right)

Shape of the phreatic surface

Different stages of the shape of the surface can also be noted from the results. There are two stages which can be identified for all the events:

1 Not fully developed phreatic surface;
2 Stationary phreatic surface;

The condition of the phreatic surface at the end of an event depends on the duration of the precipitation. If a stationary situation has developed, the phreatic surface has risen to a stable height; the increased groundwater head is now high enough to discharge the inflow from the precipitation. If this is not yet the case an intermediate situation has developed with a different, not fully developed, shape. The effect of different events will be taken into account in the analysis in the next chapter.

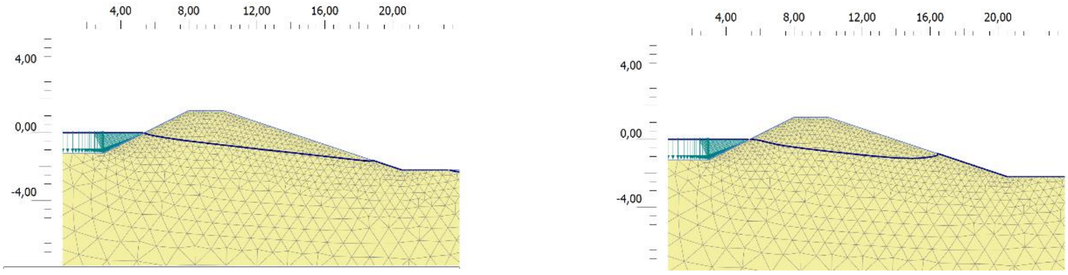


Figure 43 Illustration development phreatic surface at t=0 (left) and t=50 (right)

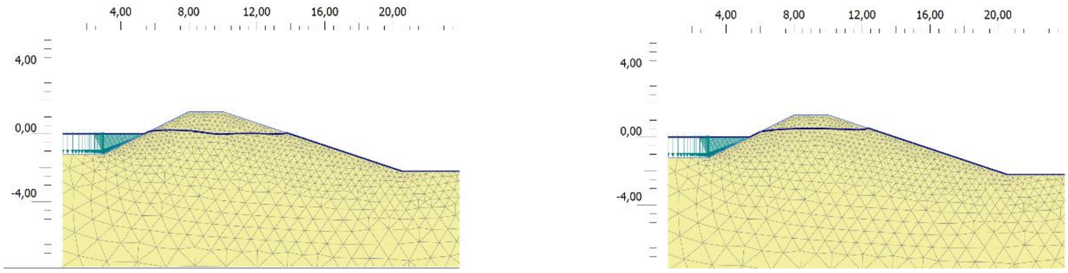


Figure 44 Illustration development phreatic surface at t=300 (left) and t>500 (right)

6.4 Comparison between the simulated pore pressures and the governing schematisation method

The results from the computations show how the pore pressures in a homogeneous dike react on precipitation. A comparison of the resulting pore pressures with the governing schematization method results in some important differences.

Schematization according to the governing method

The governing, conservative, schematization method consists of 4 points (A, B, C and D) as is illustrated in Figure 45. The effect of precipitation is only included in point 'A'. The schematization rule for the height of point A is the lowest value of 'C+L/X', 'D+L/X' or 'the crest height -0,3 m' with X is 8, 10 or 12 depending on the thickness of the cover layer 'd'. In practice this means that the schematization of the phreatic surface in a regional flood defence is 0,3 m below the crest with a hydrostatic increase of the pore pressures.

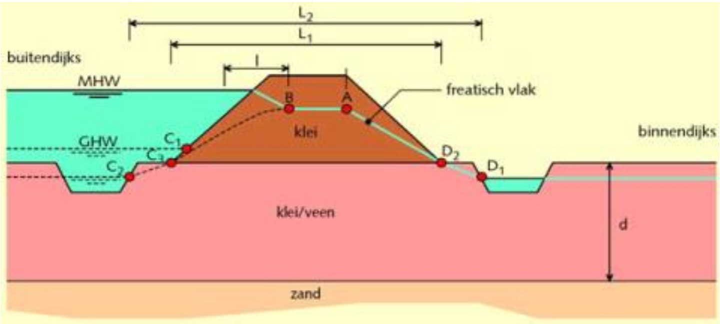


Figure 45 Illustration governing method for schematizing the phreatic surface in a flood defence (TAW, 2004)

Computed effect of precipitation

In the simulations an extreme event of 20mm/h is simulated during an infinite amount of time; such an extreme event is highly unlikely to occur and has only an illustrational purpose. The precipitation event of 20 mm/h causes the dike to become fully saturated for all three soil types; the phreatic surface is at surface level in the entire dike. The effect of precipitation on the pore pressures is concentrated in the top part of the dike, this is due to the infiltration of the water that results in a head loss due to the downward flow in the dike. The maximum effect of precipitation on the pore pressures is depicted in Figure 46. The pore pressures in the extreme scenario, when the dike is fully saturated, only depend on the dimensions of the dike and not on the permeability of the dike; therefor the pore pressures computed in this study are the maximum possible when regarding precipitation on this cross-section.

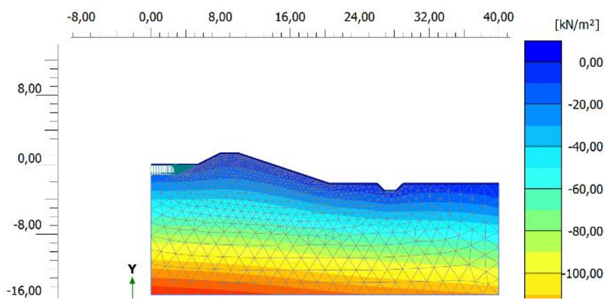


Figure 46 Pore pressures in kN/m³ in a fully saturated dike after an extreme precipitation event

Comparison computed and schematized pore pressures

An important discovery is made, when the governing schematization and the computed pore pressures are compared; in Figure 47 the maximum computed pore pressures (blue) and the schematized pore pressures (red) are depicted. The maximum computed phreatic surface is higher than the maximum schematized phreatic surface; however because the pore pressures are in reality non-hydrostatic, the governing schematization method results in an overestimation of the pore pressures in a large part of the vertical. From 1 meter below the crest and downward, the schematization from the TRWD is more conservative than physically possible due to precipitation alone. For the regarded cross-section the difference between the computed and the schematized pore pressure, due to precipitation, at a depth of 5 meters below the crest is 0,9 m water column! At larger depth the difference between the schematized and the computed pore pressure increases to 1,5 m water column.

This finding is important because only large slip circles result (immediately) in a breach; the resistance against large slip circles depends on the effect stress larger depths, in the order of several meters below the crest. With the current schematization method the pore pressures at larger depths are overestimated and thus the computed stability is too conservative.

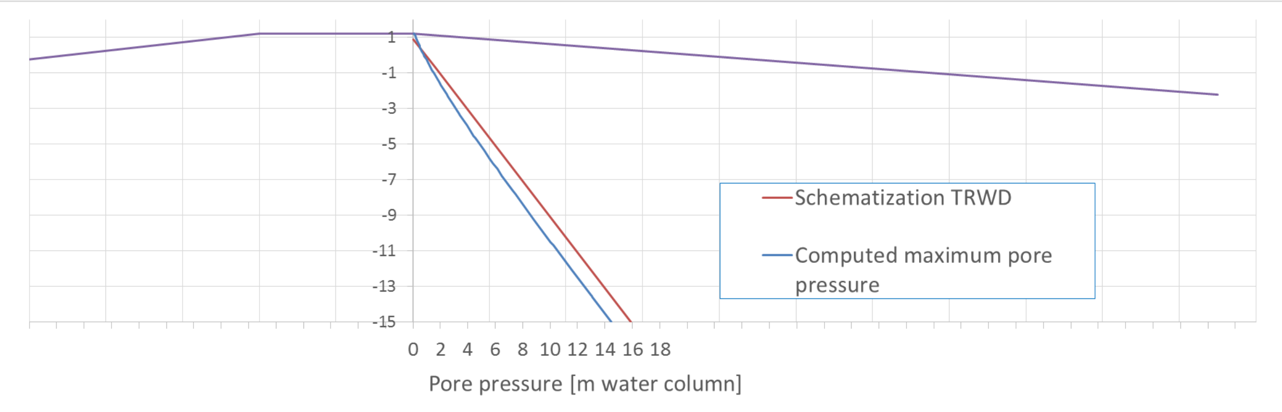


Figure 47 Comparison between the maximum schematized pore pressures according to the TRWD and the maximum computed pore pressures

6.5 Discussion

The computations are performed for a single cross-section; a sensitivity study is performed to determine the effect of the dike width and dike height (relative to the water level); the sensitivity study is included in Appendix: E. The results from the sensitivity study show that in case of a wider dike and in case of a lower water level the infiltration of the precipitation starts slower; this is because the initial phreatic surface is situated lower below the crest, therefore the degree of saturation and thus the permeability of the top layer is lower at the start of the computation. The distance between the top of the dike and the initial phreatic surface is the main factor that determines how fast the infiltration will start.

6.6 Conclusions

From the analysis of the field test and the results of the numerical test, some interesting findings on the behaviour of the phreatic surface forced by precipitation can be mentioned.

Behaviour of the phreatic surface due to precipitation

- The location that is the first to respond on a precipitation event is the toe of the dike; the increased pore pressures develop from the toe to the centre of the dike. In a more advanced stadium the phreatic surface near the drainage canal increases and this also develops towards the centre of the dike. The phreatic surface underneath the crest is the last to reach the equilibrium state;
- The soil water retention characteristics of a soil type determine how fast the infiltration reaches 'full capacity';
- The height of the phreatic surface, after an infinite precipitation event, depends only on the saturated permeability and not on the unsaturated characteristics;
- The height of the phreatic surface, after a finite precipitation event, depends on the precipitation intensity and duration in combination with the saturated and unsaturated characteristics of the soil;
- The pore pressures do not increase linearly underneath the phreatic surface during a precipitation event;

It is important for the construction of a method which includes the uncertainty of the phreatic surface due to precipitation, that the following aspects are taken into account:

Aspects to include in a probabilistic assessment of the phreatic surface

The first aspect is <u>reaction speed</u>: the speed with which the pore pressure respond to precipitation, depends on the unsaturated permeability. In simple schematization in the TRWD the penetration speed (of a high water level) is only depending on the saturated permeability. When precipitation is included in the computation such a schematization would be too simplistic.

Because the rise of the phreatic surface depends on both the duration and the intensity of the precipitation, both aspects should be considered when computing the probability density function of the phreatic surface due to precipitation.

The third aspect is the <u>non-hydrostatic pore pressure:</u> due to the infiltration of precipitation, there is flow in a downward direction. This indicates a head difference and thus a non-hydrostatic pressure field over the depth. It is important to take this into account when schematizing the pore pressures as a single phreatic surface.

Comparison between the computed pore pressures and the schematization according to the TRWD

When the findings are compared to the governing assessment method, which prescribes 1 high phreatic surface, there are 2 important aspects.

It is possible that, in the Dutch dikes, the phreatic surface can rise to surface level due to a precipitation event. The governing assessment method acknowledges the large influence of precipitation and includes a rise of the phreatic surface to a maximum of 0,3 m below the crest.

The important difference between the maximum computed pore pressures and the maximum level of the schematized phreatic surface is due to the effect of non-hydrostatic pore pressures. Due to the infiltration of precipitation the pore pressures rise mainly in the top layer of the dike. The current schematization method prescribes pore pressures due to precipitation which are higher than physically possible.

7 Quantifying the effect of precipitation on the uncertainty of the phreatic surface

The phreatic surface is influenced by multiple factors (i.e. precipitation, permeability, behaviour of the unsaturated zone, water level); all these factors influence the probability distribution of the phreatic surface. The goal of the study is to develop a framework to assess the uncertainty of the phreatic surface under influence of precipitation and the water level.

In this chapter the framework is presented to describe the effect of only the precipitation on the distribution of the phreatic surface; a framework to take into account the effect of precipitation on the water level is described in chapter 8. The following two research questions are answered:

'How can the uncertainty in precipitation events be related to the uncertainty of the phreatic surface?'

'How large is the response of the pore pressures to different combinations of hydraulic loads (multiple precipitation intensities) in different soil types (i.e. with varying (un)saturated permeability)?'

The precipitation and the height of the phreatic surface are positively related, so a longer or more intense precipitation event will result in a higher phreatic surface. In chapter 7.1 the relation between the precipitation and the phreatic surface is elaborated. With the relation between the precipitation and the rise of the phreatic surface it is possible to assess the probability distribution function of the phreatic surface which is described in chapter 7.2. In chapter 7.3 and 7.4 the resulting distributions of the phreatic surface are presented and analysed. In chapter 7.6, the answers to the research questions are provided.

7.1 Relation phreatic surface and precipitation

The rise of the phreatic surface and the precipitation intensity and duration are positively related, however the effect of the precipitation on the phreatic surface is influenced by multiple other factors. Al the variables influence the effect of the precipitation on the phreatic surface; some variables slow down the effect of precipitation whereas others determine the maximum size of the effect. In order to take these effects into account an initial situation is defined:

> *The initial phreatic surface is the phreatic surface as it would be in a homogeneous dike with a permanent and stationary water level difference without precipitation or evaporation. The initial phreatic surface is used as a reference and is depicted in* Figure 48*.*

The reference phreatic surface is the starting point for each computation; the rise of the phreatic surface is the rise above the reference level. In the next chapter the variables which influence the phreatic surface are identified. In the remainder of this chapter when 'the rise of the phreatic surface' or ' the probability distribution of the rise of the phreatic surface' is mentioned, it is always the rise of the phreatic surface <u>conditional</u> on a set of soil parameters.

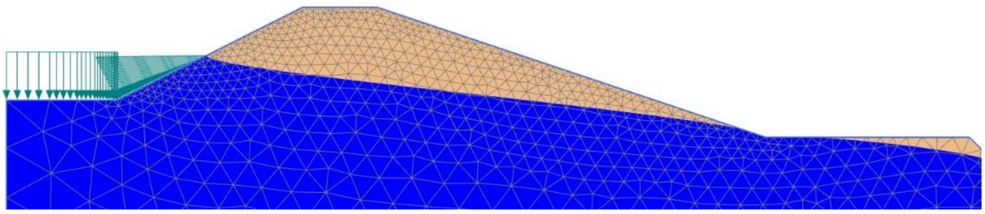


Figure 48 Illustration initial/neutral phreatic surface; starting point for each computation

7.1.1 Variables

In the research to compute the probability distribution of the phreatic surface the most important variables are included; the, for this study, less significant variables are also identified but not included in further study. The for this step important variables are briefly described; a more thorough description of the parameters that are used is necessary for and included in chapter 7.2.

Uncertainty of the permeability of the dike;

The permeability of the dike influences how much the pressure head must increase to discharge the infiltrated water. The uncertainty of the permeability is included by 4 discrete options; the process is depicted in

Uncertainty of the degree of permeability of the top layer of the dike;

This influences the (initial) infiltration capacity of the dike and will thus influence how fast the pressure head can rise. Therefor it is one of the main variables which influences the phreatic surface and is thus included.

Width of the dike;

A dike with a larger width will receive more infiltration water which must be discharged through the toe of the dike towards the polder. However a in a wider dike the distance between the crest and the initial phreatic surface is larger (illustrated in Figure 49), therefor the degree of saturation is smaller and thus less water can infiltrate in the initial situation. A sensitivity study of this variable is included in the study.

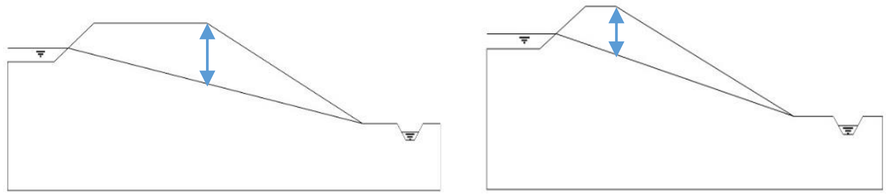


Figure 49 Illustration effect dike width on the distance between the phreatic surface and the crest

Sensitivity study on the relative crest height;

The relative crest height is the distance between the crest and the water level. When the relative crest height is larger, as is depicted in Figure 50 the distance between the initial phreatic surface and the crest is larger and thus the degree of saturation and the permeability on the top are smaller. A larger relative crest height will thus reduce the inflow of water. A sensitivity study of this variable is included in the study.

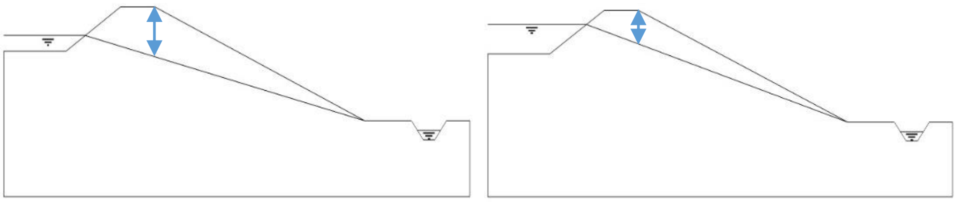


Figure 50 Illustration effect relative crest height on the distance between the phreatic surface and the crest

Heterogeneity of the soil inside the dike;

This variable is not included but is an important variable to include in the future. Heterogeneity will influence the initial phreatic surface. In the current state of the art it is too complex to include the heterogeneity in the scope of this project.

Precipitation patterns;

In this study only homogeneous precipitation events are simulated. A precipitation event can have multiple different patterns. The pattern influences the infiltration rate as follows; before the precipitation event the soil is relatively dry, therefor the initial infiltration capacity is relatively low; an event with a slow start will gradually wet the soil; when the peak of the event occurs the soil is wet and more water can infiltrate and less water will run off. The reversed pattern will cause less infiltration and thus a lower rise of the phreatic surface. The chosen approach, a homogeneous representation, provides a good starting point for the study on the effect of precipitation on the phreatic surface because it will result in the average effect.

Influence of objects (i.e. road, quay wall);

Hard structures in or on the dike influence how the water can infiltrate and how the water can flow through the dike. A sheet pile wall can block the flow entirely and a road will prevent the water from infiltrating. These features are not included in the framework due to their site specific and complex effects.

7.1.2 Statistical relation precipitation and phreatic surface

The precipitation influences the phreatic surface, therefor there is a relation between the precipitation and the height of the phreatic surface. The influence of a particular precipitation event on the phreatic surface depends on the hydraulic characteristics of the soil; therefore 'the' influence of a precipitation event on the phreatic surface is always related to the hydraulic characteristics of the soil. In the remainder of the report, when 'the influence of a precipitation on the phreatic surface' is discussed, it is always the influence of the precipitation in combination with a certain set of hydraulic soil characteristics.

Probability of occurrence of the rise of the phreatic surface related to precipitation based on continuous data

In an ideal situation the probability of occurrence of a phreatic surface should be determined with the use of the annual maxima method. The annual maxima method uses the annual maximum phreatic surface in order to derive a statistical relation between the probability of occurrence and the rise of the phreatic surface. The problem is that in reality no such data sets are available. Another possibility is to simulate continuous precipitation on a dike with the use of a large data set, preferably tens of years, to quantify the probability of occurrence of the phreatic surface in a particular dike. However this method requires significant computational efforts in order to compute the probability density functions of the phreatic surface in various dikes.

Statistical information on precipitation; IDF curves

Because computation with a continuous data set requires too much time an alternative method to relate the rise of the phreatic surface to the precipitation is developed in this study. The alternative method uses statistical information on precipitation in the Netherlands instead of a continuous data set. The statistical information on precipitation is composed by HKV-lijn-in-water and is documented in (STOWA, 2004) and is based on measurement series from the Dutch weather institute (KNMI)

The statistical information of precipitation in the Netherlands is collected in so called 'intensity duration frequency curves' (IDF curves). The Dutch weather institute (KNMI) has measured the amount of precipitation during the last 100 years; the measurements are taken with an interval of 15 minutes. The complete data series is analysed for several discrete time intervals, varying between 4 hours and 9 days. For each discrete time interval the annual maxima method is used; with this method the maximum amount of precipitation, during the discrete time interval, is used to compose statistical information on the amount of precipitation during that discrete time interval. This method is applied for 8 discrete values for the duration. The result of this method is visualised in Figure 51.

To use the statistical information from the IDF-curves it is necessary to note which information it does and does not contain. This is necessary to formulate a relation between the IDF-curves and the rise of the phreatic surface.

- Each line on the IDF graph represents possible combinations of intensity and duration for a single return period;
- All the bullets in the IDF graph which are situated in a vertical line are based on different and independent precipitation events which have occurred in different years;
- The bullets on 1 single IDF curve <u>can</u> be based on different precipitation events;
- The probability that two bullets on 1 line are based on 1 event decreases when the difference of duration is larger;
- The bullets on different IDF curves which are not situated in a vertical line <u>can</u> be based on a single event;

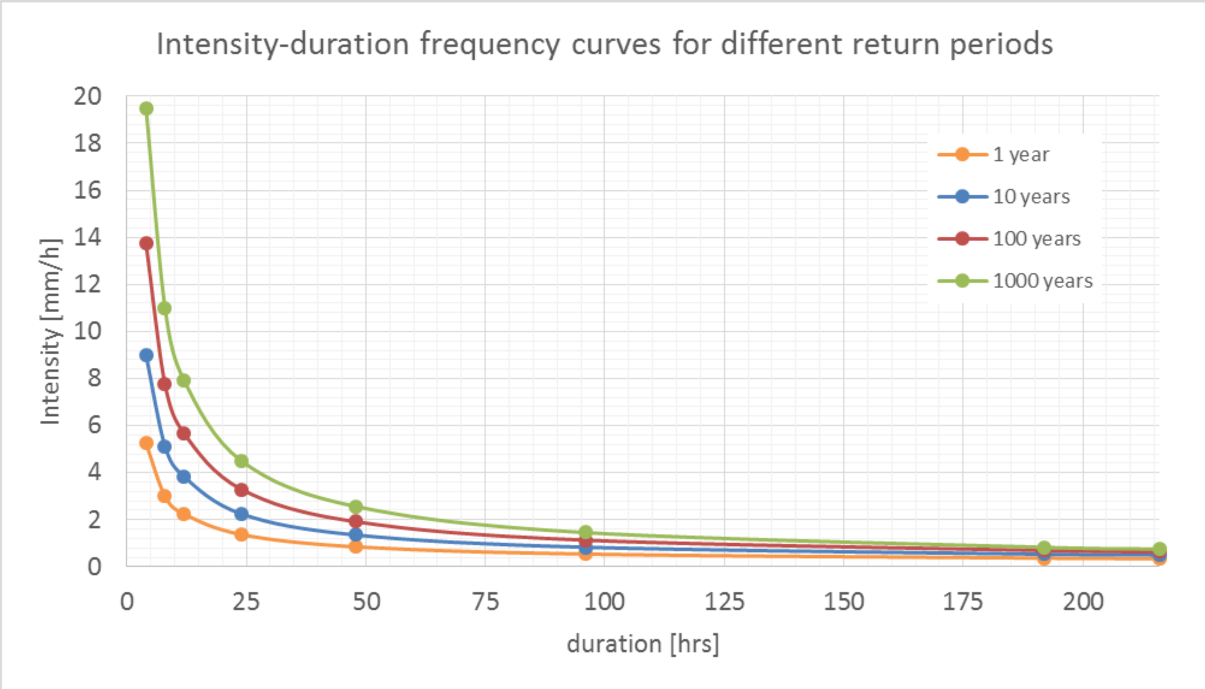


Figure 51 Intensity duration frequency curve for different return periods in de Bilt

Relation between the statistical information (IDF curves) and the rise of the phreatic surface

The effect of precipitation on the phreatic surface depends on the duration and the intensity of the precipitation event; to illustrate this an example is provided in Intermezzo 1. Because the effect on the phreatic surface depends on the duration of the precipitation it is not possible to relate the probability of occurrence of a random precipitation event to the probability of occurrence of the corresponding phreatic surface.

Based on the IDF curves it is possible to describe a relation between several specific points on the IDF curves and the rise of the phreatic surface. Therefor the following assumption is made: the precipitation events which cause the annual maximum rise of the phreatic surface have a similar duration. With this assumption it is possible to use the information from the IDF curves to compose the precipitation events which result in the annual maximum rise of the phreatic surface.

The events which result in the annual maximum rise of the phreatic surface are called the critical precipitation events. The critical precipitation events are determined by simulation of different combinations of precipitation intensity and duration in order to determine which combination has the largest effect. With the assumption that the phreatic surfaces with different return periods are caused by precipitation events with a similar duration and a different intensity, the probability of occurrence of the precipitation event and the corresponding phreatic surface is equal. In Figure 53 an illustration of the critical precipitation event is provided.

Intermezzo 1: 'Critical' precipitation intensity

An IDF curve presents information on the possible combinations of the precipitation intensity and duration with the same probability of occurrence. In Table 12 the values for a return period of 100 years are presented.

Precipitation intensity [mm/h]	4 hrs	8 hrs	12 hrs	24 hrs	2 days	4 days	8 days	9 days
1/100/year	13,8	7,8	5,7	3,3	1,9	1,1	0,7	0,6

Table 12 precipitation intensity [mm/h] for 1 exceedance frequency and different periods of duration

Each combination has the same probability of occurrence, however each combination has a different effect on the phreatic surface. To illustrate this, all the combinations in Table 12 are simulated as separate precipitation events. The effect of the separate events on the pore pressure in 1 point is depicted in Figure 52. Each curve in the graph represents the rise of the pore pressure due to 1 precipitation event.

All the simulated precipitation events have a probability of occurrence of 1/100 per year, but all the precipitation events result in different pore pressures; therefor it is not possible to relate the probability of occurrence of a random precipitation event to the probability of occurrence of the corresponding phreatic surface.

An event with a high intensity will result in a rapid rise of the pore pressures; however the most intense events are very short, therefore the pore pressures do not reach the maximum (equilibrium) level for that intensity. An event with a very long duration will reach the equilibrium situation during the course of the event; however a very long event has a low intensity which results in a low equilibrium situation.

The maximum effect on the pore pressures for this soil type and this return period is reached by an event in between the maximum and minimum duration. This event is called the critical event.

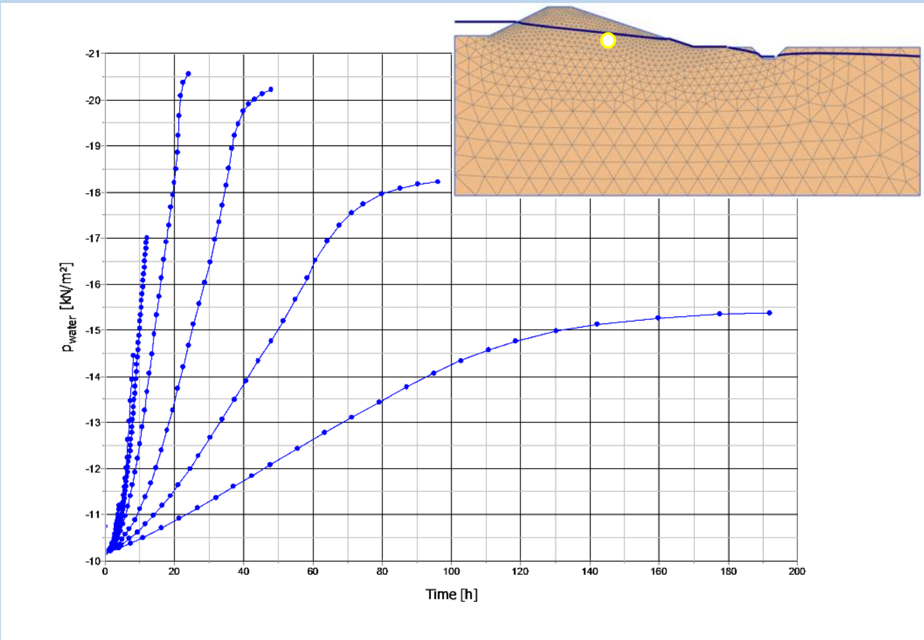
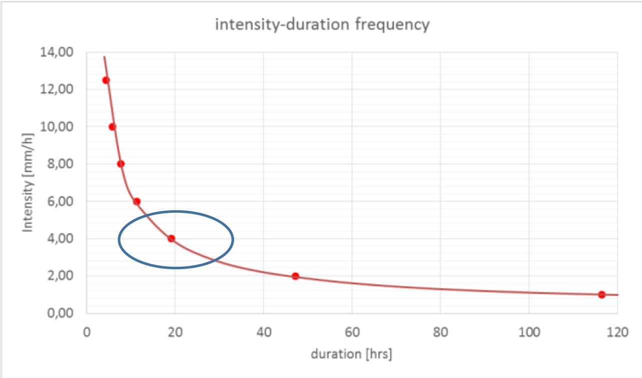


Figure 52 Pore pressure rise due to multiple events with a return period of 100 years



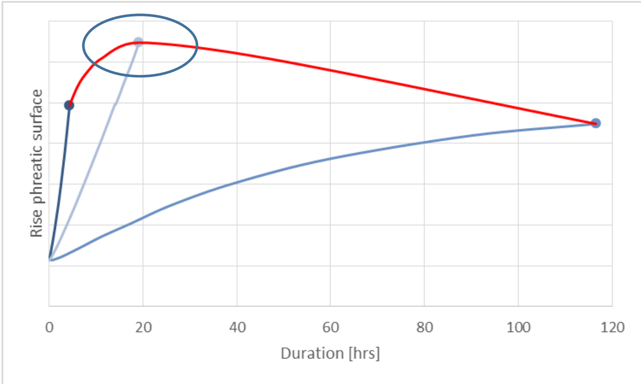


Figure 53 Different combinations of intensity and duration (left) are simulated; the corresponding rise of the phreatic surface is depicted (right). The precipitation event which causes the largest rise of the phreatic surface (blue circle) is called the critical event.

The relation between the return period of the critical event and the return period of the phreatic surface is based on the assumption that the duration of the critical event for different return periods is equal. However this assumption is only valid when regarding similar return periods. In Figure 54 the rise of the phreatic surface due to an infinite amount of separate events is depicted (with the same principle as in Figure 53) for two different return periods (100 years and 1000 years). What can be seen is that the duration of the two critical precipitation events is different; with the information on the IDF curves this means that both events could have occurred in 1 year.

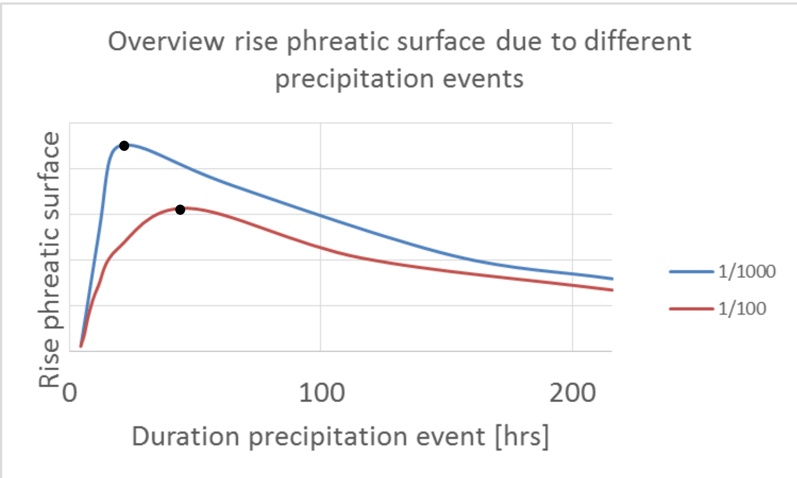


Figure 54 Rise phreatic surface (in a dike with an arbitrary set of soil parameters) due to precipitation events with two different return periods

Because the critical events for different return periods have a different duration, the following possibility occurs: during 1 year two events can occur with a different return period as depicted in Figure 55. In that case the phreatic surface due to the critical 1/100/year event is exceeded by a non-critical 1/1000/year event; this means that a critical precipitation event with a return period of 100 years results in the rise of the phreatic surface with a return period <u>smaller</u> than 100 years.

Concluding: the relation between the probability of exceedance of the critical precipitation event and the corresponding phreatic surface can be noted as:

$$P_{critical\ precipitation\ event} \leq P_{corresponding\ phreatic\ surface} \tag{8}$$

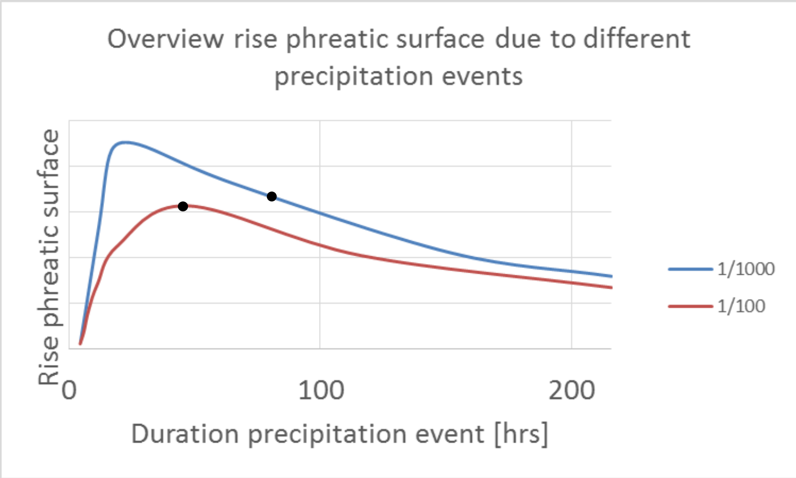


Figure 55 Rise phreatic surface (in a dike with an arbitrary set of soil parameters) due to precipitation events with two different return periods

7.2 Quantification uncertainty phreatic surface; based on precipitation statistics

For this study, the exceedance frequency of the phreatic surface, depending on the soil characteristics, is based on the exceedance frequency of a critical precipitation even. The critical precipitation event is always conditional on the set of soil parameters. This method provides the lower boundary of the phreatic surface for different return periods.

To include the uncertainty of the soil characteristics in the computation of the phreatic surface, the soil is divided in four values for the saturated permeability (i.e. $2{,}8*10^{-5}$ m/s, $2{,}8*10^{-6}$ m/s, $2{,}8*10^{-7}$ m/s, $2{,}8*10^{-8}$ m/s). The saturated permeability is assumed to be uniform for any direction of flow.

There are 3 curves selected to represent the unsaturated permeability, the three curves are based on the unsaturated behaviour of sand, clay and peat. For each combination of saturated and unsaturated permeability the critical precipitation events with a return period of 1/1 year^{-1}, 1/2 year^{-1}, 1/10 year^{-1}, 1/100 year^{-1} and 1/1000 year^{-1} are simulated. The critical precipitation event is determined by computing the rise of the phreatic surface for 10 combinations of precipitation duration and intensity. The event that causes the largest rise of the phreatic surface, in square meters, is defined as the critical event; this is the event that just reaches the equilibrium state. The corresponding rise of the phreatic surface, in square meters, is plotted as a non-exceedance curve per soil type (unsaturated characteristics) and per saturated characteristic as is illustrated in Figure 56. The probability density function (PDF) is computed by derivation of the cumulative density function (CDF) curve.

The phreatic surface is defined as: *The pressure increase at the location of the initial, neutral, phreatic surface; the neutral phreatic surface is depicted in* Figure 48*.*

The reason the pore pressure rise at the location of the neutral phreatic surface is used instead of the actual computed phreatic surface is that the pore pressures are schematized as hydrostatic. The precipitation causes however a non-hydrostatic pore pressure field, it would be a very conservative approach if the computed phreatic surface would have been used combined with hydrostatic pore pressures.

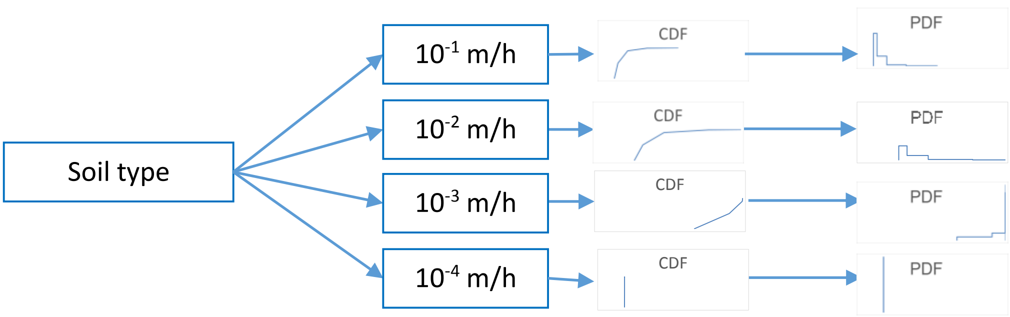


Figure 56 Illustration categorization results

7.2.1 Integration method

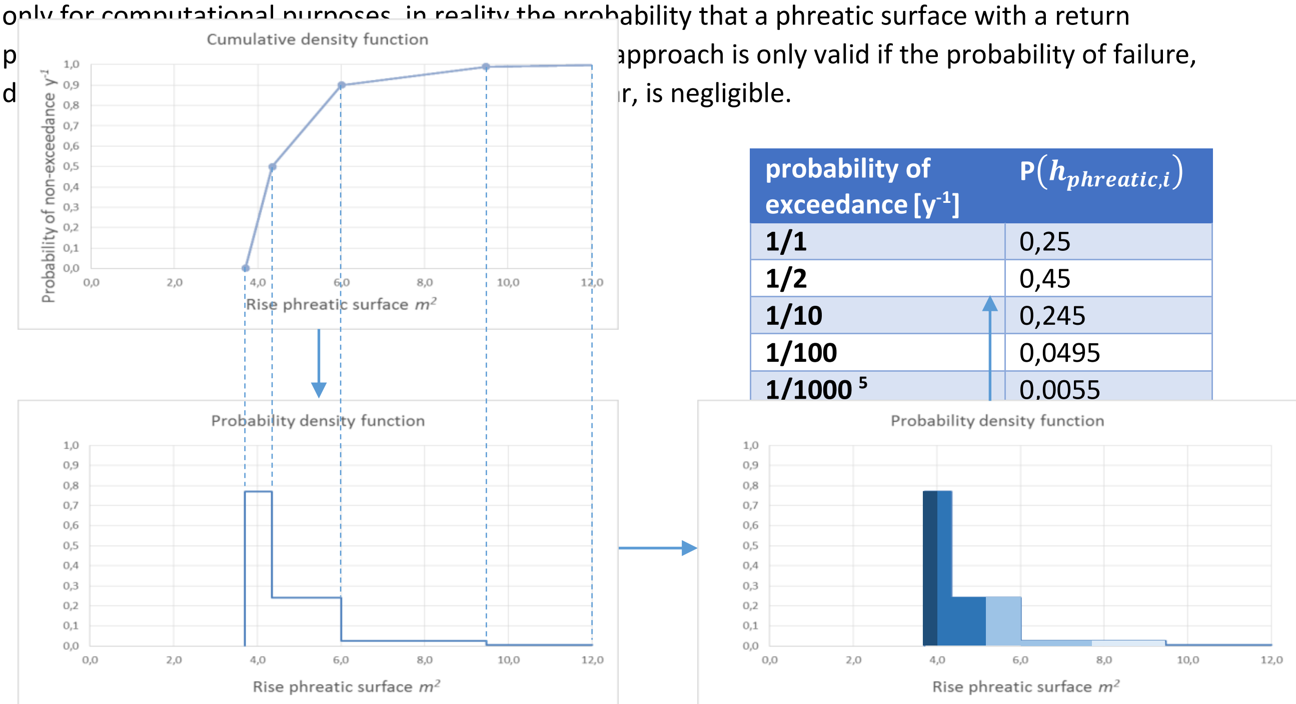
In the failure probability analysis in D-Geo Stability there is the possibility to define a maximum of 4 levels for the phreatic surface. D-Geo Stability computes the probability of failure for each defined phreatic surface. Because the computed probability of failure depends on the phreatic surface, the probability of failure is conditional $P(F|h_{\text{phreatic surface}})$. The unconditional probability of failure is:

$$P_f = \sum_{i=1}^{4} P(h_{phreatic,i})P(F|h_{phreatic,i}) \qquad (9)$$

In which $P(h_{phreatic,i})$ is the probability that the corresponding phreatic surface occurs. The value for $P(h_{phreatic,i})$ is based on the cumulative density function; for 5 points on the CDF, the probability of exceedance is computed. Through derivation of the CDF curve, the PDF curve is constructed as is depicted in Figure 57. The next step is to determine the representative probability of occurrence per computed level of the phreatic surface; this is done by integration of the PDF; the result is the probability mass corresponding to each computed level of the phreatic surface.

The four levels of the phreatic surface that are used in the computation, are the levels corresponding to the exceedance frequency 1/2 per year to 1/1000 per year. The corresponding probability of occurrence is based on integration of the PDF as is depicted in Figure 57.

The integration of the probability density function is performed as depicted in the bottom right image from Figure 57. The method of integration results in a distorted probability mass for the phreatic surface with a probability of exceedance of once a year (probability mass of 25%); this is only for computational purposes, in reality the probability that a phreatic surface with a return p… approach is only valid if the probability of failure, d… r, is negligible.



probability of exceedance [y⁻¹]	$P(h_{phreatic,i})$
1/1	0,25
1/2	0,45
1/10	0,245
1/100	0,0495
1/1000 [5]	0,0055

Figure 57 Integration of the PDF in order to compute the probability mass($P(h_{phreatic,i})$) per computed level of the phreatic surface

[5] The maximum calculated probability of non-exceedance is 0,999. The 0,001 which is unaccounted for, is added to the level with the return period of 1000 years.

7.2.2 Test set up

The computation of the rise of the phreatic surface is based on 1 profile of a regional flood defence. The dimensions of the profile are listed in Table 13 and visualised in Figure 58. The dimensions fulfil the minimum requirements of a Dutch regional flood defence.

Parameter	value
Water level drainage canal	Reference level
Level drainage canal	Reference level -1,2m
Polder level	Reference level -2,0
Crest width	2,0 meters
Gradient inner slope	1:3
Gradient outer slope	1:2
Relative crest height	0,9 meter
Water level difference	3,0 meter

Table 13 Characteristics basic schematization regional flood defence

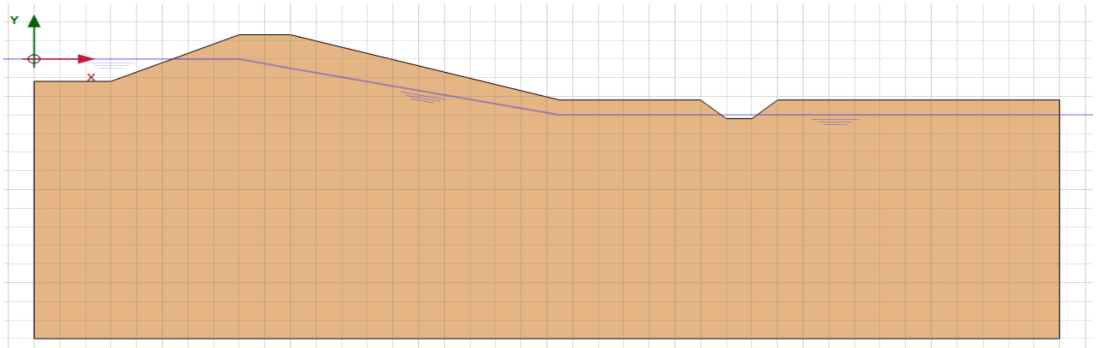


Figure 58 Schematization of a regional flood defence for the computation of the phreatic surface

Computation critical event

A critical precipitation event is the combination of intensity and duration on an IDF curve which results in the highest phreatic surface. For this study a selection of return periods varying between once every year and 1 per 1000 years is chosen. This is shown in Table 14 and Table 15.

Amount of precipitation [mm]	**Duration [hrs]**				**Duration [days]**			
Exceedance frequency [1/year]	4	8	12	24	2	4	8	9
1/1	21	24	27	33	41	52	71	75
1/2	25	29	32	39	48	60	81	86
1/10	36	41	46	54	65	80	103	109
1/100	55	62	68	79	92	109	133	138
1/1000	78	88	95	108	123	140	159	163

Table 14 Statistics total precipitation [mm] (STOWA, 2004)

Precipitation intensity [mm/h]	Duration [hrs]				Duration [days]			
Exceedance frequency 1/year	4	8	12	24	2	4	8	9
1/1	5,3	3,0	2,3	1,4	0,9	0,5	0,4	0,3
1/2	6,3	3,6	2,7	1,6	1	0,6	0,4	0,4
1/10	9,0	5,1	3,8	2,3	1,4	0,8	0,5	0,5
1/100	13,8	7,8	5,7	3,3	1,9	1,1	0,7	0,6
1/1000	19,5	11,0	7,9	4,5	2,6	1,5	0,8	0,8

Table 15 Statistics precipitation intensity [mm/h]

In order to perform the computation to find the critical intensity as efficient as possible, the precipitation events are categorized by intensity (instead of by duration). In Table 16 the simulated values are presented. The available statistics range from 4 hrs to 216 hrs (9 days), therefore the events that would last longer than 216 hours or shorter than 4 hours are not simulated.

Duration [hrs] given precipitation intensity [mm/h]										
Return period [years]	**17,5**	**15,0**	**12,5**	**10,0**	**8,0**	**6,0**	**4,0**	**2,0**	**1,0**	**0,50**
1							5,3	14,8	41,4	115,6
2							6,9	18,9	51,8	141,8
10					4,36	6,50	11,4	29,8	78,0	
100			4,3	5,8	7,7	11,3	19,1	47,2	116,5	
1000	4,5	5,5	6,9	9,0	11,9	16,88	27,8	65,0	152,0	

Table 16 Values precipitation intensity/duration for computation critical phreatic surface

For each computed event the rise of the pore pressures is measured at the initial phreatic surface; the event which causes the maximum rise is noted as the critical precipitation event conditional on the permeability, the soil type and the exceedance frequency which is regarded.

7.3 Results

The calculations are performed for each combination of the saturated and the unsaturated permeability. The results are clustered per soil type (thus clustered per unsaturated characteristic curve). Important to notice is that the dike can be considered fully saturated when the rise of the phreatic surface is 12 m^2.

7.3.1 Sand

The variation of the phreatic surface due to a precipitation on a homogeneous dike of sand is captured in Table 17, Table 18 and Figure 59. (The computation with a permeability of $2{,}8*10^{-8}$ m/s is not included because the permeability of sand is usually higher.) For less permeable sand types the response to precipitation is larger than for more permeable sand types. Less permeable soil types respond to long and less intense events (as can be seen in Table 19) and permeable types respond to short and intense events. The intensity of the events is included in Table 16.

Increase phreatic surface [m^2]	1/1	1/2	1/10	1/100	1/1000
k=$2{,}8*10^{-5}$ m/s	1,1	1,2	1,6	2,2	2,9
k=$2{,}8*10^{-6}$ m/s	2,0	2,3	2,9	3,4	4,5
k=$2{,}8*10^{-7}$ m/s	2,8	3,0	3,6	4,4	4,9

Table 17 Rise of phreatic surface [m^2] in a dike of sand per frequency of exceedance

Increase phreatic surface [m]	1/1	1/2	1/10	1/100	1/1000
k=$2{,}8*10^{-5}$ m/s	0,18	0,22	0,25	0,37	0,46
k=$2{,}8*10^{-6}$ m/s	0,29	0,30	0,36	0,54	0,63
k=$2{,}8*10^{-7}$ m/s	0,50	0,57	0,59	0,66	0,70

Table 18 Maximum rise of phreatic surface [m] in a dike of sand per frequency of exceedance

Duration of the critical event [hrs]	1/1	1/2	1/10	1/100	1/1000
k=$2{,}8*10^{-5}$ m/s	15	7	11	10	13
k=$2{,}8*10^{-6}$ m/s	216	216	78	116	65
k=$2{,}8*10^{-7}$ m/s	216	216	216	216	216

Table 19 Duration of critical event for a dike of sand per frequency of exceedance

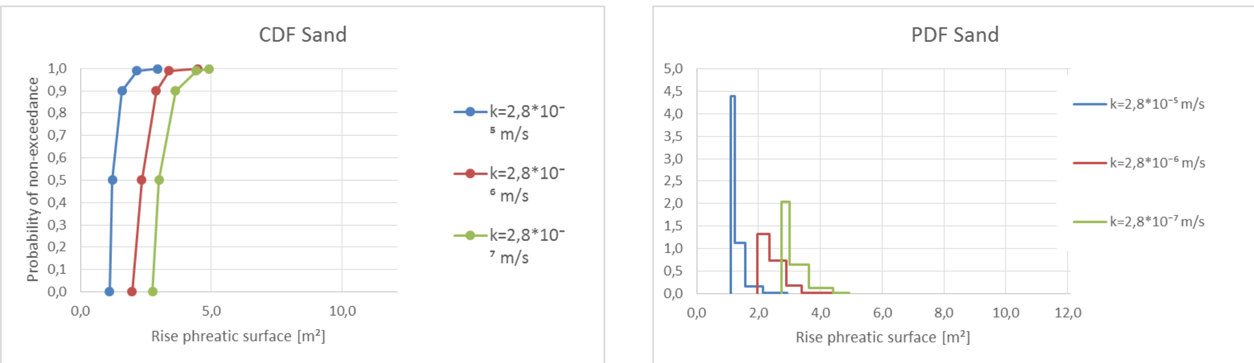


Figure 59 Cumulative density function and probability density function; sand

7.3.2 Peat

The variation of the phreatic surface due to a precipitation on a homogeneous dike of peat is captured in Table 20, Table 21, and Figure 60. Table 22 shows the duration of the events that cause the critical rise of the phreatic surface. The intensity of the events is included in Table 16. The shape of the CDF and the PDF for peat is different than from sand. The flowing things can be noted:

- In some cases the rise of the phreatic surface is 12 m^2, this means the dike is fully saturated.
- When the different degrees of permeability are compared the following can be noted: the response of the phreatic surface is the smallest for the most permeable configuration. In the next two steps ($k=2{,}8*10^{-6}$ m/s and $k=2{,}8*10^{-7}$ m/s) the response of the phreatic surface increases. For the permeability $k=2{,}8*10^{-8}$ m/s the response decreases; the permeability is to low the let the water infiltrate during the limited precipitation events.
- The duration of the critical event for $k=2{,}8*10^{-8}$ m/s is 216 and the response does not depend on the probability of occurrence of the critical precipitation event; this means the obtained results probably deviate from the reality. The accuracy of the results can be improved with the use of non-uniform precipitation events.
- The probability density function of $k=2{,}8*10^{-7}$ shows a concentration of mass near the maximum value (12m^2) in contrast with the PDF of $k=2{,}8*10^{-5}$ m/s and $k=2{,}8*10^{-6}$ m/s. The reason for this difference, is that the phreatic surface in peat with a permeability of $k=2{,}8*10^{-7}$, is very sensitive for precipitation; even with a precipitation event with a return period of 2 years the dike will be almost fully saturated.

Increase phreatic surface [m^2]	**1/1**	**1/2**	**1/10**	**1/100**	**1/1000**
$k=2{,}8*10^{-5}$ m/s	1,7	2,0	2,8	4,3	6,7
$k=2{,}8*10^{-6}$ m/s	3,7	4,4	6,0	9,5	12,0
$k=2{,}8*10^{-7}$ m/s	8,2	11,0	12,0	12,0	12,0
$k=2{,}8*10^{-8}$ m/s	2,5	2,5	2,5	2,5	2,5

Table 20 Rise of phreatic surface [m^2] in a dike of peat per frequency of exceedance

Increase phreatic surface [m]	**1/1**	**1/2**	**1/10**	**1/100**	**1/1000**
$k=2{,}8*10^{-5}$ m/s	0,25	0,27	0,38	0,53	0,71
$k=2{,}8*10^{-6}$ m/s	0,46	0,50	0,60	0,96	1,31
$k=2{,}8*10^{-7}$ m/s	0,92	1,14	1,30	1,31	1,25
$k=2{,}8*10^{-8}$ m/s	0,48	0,48	0,48	0,48	0,48

Table 21 Rise of phreatic surface [m] in a dike of peat per frequency of exceedance

Duration of the critical event	**1/1**	**1/2**	**1/10**	**1/100**	**1/1000**
$k=2{,}8*10^{-5}$ m/s	5,3	6,9	4,4	4,3	5,5
$k=2{,}8*10^{-6}$ m/s	41	52	78	47	28
$k=2{,}8*10^{-7}$ m/s	216	216	216	216	216
$k=2{,}8*10^{-8}$ m/s	216	216	216	216	216

Table 22 Duration of critical event [hrs] for a dike of peat per frequency of exceedance

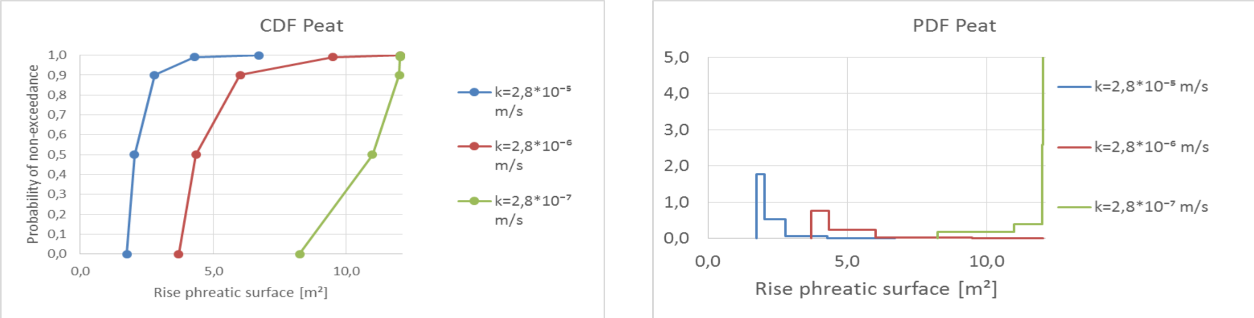


Figure 60 Cumulative density function and probability density function; peat

7.3.3 Clay

The variation of the phreatic surface due to a precipitation on a homogeneous dike of clay is captured in Table 23, Table 24 and Figure 61. Table 25 shows the duration of the events that cause the critical rise of the phreatic surface. The intensity of the events is included in Table 16.

The results for a dike of clay are similar to the results of a dike of peat. The difference between clay and peat is that the rise of the phreatic surface in a dike consisting of clay is slightly lower. The PDF of the phreatic surface in a dike of clay with a permeability of $k=2,8*10^{-7}$ has a slightly different shape. The lower part of the function behaves like expected, most of the mass is concentrated at the level of the phreatic surface with the largest probability of exceedance. There is a peak at a phreatic surface of +12 m^2; the reason is that the maximum rise is reached, therefore the mass is concentrated at the maximum value.

Increase phreatic surface [m^2]	1/1	1/2	1/10	1/100	1/1000
$k=2,8*10^{-5}$ m/s	2,0	2,6	3,7	6,5	10,4
$k=2,8*10^{-6}$ m/s	4,9	5,7	10,4	11,6	12,0
$k=2,8*10^{-7}$ m/s	11,3	11,6	12,1	12,0	12,0
$k=2,8*10^{-8}$ m/s	3,0	3,0	3,0	3,1	3,1

Table 23 Rise of phreatic surface [m²] in a dike of clay per frequency of exceedance

Increase phreatic surface [m]	1/1	1/2	1/10	1/100	1/1000
$k=2,8*10^{-5}$ m/s	0,29	0,35	0,48	0,73	1,08
$k=2,8*10^{-6}$ m/s	0,67	0,75	1,08	1,23	1,31
$k=2,8*10^{-7}$ m/s	1,20	1,23	1,31	1,31	1,31
$k=2,8*10^{-8}$ m/s	0,51	0,51	0,51	0,51	0,51

Table 24 Rise of phreatic surface [m] in a dike of clay per frequency of exceedance

Duration of the critical event	1/1	1/2	1/10	1/100	1/1000
$k=2,8*10^{-5}$ m/s	4	4	4	4	4
$k=2,8*10^{-6}$ m/s	14,8	51,8	29,8	19,1	16,8
$k=2,8*10^{-7}$ m/s	216	216	216	116	152
$k=2,8*10^{-8}$ m/s	216	216	216	216	216

Table 25Duration of critical event [hrs] for a dike of clay per frequency of exceedance

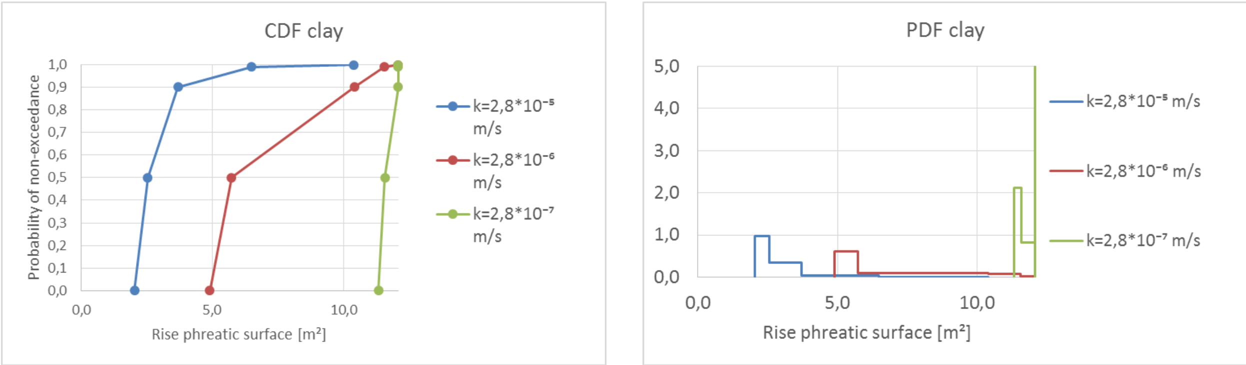


Figure 61 Cumulative density function and probability density function; clay

7.4 Analysis

In the results the following general aspects are identified:

For each soil type the rise of the phreatic surface was the largest for a $k=2,8*10^{-7}$ m/s. With a lower permeability the water is not able to infiltrate fast enough; with a larger permeability the water can infiltrate fast enough but due to the larger permeability the water is drained faster from the dike.

In both a dike of clay and a dike of peat, with a permeability of $2,8*10^{-7}$ m/s, the rise of the phreatic surface due to a yearly precipitation event is already significant; after an event with a return period of once per year the dike is almost completely saturated.

The duration of the critical event for a combination of soil characteristics depends on the exceedance frequency. Roughly stated the duration of the critical event is shorter for precipitation events with a lower probability of occurrence; however this cannot be clearly noticed from the results. The critical event is determined in a single test series without iteration; if iteration would have been used the trend would be clearly visible. For the resulting rise of the phreatic surface the iteration is not necessary; the difference in rise was in the order of 0,05 m^2 between the highest and the second highest phreatic surface.

When the results are compared to the guideline in the TRWD the following two aspects can be noted:

- The TRWD prescribes no effect of precipitation on a dike of sand; however the computations show that there is definitely a (small) increase of the pore pressures up to an increase of the hydraulic head of 0,70 meter for sand with a permeability of $2,8*10^{-7}$ m/s;
- For a dike of clay the TRWD prescribes a rise of the phreatic surface up to 0,30 meter below the crest. In some situations such a large rise due to precipitation is certainly realistic. For peat and clay with a high permeability such a large rise is not likely to occur often (once per 1000 years);

7.5 Discussion

The available information on precipitation events consisted of an amount of precipitation for precipitation events with a duration between 4 and 216 hours.

In some simulations the critical precipitation event was 4 hours. It is possible that an event of 3 hours will result in a larger rise of the phreatic surface, but because no statistical information is available on events of 3 hours this could not be verified. In this case the rise of the phreatic surface due to the event of 4 hours is used as the critical rise. The same holds for events longer than 216 hours.

For a dike consisting of peat or clay with a permeability of $2{,}8*10^{-8}$ m/s the precipitation intensity does not influence the rise of the phreatic surface. It is questionable if the results are reliable, because in reality there might be dry periods during the critical event, therefore the rise of the phreatic surface might be lower than computed.

The rise of the phreatic surface is based on the rise of the pore pressures at the initial phreatic surface with a hydrostatic gradient over the depth; in future computations it might be useful to describe the head loss due to the infiltration more accurately in order to schematize a non-hydrostatic gradient.

7.6 Conclusions

For each combination of soil type and permeability, the phreatic surface with a probability of exceedance of 1/1, 1/2, 1/10, 1/100 and 1/1000 per year is computed.

For a dike with a permeability between $2{,}8*10^{-5}$ m/s and $2{,}8*10^{-6}$ m/s to $2{,}8*10^{-7}$ m/s, the method which relates the uncertainty of the phreatic surface to the uncertainty of the precipitation seems to work. However, for soil types that are sensitive to long precipitation events, the method which is used is less reliable. The available statistical information on precipitation events longer than 216 hours do not contain enough information to accurately compute the resulting phreatic surface; the shape of the event is of importance for relatively impermeable dikes.

In several cases the phreatic surface rises to the surface level; the dike is now fully saturated. Due to the downward flow in the dike, the pore pressures are non-hydrostatic. Therefore a schematization of a fully saturated dike would be a large overestimation of the pore pressures. To prevent a conservative approach, the definition of the computed phreatic surface is: the phreatic surface is the linear extrapolation of the rise of the pore pressures at the initial phreatic surface.

For each frequency of exceedance, 10 different combinations of intensity and duration are simulated based on the statistical information from the IDF curves. Only one of the combinations results in the largest rise of the phreatic surface; the combination of intensity and duration that resulted in the largest rise of the phreatic surface is called the critical event. For each probability of exceedance (1/1, 1/2, 1/10, 1/100 and 1/1000 per year) the critical events are computed for all different combinations of soil type (clay, peat and sand) and permeability. The information on the intensity-duration ratio of all the critical events is included in this chapter; in general can be said that the duration of the critical event is shorter for events with a lower probability of occurrence; the duration of the critical event is also shorter for dikes with a larger permeability.

The probability of exceedance of the critical event is assumed to be equal to the probability of exceedance of the corresponding phreatic surface. This assumption is necessary because the amount of individual events, on which the IDF curves are based is not known. The result of the assumption is that the computed rise of the phreatic surface is a lower boundary.

The simulation results show large differences of the effect of precipitation on the phreatic surface between dikes consisting of different soil types. The probability distribution is composed for all three materials and is conditional on the permeability of the dike. The saturated permeability in combination with the unsaturated characteristics determine the variation of the phreatic surface; therefore it is important to acquire information on both parameters in order to schematize the distribution of the phreatic surface accurately.

8 Schematization uncertainty phreatic surface in regional flood defence

In this chapter the answer on the research question is answered:

'How to assess the uncertainty of the phreatic surface in a regional flood defence influenced by precipitation and the water level?'

The phreatic surface is the schematization of the pore pressures inside a dike. The pore pressures inside a dike are influence by the hydraulic conditions such as the recharge by precipitation and change in water level. The size of the influence of the boundary conditions is influenced by the soil hydraulic characteristics and the geometry of the dike.

The type of answer on the research question is a framework to include the phreatic surface in a failure probability analysis. The framework provides tools and insights in order to schematize the uncertainty of the phreatic surface. The framework aims to give a scientific basis for the schematization of the uncertainty of the phreatic surface.

Due to the scope of the project, the framework is based on several assumptions, the assumptions are clearly identified in the framework in order to prevent misunderstanding of the method or the conclusions.

The method to schematize the uncertainty of the phreatic surface consists of 7 steps and is elaborated as a whole in chapter 8.1. In chapter 8.2 the separate steps are extensively elaborated. In respectively chapter 8.3 and 0 the method is discussed and the conclusions are presented.

8.1 Schematization distribution phreatic surface; general overview

In the probabilistic method the limit state function is used in order to determine the probability of failure. The limit state function takes into account the mean value and distribution of each contribution variable. In this study a method to determine the distribution of the phreatic surface is composed.

The method is based on a predefined set of conditional distributions of the phreatic surface. The set of distributions is conditional on the saturated and unsaturated permeability of a homogeneous dike.

The method to describe the distribution of the phreatic surface consists of the following 7 steps:

1 Establish the cross-section of the dike;
2 Establish the initial (average) phreatic surface inside the dike;
3 Determine the hydraulic characteristics of the soil;

4 Establish the distribution of the phreatic surface according to the previous established information;
5 Scale the distribution of the phreatic surface for the differences in geometry between the standard case and the case at hand;
6 Determine the correlation between the water level in the drainage canal and the rise of the phreatic surface due to precipitation;

7 Computation unconditional failure probability;

8.2 Schematization distribution phreatic surface; detailed overview

8.2.1 Schematization cross-section of the dike

The first step in the modelling of the dike is the schematization of the geometry of the dike. The definition of a dike section has the criteria ‘as less as possible differences of geometry and geology inside a section’. To keep the amount of sections, and thus the amount of computations, as low as possible there will be some variation inside each section

If macro stability occurs inside a section it will happen at the weakest spot of the section. Therefor the cross-section which should be schematized is the part where macro instability is most likely to occur. This can be a for example a very small cross-section, a cross-section with unfavourable soil conditions. This step is based on expert judgement of the area an on macro stability.

8.2.2 Initial (average) phreatic surface

The second step is to determine the initial phreatic surface inside the dike. The initial phreatic surface is used as basis for the further computation; in the next steps the rise of the phreatic surface on top of the initial phreatic surface is computed.

The simplest representation of the initial phreatic surface is a linear connection between the toe and the water level in the drainage canal as depicted in Figure 62. The schematization of the phreatic surface is based on a homogeneous dike.

A more detailed insight in the composition of the dike can be gained with the use of multiple head monitoring wells inside 1 cross-section. Already with 2 head monitoring wells differences in the initial phreatic surface can be detected. It is necessary to measure in a relatively normal situation; the goal of the measurement is to detect deviation from the straight phreatic surface. How to include the measured groundwater level has to be studied in future research.

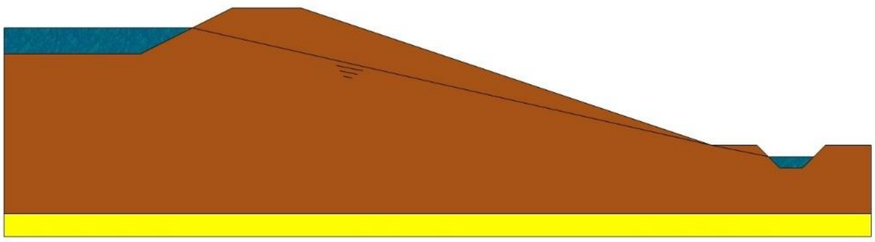


Figure 62 initial (average) phreatic surface

8.2.3 Hydraulic soil characteristics

The third step in the method is to determine the hydraulic characteristics of the soil; these characteristics determine how fast the precipitation will infiltrate in the dike. The hydraulic soil characteristics determine the type of precipitation event on which the phreatic surface will start to rise.

The hydraulic soil characteristics are composed of 2 components:

- Saturated permeability;
- Unsaturated characteristics;

Saturated permeability

The saturated permeability determines how fast the infiltration water can be discharged through the dike. Therefore the saturated permeability determines the maximum rise of the phreatic surface given a precipitation event with a certain intensity.

The saturated permeability is schematized as a deterministic value. The framework is based on one out of four possible values for the saturated permeability: $2,8*10^{-5}$ m/s, $2,8*10^{-6}$ m/s, $2,8*10^{-7}$ m/s, $2,8*10^{-8}$ m/s. Through expert judgement a probability distribution of the four possible values for the permeability should be composed. It is important to determine the saturated permeability of the core of the dike or near the toe of the dike; this is the part through which the water will drain from the dike.

Unsaturated characteristics

The unsaturated characteristics of the soil determine the speed with which the precipitation can infiltrate in the dike material. The unsaturated permeability varies over the depth due to the different degrees of saturation. The unsaturated permeability depends on the saturated permeability and the unsaturated characteristics of the soil.

There are three types of unsaturated characteristics included in the method; the types are based on three different soil types. Note must be taken that the unsaturated characteristics only provide information on the shape of the unsaturated permeability and not on the height of the unsaturated permeability. The total unsaturated permeability depends on the unsaturated characteristics, the saturated permeability and the degree of saturation. The three different curves of unsaturated behaviour are listed in Table 26.

The choice of the unsaturated characteristics must be based on the unsaturated characteristics of the top layer of the dike. The three soil types which are available to use are chosen because those are the three main components of Dutch regional flood defences.

‘material’	Characteristics
Peat	relatively permeable in top layer due to a combination of medium capillary rise and higher permeability in unsaturated conditions than clay
Sand	relatively impermeable in the top layer due to low capillary rise
Clay	medium permeability at top layer; although the capillary rise is higher than in peat the unsaturated characteristics result in a lower permeability at the top than with peat

Table 26 Unsaturated characteristics

8.2.4 Distribution phreatic surface

The distribution of the phreatic surface is divided into two parts, a distribution for a high phreatic surface and a distribution for a low phreatic surface (the latter is not included in this study) as is depicted in Figure 63. The separate probability density for both the low and the high phreatic surface is 100%; the sum of both is thus 200%. The reason for this approach is that the failure probability has the unit 'per year' there for the probability of the phreatic surface must have the same unit; during 1 year the phreatic surface will have a maximum and a minimum, therefor the failure probability can be computed for both a maximum and a minimum phreatic surface. The lowest value for the high phreatic and the highest value for the low phreatic surface are the phreatic surfaces which are exceeded once a year; the yearly probability of occurrence is thus 100%. The average phreatic surface is thus not included in the computation because the average value is exceeded (both positive and negative) more than once a year.

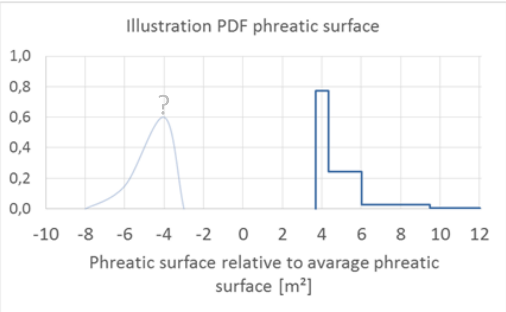


Figure 63 Illustration probability density distribution of the phreatic surface

The distribution of the rise of the phreatic surface can be determined with the (un)saturated characteristics and the geometry of the dike. For each combination of characteristics the rise of the phreatic surface is computed and listed in Table 27, Table 28 and Table 29. The distribution of the rise of the phreatic surface is clustered on the unsaturated characteristics of the soil. The rise of the phreatic surface is provided in m^2 over the total width of the dike. The shape of the phreatic surface for each combination of saturated and unsaturated permeability is provided in Appendix: D. The distribution of the height of the phreatic surface is computed for a dike with a homogeneous saturated permeability forced by a homogeneous precipitation event.

Distribution Increase phreatic surface [m^2]	1/1	1/2	1/10	1/100	1/1000
k=2,8*10^{-5} m/s	1,1	1,2	1,6	2,2	2,9
k=2,8*10^{-6} m/s	2,0	2,3	2,9	3,4	4,5
k=2,8*10^{-7} m/s	2,8	3,0	3,6	4,4	4,9

Table 27 Rise of phreatic surface [m^2] in a dike of sand per frequency of exceedance

Increase phreatic surface [m^2]	1/1	1/2	1/10	1/100	1/1000
$k=2,8*10^{-5}$ m/s	1,7	2,0	2,8	4,3	6,7
$k=2,8*10^{-6}$ m/s	3,7	4,4	6,0	9,5	12,0 [7]
$k=2,8*10^{-7}$ m/s	8,2	11,0	12,0 [7]	12,0 [7]	12,0 [7]
$k=2,8*10^{-7}$ m/s [6]	*2,5*	*2,5*	*2,5*	*2,5*	*2,5*

Table 28 Rise of phreatic surface [m^2] in a dike of peat per frequency of exceedance

Increase phreatic surface [m^2]	1/1	1/2	1/10	1/100	1/1000
$k=2,8*10^{-5}$ m/s	2,0	2,6	3,7	6,5	10,4
$k=2,8*10^{-6}$ m/s	4,9	5,7	10,4	11,6	12,0 [7]
$k=2,8*10^{-7}$ m/s	11,3	11,6	12,0 [7]	12,0 [7]	12,0 [7]
$k=2,8*10^{-7}$ m/s [6]	*3,0*	*3,0*	*3,0*	*3,1*	*3,1*

Table 29 Rise of phreatic surface [m^2] in a dike of clay per frequency of exceedance

8.2.5 Scaling of distribution phreatic surface for geometry of the dike

The next step is to adjust the rise of the phreatic surface, in order to take the differences between the geometry of the dike which is assessed and the geometry used for the computation of the distribution. The distribution of the phreatic surface which is computed for the basic geometry is called the basic distribution.

There are two aspects of the geometry which influence the effect of precipitation on the phreatic surface: the width of the dike and the difference between the water level and the crest; the latter is called the relative crest height. Both aspects influence the initial saturation at the top layer of the dike and thereby influence the initial infiltration capacity of the dike.

To scale the rise of the phreatic surface a basic set scaling tables are composed; the scaling tables are based on the sensitivity study. The sensitivity study is performed with the use of precipitation events with a return period of 100 years. The width and the height of the dike is varied in order to quantify the effect of the two variables. To acquire more detailed results, the combined effect of crest width, relative crest height and precipitation on the precipitation should be studied further.

Scaling for crest width

The distribution of the rise of the phreatic surface depends on the width of the dike; for a wider dike the relative rise of the phreatic surface is smaller. In Table 30, Table 31 and Table 32 the relative rise of the phreatic surface is presented for 3 different soil types.

In general the relative rise of the phreatic surface is lower for a wider dike; however when the dike becomes (near) saturated in the basic situation the same will happen in a wider dike; because a wider dike has a longer phreatic surface the relative rise is larger instead of smaller.

[6] The results of the rise of the phreatic surface for permeability $k=10^{-4}$ m/h are indicative, further research must provide more accurate information.
[7] Note must be taken that 12,0 is the maximum rise of the phreatic surface possible; the schematization of the phreatic surface should not be on surface level; in that case the pore pressures would be largely overestimated.

Sand	$k=2,8*10^{-5}$ m/s	$k=2,8*10^{-6}$ m/s	$k=2,8*10^{-7}$ m/s
Basic (crest width 2 m)	1	1	1
Crest width 4 m	0,98	1,00	0,99
Crest width 6 m	0,94	1,02	0,98

Table 30 Relative rise phreatic surface [-], scaling for crest width, sand

Peat	$k=2,8*10^{-5}$ m/s	$k=2,8*10^{-6}$ m/s	$k=2,8*10^{-7}$ m/s	$k=2,8*10^{-8}$ m/s
Basic (crest width 2 m)	1	1	1	1
Crest width 4 m	1,04	1,08	1,33	0,98
Crest width 6 m	1,05	1,15	1,66	0,96

Table 31 Relative rise phreatic surface [-], scaling for crest width, peat

Clay	$k=2,8*10^{-5}$ m/s	$k=2,8*10^{-6}$ m/s	$k=2,8*10^{-7}$ m/s	$k=2,8*10^{-8}$ m/s
Basic (crest width 2 m)	1	1	1	1
Crest width 4 m	0,97	1,24	1,34	1
Crest width 6 m	0,99	1,59	1,70	1

Table 32 Relative rise phreatic surface [-], scaling for crest width, clay

Scaling for crest height

The distribution of the rise of the phreatic surface depends on the relative crest height of the dike; for a larger relative crest height the relative rise of the phreatic surface is smaller. In the relative rise of the phreatic surface is presented for 3 different soil types.

In general the relative rise of the phreatic surface is larger for a dike with a smaller relative crest height; however when the dike becomes (near) saturated in the basic situation the same will happen when the relative crest height is smaller. With a smaller relative crest height the initial phreatic surface is closer to the surface and thus the maximum rise of the phreatic surface is less.

Sand	$k=2,8*10^{-5}$ m/s	$k=2,8*10^{-6}$ m/s	$k=2,8*10^{-7}$ m/s
Rel. crest height 0,9 m	1,10	1,07	1,03
Rel. crest height 1,2m (basic)	1	1	1
Rel. crest height 1,5 m	0,93	0,93	0,97

Table 33 Relative rise phreatic surface [-], scaling for relative crest height, sand

Peat	$k=2,8*10^{-5}$ m/s	$k=2,8*10^{-6}$ m/s	$k=2,8*10^{-7}$ m/s	$k=2,8*10^{-8}$ m/s
Rel. crest height 0,9 m	1,12	1,03	0,82	1,06
Rel. crest height 1,2m (basic)	1	1	1	1
Rel. crest height 1,5 m	0,99	0,90	1,18	0,96

Table 34 Relative rise phreatic surface [-], scaling for relative crest height, peat

Clay	$k=2,8*10^{-5}$ m/s	$k=2,8*10^{-6}$ m/s	$k=2,8*10^{-7}$ m/s	$k=2,8*10^{-8}$ m/s
Rel. crest height 0,9 m	1,15	0,76	0,82	1,08
Rel. crest height 1,2m (basic)	1	1	1	1
Rel. crest height 1,5 m	0,91	1,06	1,18	0,95

Table 35 Relative rise phreatic surface [-], scaling for relative crest height, peat

8.2.6 Correlation water level and phreatic surface

The sixth step in the computation of the distribution of the phreatic surface is to define the combined probability of occurrence of a water level in the drainage canal and a rise of the phreatic surface due to precipitation. The precipitation can influence the water level in the canal and the water level in the canal can influence the infiltration capacity in the soil and thereby the influence of precipitation on the phreatic surface.

The starting point is to determine the unconditional probability of occurrence of different water levels $P(h_i)$ and the unconditional probability of occurrence of the phreatic surface due to precipitation $P(Ph_i)$. The first one is out of the scope of this study, a possible distribution of the water level is presented in Figure 64. The distribution of the phreatic surface due to precipitation is computed in the previous steps and can look like Figure 65.

The software to compute the failure probability provides the possibility to use 4 different levels of the phreatic surface, therefor the probability mass function is based on 4 levels of the phreatic surface with a return period of 2 years, 10 years, 100 years and 1000 years.

To determine the combined probability of occurrence (visualized in Table 36) the following two questions need to be answered:

- *What is the cause for a high water level in the drainage canal?*
- *Are the water level in the drainage canal and the phreatic surface (due to precipitation) sensitive for the same type of precipitation event?*

Conditional probability of occurrence	**Ph_1**	**Ph_2**	**Ph_3**	**Ph_4**	$\sum_{j=1}^{4} P(h\|Ph_j)$
h_1	$P(h_1\|Ph_1)$	$P(h_1\|Ph_2)$	$P(h_1\|Ph_3)$	$P(h_1\|Ph_4)$	**$P(h_1)$**
h_2	$P(h_2\|Ph_1)$	$P(h_2\|Ph_2)$	$P(h_2\|Ph_3)$	$P(h_2\|Ph_4)$	**$P(h_2)$**
h_3	$P(h_3\|Ph_1)$	$P(h_3\|Ph_2)$	$P(h_3\|Ph_3)$	$P(h_3\|Ph_4)$	**$P(h_3)$**
h_4	$P(h_4\|Ph_1)$	$P(h_4\|Ph_2)$	$P(h_4\|Ph_3)$	$P(h_4\|Ph_4)$	**$P(h_4)$**
$\sum_{i=1}^{4} P(h_i\|Ph)$	**$P(Ph_1)$**	**$P(Ph_2)$**	**$P(Ph_3)$**	**$P(Ph_4)$**	**1**

Table 36 Combined probability of occurrence water level - phreatic surface due to precipitation

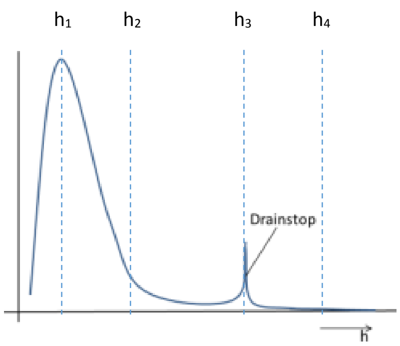


Figure 64 Probability density function water level in drainage canal (Lendering, Jonkman, & Kok, 2015)

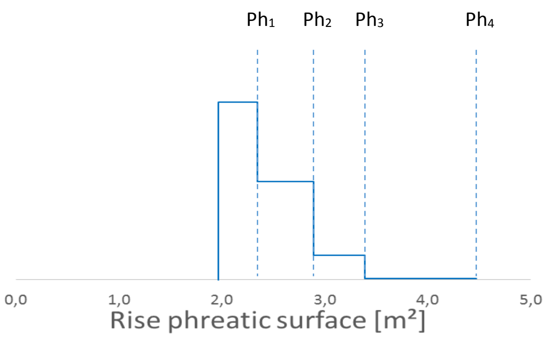


Figure 65 Probability density function rise phreatic surface due to precipitation

Analysis of the cause of a high water level

There are two possible causes which can result in a high water level in the drainage canal. The first possibility is a to low pumping capacity into the outside water, depicted in Figure 67. A low pumping capacity into the outside water is defined as a capacity lower than the inflow into the drainage canal from the polders.

The second possibility, although not likely, is a high outside water level which forces a drain stop from the drainage canal into the outside water. This is possible when a river flood causes the water level in the river to rise to a critical level. If this happens, extra water from the drainage canal into the river could cause a breach in the primary flood defence system. The water level in the drainage canal will rise when the drainage canal cannot discharge in the outside water and the polders keep discharging in the drainage canal which is depicted in Figure 67.

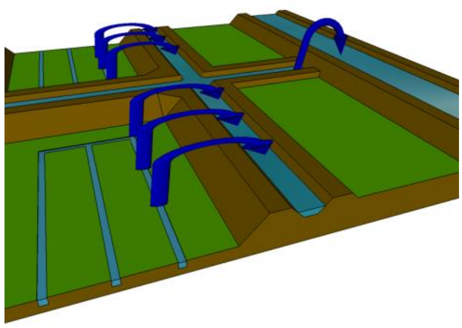


Figure 67 situation 1: Pumping capacity to outside water to low

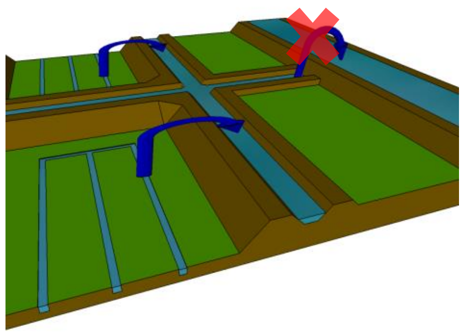


Figure 67 situation 2: pumping capacity to outside water zero due to river flood

Analysis of the polder system should provide information on which one of the two causes is dominant in the system. With the cause of the high water level the next question to determine the combined probability of occurrence of the water level (h) and the phreatic surface due to precipitation (Ph) should answered.

Analysis of the sensitivity of the water level in the drainage canal and the phreatic surface for similar precipitation events

With the analysis of the previous step there are 2 different situations which can occur:

- *Pump capacity into outside water to low:*
  In this case the phreatic surface and the water level in the drainage canal are both sensitive to local precipitation; for both system 'the critical precipitation' event should be determined in order to determine the combined probability of occurrence.
- *Drain stop to outside water level*:
  In this case the reason for a high outside water level should be studied; if the outside water level is mainly influenced by precipitation (far) upstream, then the water level in the drainage canal and the phreatic surface can be assumed uncorrelated.
  If the water level in the outside canal is influenced mainly by local precipitations, then both systems should be analysed on which type of precipitation event causes the extreme levels in step 2.

To determine the type of event for which both systems are sensitive the precipitation events can be categorized as:

- long duration and low intensity;
- medium duration and medium intensity;
- short duration and high intensity;

The duration of the critical event for each combination of saturated and unsaturated permeability is provided in Appendix: E. If both systems (water level and phreatic surface) are sensitive for the same type of event, the water level and the phreatic surface are positively correlated. In this case it is likely that an extreme of both variables occurs at the same time. If both variables are sensitive for completely different types of events both, the extremes are mutually exclusive.

Detailed analysis

After the first analysis is performed it is possible to perform a detailed assessment to determine the relation between the precipitation and the phreatic surface. A detailed assessment requires 2 aspects water level measurements and precipitation data.

With these data sets the relation between the water level and the precipitation can be composed to determine the sensitivity for different precipitation events.

8.2.7 Unconditional failure probability

With the previous step the distribution of the phreatic surface due to precipitation and the water level is composed. The one but last step to compute the probability of failure is to compute the conditional probability of failure in D-geo Stability.

The first step is to enter the schematized cross-section of the dike together with all the soil layers (a non-homogeneous dike consisting of homogeneous soil layers); for all strength characteristics the basic methods apply; for all the soil types which are present in the profile the strength characteristics distribution types and parameters should be entered.

To include all (necessary) combinations of the outside water level and the phreatic surface multiple computations must be performed. The amount of computations equals the amount of different water levels in the drainage canal.

In each computation one outside water level can be simulated together with 4 corresponding (scaled) levels of the phreatic surface; this should be done for all different water levels. Through each computation the conditional probability of failure is obtained.

The last step to compute the total probability of failure regarding macro instability is to integrate over the conditional failure probabilities for all the possible combinations of h (water level) and Ph (level of phreatic surface due to precipitation). Integration results in the following total probability of failure given macro instability:

$$P(F) = \sum_{j=1}^{4} \sum_{i=1}^{n} P\left(F \middle| h_i, Ph_j\right) * P(h_i \cap Ph_j) \tag{10}$$

8.3 Case study on the inclusion of precipitation

This thesis study originated from the study (Lendering, Jonkman, & Kok, 2015). In that study a probabilistic method is proposed to quantify the failure probability of a regional flood defence. For a failure probability computation regarding macro stability, the uncertainty of the phreatic surface is an important variable; because no statistical information was available on the probability distribution of the phreatic surface an assumption, based on expert judgement, is used.

In this thesis study an alternative method is developed to assess the uncertainty of the phreatic surface for a failure probability analysis. In this chapter the method based on expert judgement and the method developed on this thesis study are compared. The comparison between the two methods is illustrated with a short case; the focus of the case study is to highlight the differences between the two methods and to illustrate the advantages of the newly developed assessment method of the phreatic surface.

The illustrative case study is performed on a case from the HHNK (Hoogheemraadschap Hollands Noorderkwartier, 2014) in the area of Heerhugowaard; the case is used as well in (Lendering, Jonkman, & Kok, 2015). The HHNK has divided the polder into 17 sections which are depicted in Figure 68. For this study section 17 is used as for the case study. First the information on section 17 is provided. Next the schematization according to both methods is elaborated; the schematization according to the method developed in this study is elaborated in detail, the schematization from (Lendering, Jonkman, & Kok, 2015) is presented in a short and factual manner. In the last paragraph the schematization methods are compared.

In the case study only 1 level is used for the water in the drainage canal combined with multiple levels for the phreatic surface; to illustrate both methods and to identify the differences it suffices to use only 1 water level in the drainage canal, instead of a range of water levels.

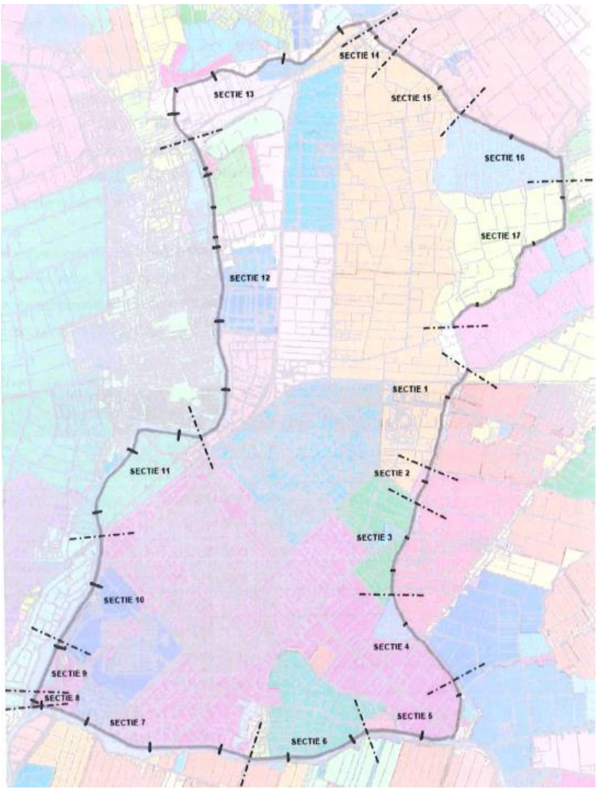


Figure 68 Overview sections Heerhugowaard (Hoogheemraadschap Hollands Noorderkwartier, 2014)

8.3.1 Case description

Section 17 from the polder Heerhugowaard is the subject of the case study; a schematization of cross-section 17 is presented in Figure 69. The dike in section 17 consists of clay on top of a sand base. The permeability of the dike material, clay and peat is not known; the permeability of the sandy layer below the dike is 1,16 $*10^{-5}$ m/s. In a daily situation the water level in the drainage canal is NAP -0,6 m; the water level in the polder is NAP -2,9 m. The water level difference in a daily situation is thus 2,3 m. The daily water level is used in the case study. For this section no measurements are available on the effect of precipitation.

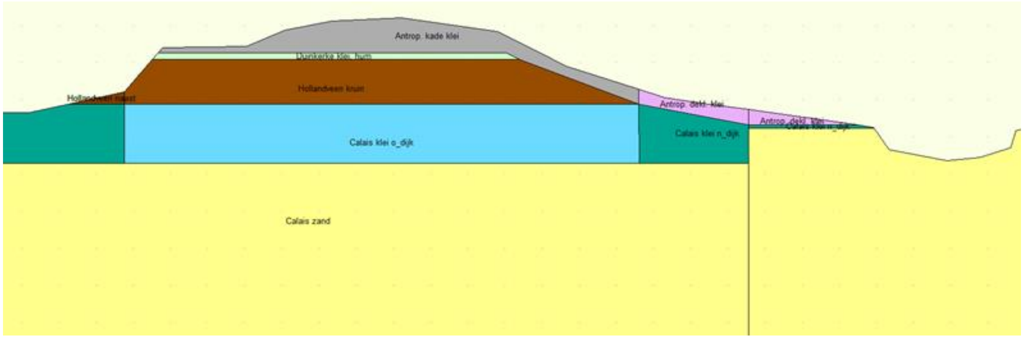


Figure 69 Cross-section of section 17 (Hoogheemraadschap Hollands Noorderkwartier, 2014)

8.3.2 Schematization distribution phreatic surface according to the method used in (Lendering, Jonkman, & Kok, 2015)

In this method the phreatic surface is schematized as three discrete levels, a high, medium and low level with a probability density of respectively 15%, 80% and 5%.

In this section no head measurements are available; the high, average and low phreatic surface are schematized as depicted in Figure 70. All three levels are based on expert judgement.

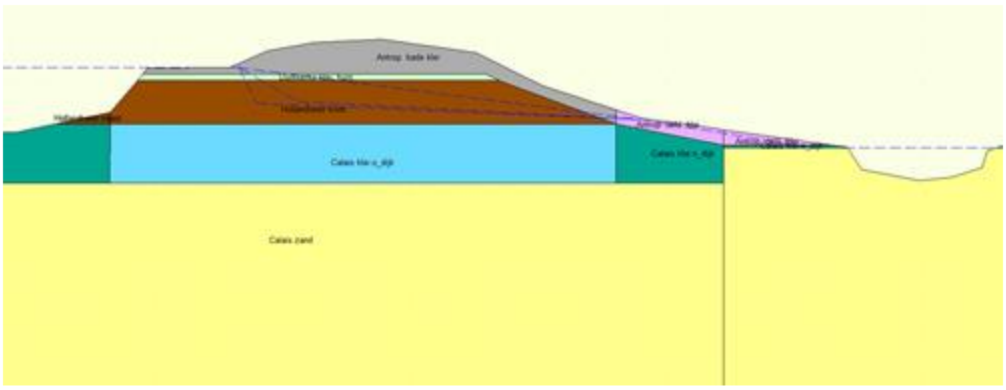


Figure 70 Illustration schematization phreatic surface section 17; original method (Lendering, Jonkman, & Kok, 2015)

Schematization phreatic surface according to the method developed in this thesis study

In the method developed in this thesis the assessment of the phreatic surface is according to three steps. The result after the third step is a set of 4 different phreatic surfaces corresponding to a phreatic surface with a yearly probability of exceedance of 1/1000, 1/100, 1/100 and 1/2. The corresponding probability of occurrence is 0,55%, 5%, 25% and 45%.

1. Determine initial/neutral phreatic surface;
2. Determine hydraulic characteristics of the dike;
3. Scale the effect of precipitation on the phreatic surface according to the scaling tables;

Because no measurements of the hydraulic head are available, the average /initial phreatic surface is estimated to be the linear connection of the polder water level and the water level in the drainage canal.

The second step in the schematization is to determine the saturated permeability of the dike; the saturated permeability determines how fast the infiltrated water can be discharged through the toe of the dike. Because the dike is based on a sandy layer and the water will be discharged through the sand, the permeability of the sandy layer is used in the computation; in this case the possible options for the permeability are $2{,}8*10^{-5}$ m/s or $2{,}8*10^{-6}$ m/s. Because $2{,}8*10^{-5}$ m/s is closest to the measured value, this value is used in the computation.

The dike consists of clayey material, therefor the unsaturated part of the dike will behave as a clay dike; therefor the unsaturated characteristics from clay are used in the computation.

For a basic dike of clay, with a permeability of $2{,}8*10^{-5}$ m/s the probability distribution of the rise of the phreatic surface is provided in Table 37.

Increase phreatic surface [m^2] basic dike	1/1	1/2	1/10	1/100	1/1000
k=$2{,}8*10^{-5}$ m/s	2,0	2,6	3,7	6,5	10,4

Table 37 Rise of phreatic surface [m^2] in a basic dike of clay with a permeability of $2{,}8*10^{-5}$ m/s, per frequency of exceedance

The last step in the schematization is the scaling of the effects, because the dike in section 17 has slightly different dimensions from the basic dike used in this thesis; the scaling is performed according to Table 32 and Table 35 (scaling tables for a dike consisting of clay). The dike in section 17 is 1 meters wider than the dike in the basic computation, therefor the relative rise of the phreatic surface is 99,5% of the original value; because the water level in section 17 is closer to the crest than in the basic situation the phreatic surface for each probability of exceedance will increase with 15%.

Increase phreatic surface [m^2] section 17	1/1	1/2	1/10	1/100	1/1000
k=$2{,}8*10^{-5}$ m/s	2,3	3,0	4,2	7,4	11,8

Table 38 Rise of phreatic surface [m^2] in section 17, per frequency of exceedance

An illustration of the resulting schematization of the high phreatic surface according to the assessment method from this theses is presented in Figure 71.

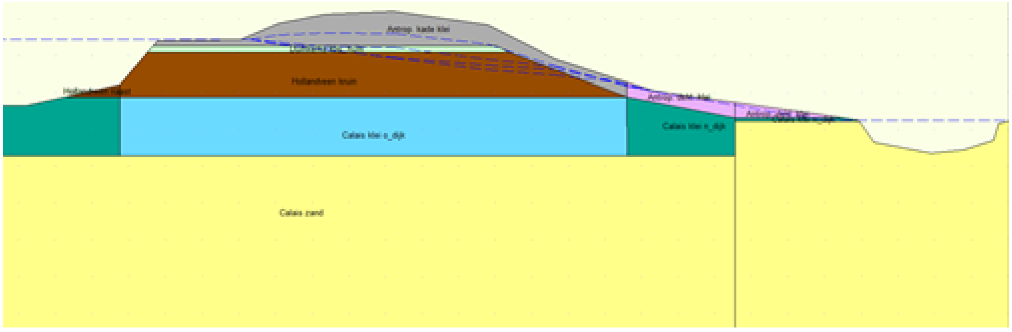


Figure 71 Illustration schematization phreatic surface section 17; new method

8.3.3 Comparison schematization methods

Spectrum of the phreatic surface

The methods show some clear differences in the assessment of the phreatic surface. The first important difference is that in the original method the spectrum of the phreatic surface ranges from a low level to a high level. In the method developed in this thesis study only the high spectrum of the phreatic surface is included; this results in a larger probability of occurrence for the high phreatic surface in the new method, which results in a larger probability of failure.

When regarding the probability of failure due to macro instability, the dike can fail due to a high phreatic surface or due to a low phreatic surface; the result of the failure probability computation is a failure probability per year. In the computation, the probability of occurrence of a level of the phreatic surface is thus a probability per year; when the phreatic surface is observed during 1 year, the phreatic surface will reach a maximum level *and* a minimum level. Therefor the total probability density under the curve of the rise of the phreatic surface is 100% and the total probability density of a decrease of the phreatic surface is 100% as well. Because this study focussed on the rise of the phreatic surface, only the PDF of the rise of the phreatic surface is composed. The PDF of the decrease of the phreatic surface is not included in this study.

Effect of precipitation

In the new developed method, the effect of precipitation is clearly included. However in the schematization of the phreatic surface in the original method, the effect of precipitation on the phreatic surface is not noticeable. Although the effect of precipitation is not noticeable, the report (Hoogheemraadschap Hollands Noorderkwartier, 2014) does state 'the shape of the phreatic surface is conservatively estimated'. The reason of this fact is unknown; a possible reason can be that in the other sections, where head measurements where available, the average phreatic surface was lower than a linear phreatic surface. The best guess is that the water board might have thought that a linear schematization would be conservative enough given that the average phreatic surface is lower than a linear connection.

Use of measurements

In some sections, head measurements are used. The head measurements show an average phreatic surface which is lower than the phreatic surface in a simple schematisation. The average phreatic surface, from the head measurements, can be used in a simplified manner in the new method. In reality the lower phreatic surface will have a complex influence on the degree of saturation of the top layer and thereby on the initial permeability. Therefor the infiltration will be lower than used in the new method.

Geometry

The current assessment method in the TRWD, used in the approach of (Lendering, Jonkman, & Kok, 2015), can include any range of dike widths. The newly developed method is thus far not able to assess the phreatic surface in broad dikes. This is a disadvantage of the new method; in future study the disadvantage should be minimized by either a more thorough sensitivity study on the effect of the dike width or by constructing an empirical formula. The geometry of the dike in section 17 is different from the geometry used in this study, so far only expert judgement can be used to translate the computed phreatic surface to the phreatic surface in the case.

Permeability

In this case study the dike, consisting of clay (and sometimes sand and clay) is located on a thick layer of sand. Because a sandy layer increases the possibility on hydraulic short circuiting measurements are conducted to determine the hydraulic conductivity of the sand layer. This provided useful information to assess the probability density function of the phreatic surface.

In practice the hydraulic conductivity of the dike foundation is often not know; therefor expert judgement is needed to use the new method, through this the accuracy of the method will decrease. There is no method yet, to translate for example head measurements to a value for the hydraulic conductivity.

8.4 Discussion

The framework of the method to include the uncertainty of the phreatic surface contains several valuable aspects. The first aspect is scientific insight in the reaction of the phreatic surface on precipitation. The second aspect is insight in how to include the effect of precipitation in the uncertainty of the phreatic surface.

The framework of the method is however limited in detail, due to the scope of the study. Therefore improvement of some aspects is possible and desired.

8.4.1 Use of stochastic soil data instead of deterministic values for the permeability

The scientifically most complete method to describe the uncertainty of the phreatic surface in a failure probability analysis would be: a model based on Monte Carlo simulations of both the non-homogeneous hydraulic soil characteristics (as described in ‘A stochastic approach to rainfall-induced slope failure’ (Arnold & Hicks, 2011)) and the precipitation duration-intensity ratio.

The most accurate way to determine the effect of the uncertainty of the phreatic surface on the stability is to include the Monte Carlo simulation of the phreatic surface in the model to compute the failure probability. In this case the computation would result in the probability of failure distribution instead of only the distribution of the phreatic surface.

Scientifically this approach would gain the most accurate answers; however this approach is time consuming and only useful to gain scientific insights. For practical application this method is not advised.

8.4.2 Relation between precipitation and the phreatic surface

In the framework the relation between the rise of the phreatic surface and the precipitation is defined as: $P_{exc,crit\ prec} = P_{exc,crit\ phr}$. The relation is based on the assumption that each IDF curve is based on 1 single precipitation event. Because the assumption is an underestimation of the probability of exceedance the resulting probability distribution results a lower limit of the failure probability; further study is necessary to determine the average rise of the phreatic surface.

There are two methods to validate or replace the assumption ($P_{exc,crit\ prec} = P_{exc,crit\ phr}$). The first method is with the use of a complete precipitation data series from the KNMI. The second option is to categorize precipitation events and rank the events with the use of copula. The result of both methods is a more accurate representation of the probability density function of the phreatic surface (conditional on the geometry and the hydraulically soil parameters). Both methods are briefly touched upon in the next paragraphs.

Use of real data series to determine the PDF of the phreatic surface

A possibility to determine the PDF of the phreatic surface is with the use of raw precipitation data. In the method the raw data from the past 50 to 100 years, collected by the KNMI, can be used to compute the phreatic surface inside the schematized dike. After the computation the yearly maximum phreatic surfaces can be identified and ranked. Next the PDF curve can be computed for the phreatic surface. In this method it is not necessary to determine the characteristics of critical event in advance. An illustration of the procedure is depicted in Figure 72.

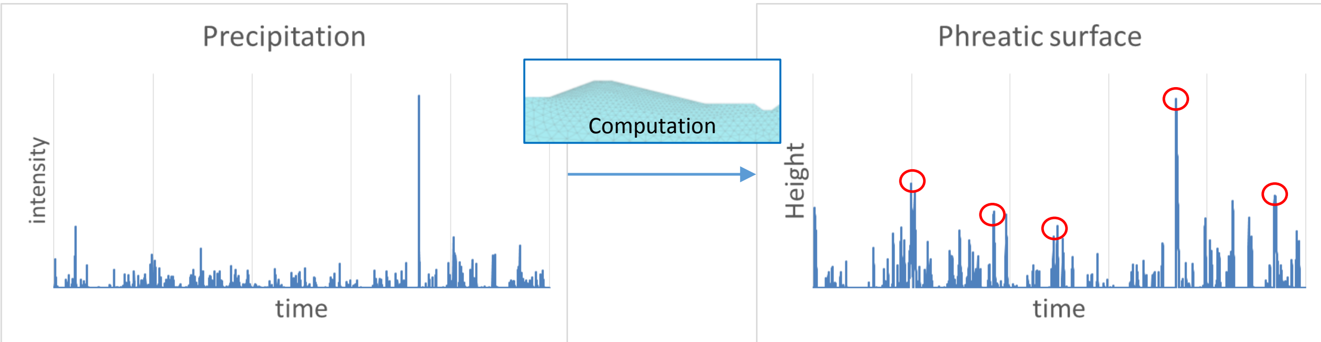


Figure 72 Illustration method based on precipitation series

An interesting possibility of this method is that it might be possible to relate the probability of occurrence of a precipitation event to the probability of occurrence of the rise of the phreatic surface. In this way it might be possible to determine how much the assumption in the method approached the true uncertainty of the phreatic surface.

If a computation with the entire set of precipitation data results in similar distributions of the phreatic surface, then the simple method in which '$P_{exc,crit\ prec} = P_{exc,crit\ phr}$' can be applied in the future. If the computation proves that the results from the simple schematization is a large underestimation then the first method should be rejected.

Another feature of this method is that it includes detail on the shape of a precipitation event. The shape of the event might influences the total rise of the phreatic surface; for example an event with a peak in the beginning will result in a smaller effect on the phreatic surface than an event with a peak at the end. The significance of the simplification of the shape of the event is not yet known.

A disadvantage of this method, is that the results only gives insight in the PDF of the phreatic surface without a physical relation to the precipitation event. In this case it is hard to link the precipitation event (which caused the rise of the phreatic surface) with for example the water level rise. For a general understanding and scientific knowledge it is useful to describe the rise of the phreatic surface in a more elementary way.

The largest disadvantage of this method is the computation time which is needed to determine the CDF curves; to compute the annual maximum rise of the phreatic surface 50 to 100 years of quarter hourly data must be simulated for multiple combinations of soil type and permeability. With Plaxflow the computations would take too much time to be of use for this study.

Joint probability of precipitation intensity and duration with the use of copulas

The second option to compute the probability distribution of the phreatic surface is with the use of a copula.

The method to rank precipitation events with the use of copula is proposed in the papers '*Climate change in asset management of infrastructure: A risk-based methodology applied to disruption of traffic on road networks due to the flooding of tunnels'* (Huibregtse et al., 2016) and 'Characterization *of precipitation through copulas and expert judgement for risk assessment of infrastructure'* (Morales Nàpoles et al., 2016).

This chapter only briefly describes the possibilities of using a copula; for more scientific detail on the background of copula or the practical application the report (Joe, 2014) should be advised.

With the use of a copula it is possible to describe the relation between the precipitation duration and intensity. This can be done with the use of the precipitation data series from the KNMI which are publically available (STOWA, 2016). The data series can be plotted and a copula, which describes the joint probability, can be fitted to the data to generate random samples; 'a bivariate copula which fits the Dutch precipitation data is the Gumbel copula' (Huibregtse et al., 2016).

The largest benefit of the use of a copula is that the amount of precipitation events is accounted for without the use of a continuous data series. The computation of the distribution of the phreatic surface is based on single events which reduces the computation time significantly.

A point of importance is the definition of 'an event'; in a copula the precipitation events are ranked to determine the joint probability of occurrence. The definition of an event is arbitrary; the phreatic surface in some dikes is sensitive for long events, even with dry periods in between, whereas the phreatic surface in some dikes is sensitive for short events. In the analysis to distinguish the separate events the duration without precipitation is the criterion for the end of an event, this duration should be determined with care.

It would be scientifically useful to compose this method; when this method is composed it could be easily used in different circumstances to compute the distribution of the phreatic surface.

8.4.3 Empirical formula for the infiltration of precipitation

In the current method the phreatic surface is based on a limited set deterministic values of the soil type and the permeability for a homogeneous dike. In order to improve the method an empirical formula could be created to describe the rise of the phreatic surface. The first two variables in the formula would be:

- Infiltration capacity (cover layer);
- Discharge capacity (core and toe of the dike);

The advantage of an empirical formula is that the rise of the phreatic surface can be computed for any degree of permeability instead of only 4 discrete values. The infiltration capacity can be determined on the base of a SWCC, the discharge capacity can be based on the saturated permeability underneath the toe of the dike.
The discharge capacity determines the final height of the phreatic surface given a certain storm

intensity and duration. The infiltration capacity then determines the speed with which the phreatic surface rises towards the maximum level.

The formula would consist of two parts:

- Computation of the total rise of the phreatic surface in [m^2];
- Computation of the horizontal 'travel' speed of the increase of the phreatic surface as is depicted in Figure 73;

This additional advantage of this approach is that it can include the effect of a drainage system or reduced or increased infiltration.

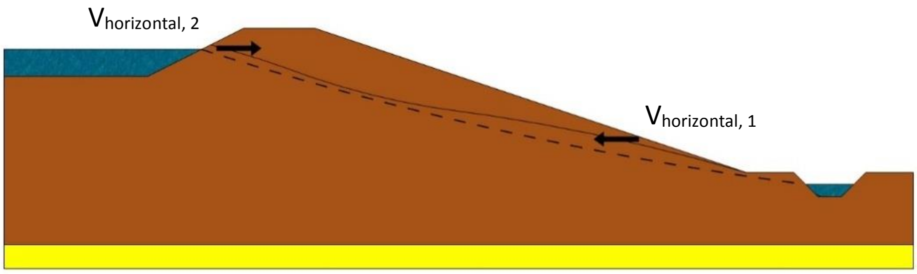


Figure 73 Illustration horizontal travel speed phreatic surface under influence of precipitation

8.4.4 Translation of the computed phreatic surface to a case

The rise of the phreatic surface which is computed for a single profile can be used for different cross-sections. However the translation can be improved in future studies. The difficulty arose because the lower part of the inner slope of section 17 was steeper than the upper part. Therefor the computed phreatic surface would be situated above surface level; this is not possible. Therefor expert judgement is used to translate the computed phreatic surface to the case study. The results give an indication on the levels of the phreatic surface which can be expected.

8.5 Conclusions

The framework constructed in this study is a concept method to take the uncertainty of the phreatic surface into account. It takes the uncertainty of the phreatic surface into account, based on the effect of saturated permeability (homogeneous and deterministic), unsaturated permeability (homogeneous and deterministic), precipitation statistics (homogeneous) and dike geometry (homogenous).

The new method includes the effect of precipitation on the phreatic surface; with the new method it is possible to estimate the probability distribution of the phreatic surface in a homogeneous dike consisting of peat, clay or sand with a permeability varying between $2{,}8*10^{-5}$ m/s and $2{,}8*10^{-8}$ m/s. Furthermore the method provides a tool to determine the combined probability of occurrence of a high water level and the phreatic surface inside the dike due to precipitation.

When the method is compared to the original approach in (Lendering, Jonkman, & Kok, 2015) there are some clear differences. The method developed in this thesis study ascribes a probability density of 100% to the spectrum of high phreatic surfaces and 0% to an average phreatic surface; the method in the original approach ascribes only 15% to a high phreatic surface (and 80% to an average and 5% to a low phreatic surface). The reason for the new allocation of probability density is that the failure probability is computed per year. The average phreatic surface is exceeded every year with a 100% probability and is there for not relevant for the failure probability computation.

In the method the dike is schematized as a homogeneous soil body; the hydraulic soil characteristics are however heterogeneous in practice, this influences the infiltration capacity and the shape of the phreatic surface. Without the use of specialized expensive software it was not possible to include heterogeneity yet. A possible improvement can be the addition of heterogeneity by distinguishing between the cover layer, the core of the dike and the drainage of the dike.

9 Conclusions and discussion

The study on the uncertainty of the phreatic surface consisted of 2 main parts: a study on the influence of precipitation on the phreatic surface and a study on the relation between probability distribution of the precipitation, the water level and the phreatic surface.

The results of the study provide valuable insights in the behaviour of the phreatic surface in dikes consisting of different soil types, influenced by different precipitation events. In chapter 9.1 the conclusions of the study are presented. In chapter 0 and chapter 0 the discussion and the recommendations are presented.

9.1 Conclusions

The framework that is constructed in this study provides a more scientific basis for the probability distribution of the rise of the phreatic surface; furthermore it provides insight in the behaviour of the pore pressures under influence of precipitation. In this chapter answers on the research questions are provided.

Influence of the phreatic surface on the stability of a regional flood defence

The degree of stability of the inner slope of a regional flood defence is determined by the geometry, soil characteristics (i.e. angle of internal friction and cohesion) and the phreatic surface. Instability will occur when the driving force is larger than the resistance. The main driving force is the weight of the driving soil body and the main resisting force is the shear force along the slip surface. The shear force is generated through the effective stress in the soil.

The role of the pore pressures in the stability computation, schematized as a phreatic surface, is a reduction of the effective stress. When the phreatic surface rises, the pore pressures will increase; the increased pore pressures result in a lowered effective stress, which will negatively influence the stability. The phreatic surface is the main variable that varies over time, which can cause macro instability.

Pore pressure development under the influence of precipitation

Precipitation on a dike acts as a recharge of the groundwater. The precipitation will infiltrate in the dike, the infiltration rate depends on the permeability of the top layer of the dike. When the infiltration capacity is larger than the precipitation intensity, all the precipitation will infiltrate, when the infiltration capacity is smaller than the precipitation intensity not all the water will infiltrate, the excess water will run-of and will not influence the phreatic surface.

The infiltrated water flows through the dike towards the polder, in order to discharge the infiltrated water through the network of pores, a difference in pore pressure between the top of the dike and the toe of the dike is necessary. The pore pressures inside the dike will rise due to the infiltration, until the pressure in the dike is large enough to facilitate the discharge of the infiltrated water. The (saturated) permeability of the dike determines the maximum rise of the phreatic surface given a precipitation event. In some cases the phreatic surface will rise up to the surface level.

The pore pressure rise due to precipitation is non-hydrostatic; following Darcy's law, water will only flow when there is a pressure difference. The infiltrating water flows through the dike from the top towards the toe of the dike. Because there is a vertical component of the flow there is a head difference in the vertical direction. The effect of precipitation on the pore pressures is the largest in the top layer and diminishes over the depth as is depicted in Figure 75.

Assessment of the probability distribution of the phreatic surface

The phreatic surface is influenced by the precipitation and is thus related to the precipitation. In this study, the relation between the phreatic surface and the precipitation is based on the statistical information of the 'intensity-duration-frequency curves' from the Bilt. The IDF curves provide information on the probability of exceedance for different combinations of intensity and duration; this makes that there are different precipitation events with the same probability of exceedance.

The relation between the IDF curves and the corresponding rise of the phreatic surface is defined as: The precipitation event, from all the combinations of intensity and duration on one IDF curve, which causes the largest rise of the phreatic surface, is called the critical event; the exceedance frequency of the corresponding rise of the phreatic surface is assumed equal to the exceedance frequency of the precipitation event. The implicit assumption of this definition is that each IDF curve is based on one single event; in reality each curve is based on a (unknown) small amount of events, therefor the defined relation provides a lower boundary of the phreatic surface.

The probability distribution of the phreatic surface is computed for 11 combinations of the saturated and unsaturated characteristics; the saturated characteristics are schematized as 4 degrees of permeability (i.e. k: $2{,}8*10^{-5}$, $2{,}8*10^{-6}$, $2{,}8*10^{-7}$ and $2{,}8*10^{-8}$ m/s) and the unsaturated characteristics are based on the unsaturated flow in peat, sand or clay. The results of the computation with a permeability of $2{,}8*10^{-8}$ m/s are not included because they are too conservative. The probability density function of the rise of the phreatic surface for the 9 combinations is presented in Figure 74.

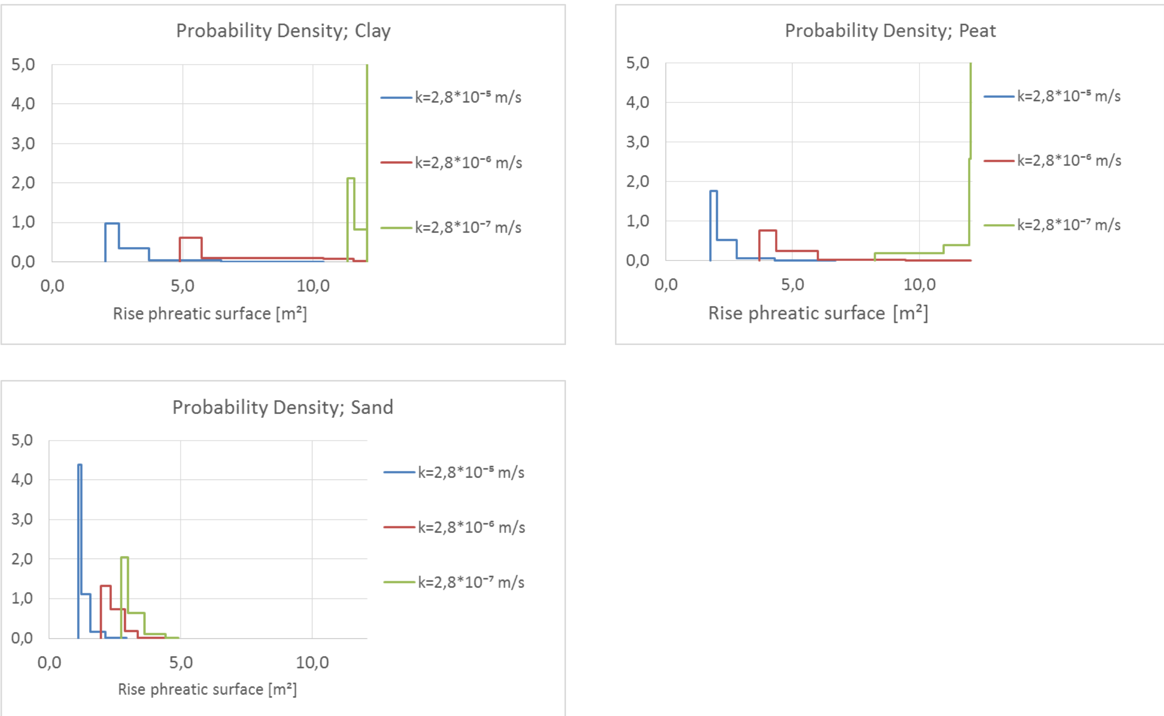


Figure 74 Probability density functions for a dike of clay, peat and sand for different degrees of permeability

Comparison pore pressure schematisation in the conventional deterministic method and the new method

The main difference between the information found in this study and the current schematization (TAW, 2004) of the pore pressures, is the pore pressure distribution over the depth. The current guidelines prescribe a maximum rise of the phreatic surface which is 0,3 m below the crest (in a dike consisting of clay). The schematization of the pore pressures is hydrostatic below the phreatic surface, the resulting pore pressures (for the geometry used in this study) underneath the inner crest line are presented in Figure 75 as the red line.

In this thesis study the maximum possible pore pressures due to precipitation are computed and are represented by the blue line. It is clear the schematization used in the TRWD overestimates the pore pressures in a large part of the dike; from 1 meter below the crest and downwards the pore pressures are overestimated if the conservative approach is chosen.

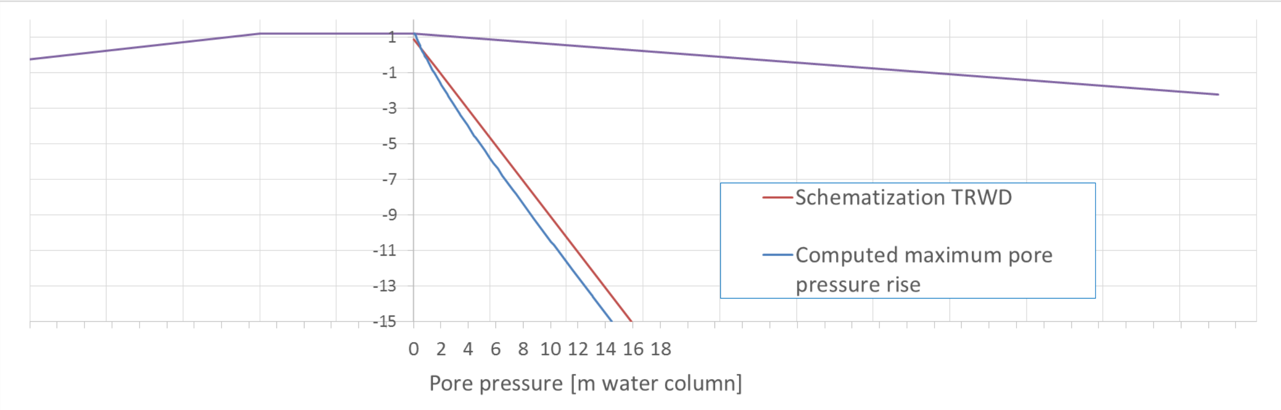


Figure 75 Comparison of the pore pressure, in meters water column, between the TRWD (hydrostatic) and the computation (non-hydrostatic)

Comparison pore pressure schematisation in the conventional deterministic method and the new method

The original method to assess the probability distribution of the phreatic surface (Lendering, Jonkman, & Kok, 2015) consists of three levels for the phreatic surface (i.e. low, average and high) as depicted in Figure 77. There are 3 main differences between the original method and the proposed framework.

The first difference between both methods is that the new method contains information based on the permeability of the dike in 11 different combinations (i.e. sand, clay or peat with a 4 different options for the permeability) whereas the original method only contains information based on two different configurations (sand or clay).

The second difference is in the probability mass function. In the original method the probability mass function contains a low, an average and a high phreatic surface. In the new method the average phreatic surface is not a part of the probability density function as is depicted in Figure 76. The reason the average phreatic surface does not influence the yearly probability of failure in the computation is because the failure probability has the unit ‘probability per year’. The average phreatic surface is exceeded countless times per year; only the values for the phreatic surface which are exceeded less than once a year will influence the probability of failure.

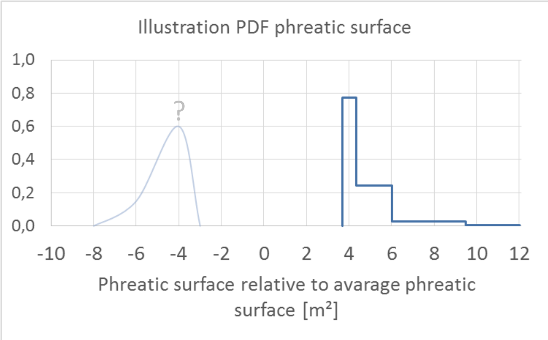


Figure 76 Illustration probability density phreatic surface according to the method developed in this thesis; the shape of the probability density of the low phreatic surface is not studied and randomly chosen.

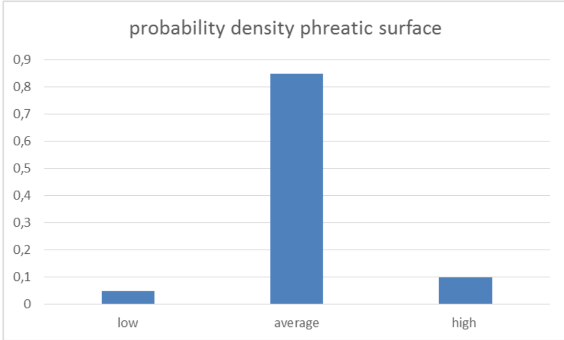


Figure 77 Probability density phreatic surface according the original method (Lendering, Jonkman, & Kok, 2015)

The second large difference in the probability mass function is that, in the original method, the probability mass function contains both the low (5% probability density) and the high phreatic surface (10% probability density); together with the average phreatic surface the probability mass was 100%. In the new method, the probability density of the high phreatic surfaces together are 100% and the probability density of the low phreatic surfaces are 100% as well. During one year, both an extreme low and an extreme high phreatic surface will occur, therefor the two extremes should be regarded separately. The resulting probability of failure according to the new schematization is thus larger than in the original method because the high phreatic surface has a larger probability of occurrence.

The cases that are studied in the original method show that the average pore pressures are mostly lower than the expected phreatic surface in a homogeneous dike; the expected linear phreatic surface is used as starting point for the schematization. This stretches the importance of field measurement in order to make an accurate computation of the failure probability.

9.2 Discussion

Relation phreatic surface and precipitation

In this study the exceedance probability of the phreatic surface is related to the critical precipitation intensity corresponding with that probability of exceedance. This approach assumes that each IDF curve is based on 1 precipitation event; in reality each curve is based on the statistical information of an unknown limited amount of events. This means that the probability density functions which are found in this study are a lower boundary for the phreatic surface. Because each IDF curve is based on a limited amount of precipitation events it is assumed that the lower limit, computed in this study, approaches the mean value.

Homogeneous soil body

The computations are performed with the use of a homogeneous soil body; this is a simplification of the reality where a dike consists of various heterogeneous layers. Therefore the simple schematization of the phreatic surface, not based on head measurements, is likely to be different from the real shape of the phreatic surface. Therefor it is important to measure the head inside the dike in order to calibrate the schematization.

Uniform precipitation events

The precipitation events in the study have an uniform intensity pattern which is a simplification of the reality. In nature precipitation events can have various shapes, for example peaked in the beginning, peaked at the end or with multiple peaks. For relatively permeable soils (up to $2{,}8 * 10^{-7}$ m/s) the uniform schematization is valid, because the infiltration capacity is high enough to prevent major run-off. For dikes with a permeability lower than $2{,}8 * 10^{-7}$ m/s the simplification of the precipitation event is not valid because only the time with actual precipitation influences the phreatic surface disregarding the intensity. Therefor the results belonging to the computation with $2{,}8 * 10^{-7}$ m/s are not included in the method.

9.3 Recommendations

Empirical formula rise phreatic surface

In this study the relation between the precipitation and the distribution of the phreatic surface is based on a limited amount of computations. For future studies it is recommended to determine an empirical relation between the precipitation intensity, duration, permeability of the dike, geometry and the infiltration capacity in order to clearly include the physical aspects. This would provide a more general method to assess the phreatic surface. The empirical formula should compute the horizontal penetration of the rise of the phreatic surface as is depicted in Figure 78.

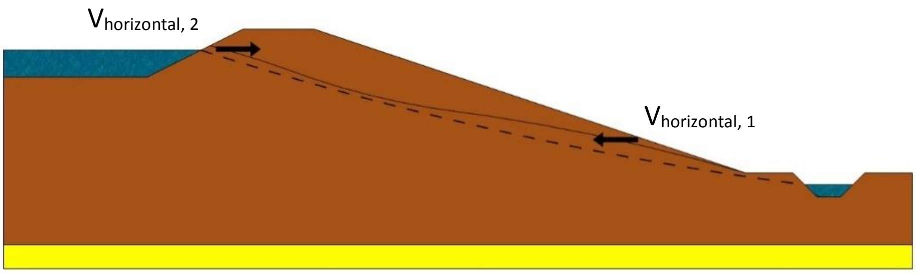


Figure 78 Illustration horizontal travel speed phreatic surface under influence of precipitation

Validation relation precipitation and phreatic surface

In this study, a relative simple relation is formulated between the exceedance frequency of the phreatic surface and the exceedance frequency of a critical precipitation event; the relation should be validated in order to check how close the assumption is to the mean value.

To validate the relations two methods are proposed. The first option is a simulation of a full precipitation data series on a geometry used in this study. The resulting phreatic surface should be analysed with the annual maxima method in order to compute the statistical information. The information should be compared to the probability density functions resulting from this thesis study.

Step size permeability

In this method the permeability is included in 4 discrete options; there are large differences in the results between the 4 possible options; therefor it would be useful to make the step size smaller in order to obtain more accurate results.

Hydraulic soil parameters

In the study on the behaviour of the phreatic surface under influence of the precipitation, 3 soil types are selected (i.e. sand, peat and clay). For each soil type typical values for the unsaturated characteristics are assumed based on the Staring data series. The unsaturated characteristics describe the permeability of the soil depending on the degree of saturation; the description of the unsaturated permeability is only depending on the soil type and not on local circumstances. In practice, vegetation or the presence of animals will influence the permeability of the topsoil, therefor the unsaturated characteristics for a dike differ from the characteristics of agricultural land. In the future it would be wise to compose SWCC for dike materials as well.

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Appendix: A Glossary

Boezem canal	The canal which drains the water from a polder towards a river or canal;
cumulative	Cumulative density function;
Drainage canal	The canal that transports the excess water from the polder towards a large river or the sea;
IDF	Intensity duration frequency curve;
Failure	Loss of the water retaining function through loss of height;
FEM	Finite Element Method;
Outside water	The water system which is no part of the polder system (i.e. the sea or a large river)
PDF	Probability density function;
Plaxflow	Finite element software package;
Phreatic surface	An imaginary surface where the pore pressure equals the air pressure;
Relative crest height	The difference between the water level and the crest level;
STOWA	Foundation for applied water research;
SWCC	Soil water characteristic curve; describes the ability of the soil to keep water under stress;
TRWD	Technical report pore pressures in dikes (TAW, 2004);

List of symbols

Angle of internal friction	Φ	Pore pressure	u
Cohesion	c	Probability of failure	P_F
Effective stress	σ'	Reference level	z
Elastic bulk modulus	K	Resistance	R
Factor of safety	F_s	Saturation	S_{sat}
Failure	F	Shear force	τ
Flow	q	Solicitation	S_{sol}
Gravitational constant	g	Specific weight of water	ρ_w
Level of the Phreatic surface	Ph	Time	t
Moment (Driving)	M_D	Total stress	σ
Moment (resistance)	M_R	Velocity	v
Permeability	k	Water level	h
Porosity	n		

Appendix: B List of figures and tables

Figures

Tables

Appendix: C Technical description of Plaxflow

General information on the computation steps

The first step to perform the computation is the definition of a 2D soil profile. The dimensions of a soil profile can be entered together with an estimation of the ground water table. For all the soil types in the model the hydraulic properties must be given in order to perform the flow computations.

The second step is the computation of the 'initial' ground water conditions; Plaxflow computes the initial condition according to the user defined geometry, geology and (hydraulic) boundary conditions. The result of the computation is the initial condition for:

- The pore pressure in each element;
- The degree of saturation in each element;
- The permeability of each element;
- The flow through each element;

Ground water flow

The flow of water through the ground consists of two parts, the flow through the saturated zone and the flow through the unsaturated zone. Because the flow through the soil, the flow velocity and the degree of saturation influence each other the following computation steps are performed simultaneously by Plaxis:

- computation of the degree of saturation (unsaturated zone);
- computation of the permeability;
- computation of the flow through the element;
- computation of the pore pressure;

The flow through the soil body depends on the permeability of the soil and the pressure gradient over an element. The discharge is computed according to Darcy's law for flow through porous media:

Darcy's law: $\underline{q} = -\frac{\underline{\underline{k}}}{p_w g}(\underline{\nabla} p_w - p_w \underline{g})$ (11)

With:

$\underline{q}$= specific discharge;

$\underline{\underline{k}}$= tensor of the permeability;

$\underline{g}$= gravitational acceleration vector;

p_w= density of water;

In unsaturated conditions the permeability is related to the saturated permeability and reads:

$$\underline{\underline{k}} = k_{rel}\underline{\underline{k}}^{sat} \tag{12}$$

In which

$$\underline{\underline{k}}^{sat} = \begin{matrix} k_x^{sat} & 0 \\ 0 & k_y^{sat} \end{matrix} \tag{13}$$

Simultaneously with the flow computation, the degree of saturation for each element is computed with the continuity equation. The continuity equation in Plaxflow is based on (Song, 1990) and is simplified by assuming 1) no particle movement and 2) no gradients in the density of water (Boussinesq's approximation) (Plaxis, 2016)

Continuity equation: $$\nabla^T \left(\frac{k_{rel}}{p_w g} \underline{\underline{k}}^{sat} \left(\underline{\nabla} p_w \underline{g}\right)\right) - n \left(\frac{S}{K_w} - \frac{\partial S}{\partial p_w}\right) \frac{\partial p_w}{\partial t} = 0 \qquad (14)$$

With:

n=porosity
S=saturation
K_w=Elastic bulk modulus of water

The last function which determines the end result of a groundwater flow computation is the formula for the relative permeability. When a soil element is not fully saturated the permeability of that element is a percentage of the saturated permeability. The relative permeability based on the saturation and is computed with the following formula (S_e, and 2 fitting parameters g_n and g_l)

Relative permeability: $$krel(S_e) = \max\left[S_e^{g_l} \left(1 - \left[1 - S_e^{\frac{g_n}{g_n - 1}}\right]^{\frac{g_n - 1}{g_n}}\right)^2, 10^{-4}\right] \qquad (15)$$

The saturation is determined by the van Genuchten soil water retention curve. This curve describes the relation between pore suction and the degree of saturation.

Saturation: $$S(\phi_p) = S_{res} + (S_{sat} - S_{res}) \left(1 + \left(g_a |\phi_p|\right)^{g_n}\right)^{g_c} \qquad (16)$$

In which:
ϕ_p= pressure head;
S_{res}=residual saturation
S_{sat}=determines the degree of saturation when the pores are maximally saturated; the default is '1';
g_a=fitting parameter related to the air entry value of the soil;
g_n=fitting parameter related to the rate of water extraction when the air entry value is exceeded;
g_c= Fitting parameter related to the van Genuchten equation;

The values of the fitting parameters can be measured or chosen from standard soil parameter data sets.

Groundwater flow theory

In Plaxis the module Plaxflow provides a numerical framework for the ground water flow. The framework is based on Darcy's law on flow through porous media.

To describe the flow through unsaturated soil a soil water characteristic curve (SWCC) is used; the equations describe the hydraulic behaviour given a certain moisture content of the soil and the ability to hold water at different stress levels.

The model used to define the degree of saturation is the Van Genuchten model. The saturation is described depending on three different parameters and relates the saturation to the suction head.

Parameter description

In groundwater flow parameters used in Plaxflow are described below.

$S(\phi_p)$	[]	Saturation
S_{res}	[-]	Residual saturation which describes the part of the fluid which remains in the pores even at high suction
S_{sat}	[-]	Compensation factor for trapped air ≤ 1,0
S_e	[-]	Effective saturation
g_a	[1/L]	Fitting Parameter for the air entry volume ≥ 0
g_n	[-]	Fitting parameter for the rate of water extraction from the soil after g_n is exceeded
g_c	[-]	Fitting parameter used in Plaxis
ϕ_p	[m^3]	Pressure head
p_w	[kN]	Suction pore stress
Υ_w	[kN/m^3]	Unit weight of water
k_{rel}	[-]	Relative permeability

Van Genuchten model

'A soil Water Characteristic Curve (SWCC) is introduced to describe the hydraulic parameters of the groundwater flow in unsaturated zones (usually above the phreatic surface).' (Plaxis, 2016)

The Van Genuchten equation provides reasonable results when suction levels are low to medium. Since the central aspects in this research are infiltration in extreme wet situations this equation is valid for no high suction levels are expected.

In which

$$Se = \frac{S - Sres}{Ssat - Sres}$$

$$Krel(S) = \max[(Se)^{gl}\left(1 - \left[1 - Se^{\frac{gn}{gn-1}}\right]^{\frac{gn-1}{gn}}\right)^2, 10^{-4}]$$

In this study the following default parameters are used in the hydraulic formula.

	Clay (Light clay O11)	Peat (Peaty layer O18)	Sand
k_x and k_y [m/s] **(permeability)**	$1{,}6*10^{-6}$	$4{,}0*10^{-7}$	$2{,}9*10^{-6}$
e **(void ratio)**	0,5	0,5	0,5
g_a **(fitting parameter)**	1,91	1,38	5,21
g_l **(fitting parameter)**	-1,38	-1,2	0
g_n **(fitting parameter)**	1,15	1,32	2,37

Table 39 Default hydraulic parameters three soil types; dataset Staring

Appendix: D Resulting phreatic surface

In this appendix the phreatic surface due to precipitation events with different return periods is plotted.

Sand

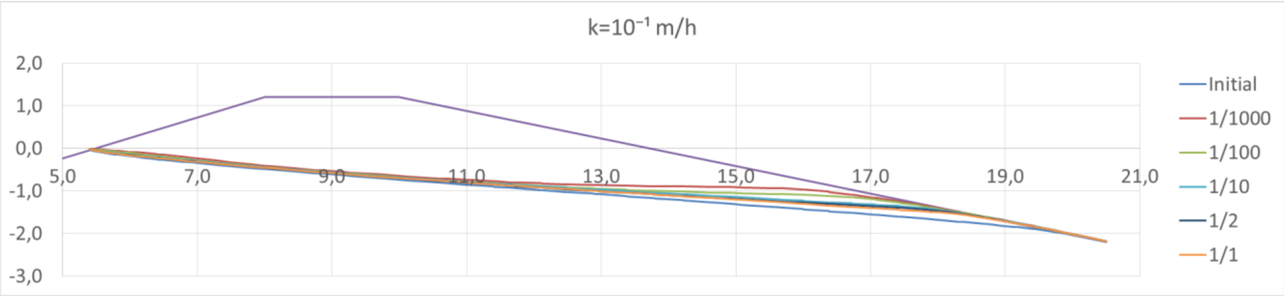


*Figure 81 Rise phreatic surface in sand k=2,8*10-⁵ m/s for different return periods*

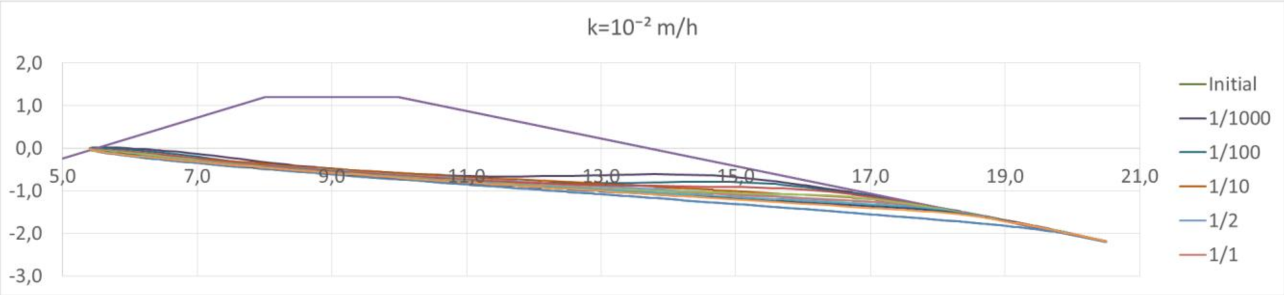


*Figure 80 Rise phreatic surface in sand k=2,8*10-⁶ m/s for different return periods*

*Figure 79 Rise phreatic surface in sand k=2,8*10-⁷ m/s for different return periods*

Peat

*Figure 85 Rise phreatic surface peat k=2,8*10-5 m/s for different return periods*

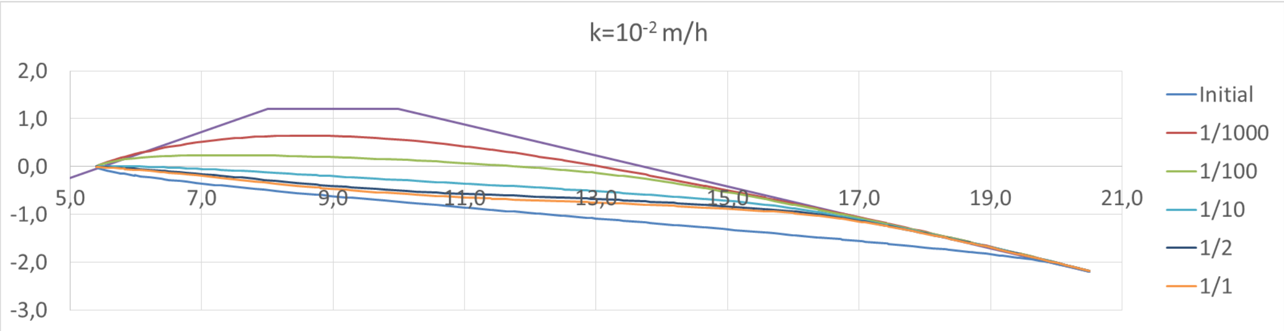


*Figure 84 Rise phreatic surface peat k=2,8*10-6 m/s for different return periods*

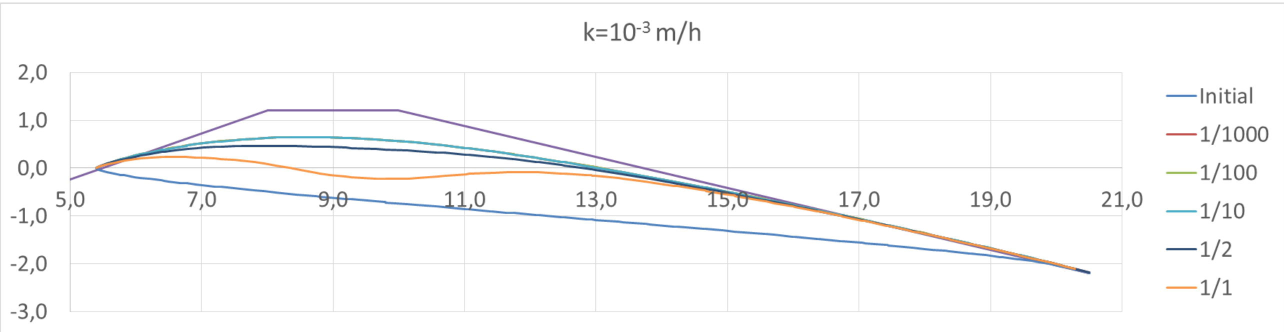


*Figure 83 Rise phreatic surface peat k=2,8*10-7 m/s for different return periods*

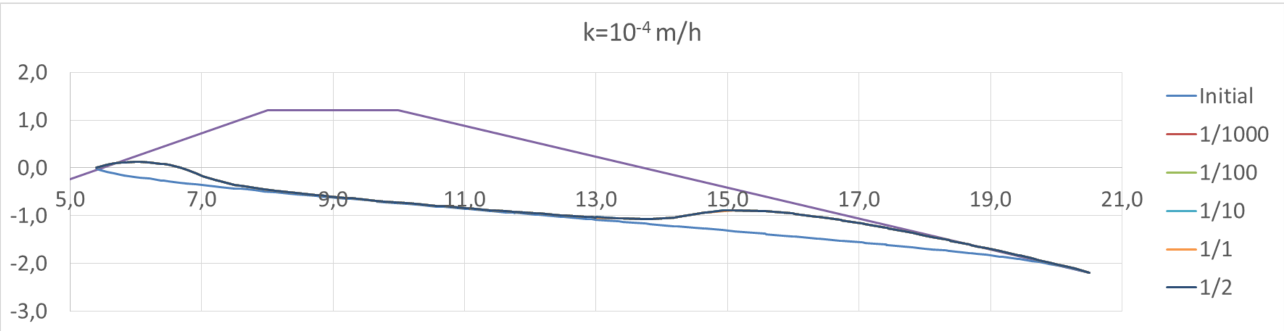


*Figure 82 Rise phreatic surface peat k= k=2,8*10-8 m/s for different return periods*

Clay

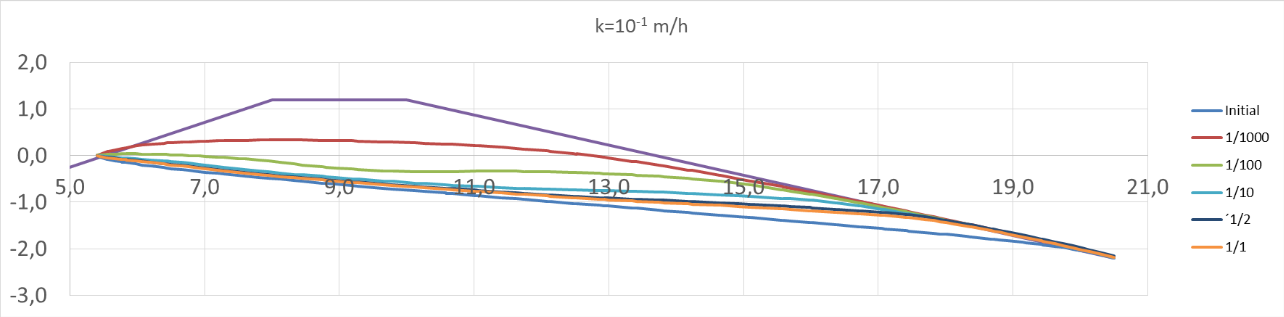


*Figure 89 Rise phreatic surface clay k=2,8*10^{-5} m/s for different return periods*

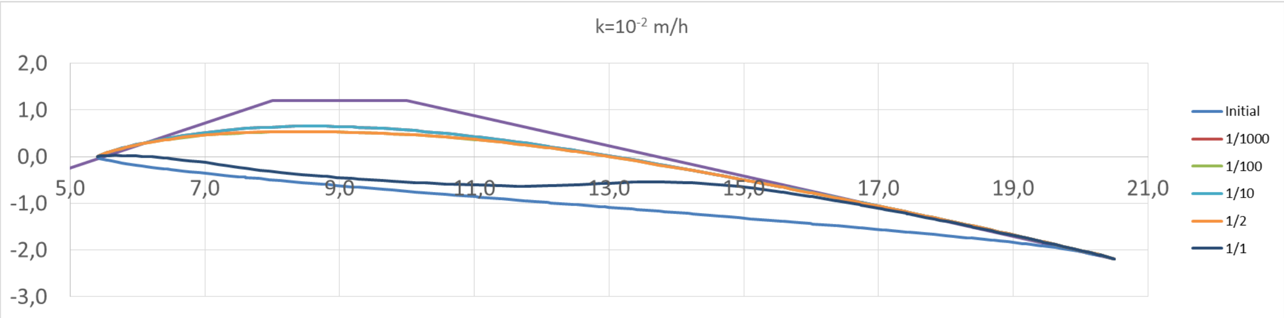


*Figure 88 Rise phreatic surface clay k=2,8*10^{-6} m/s for different return periods*

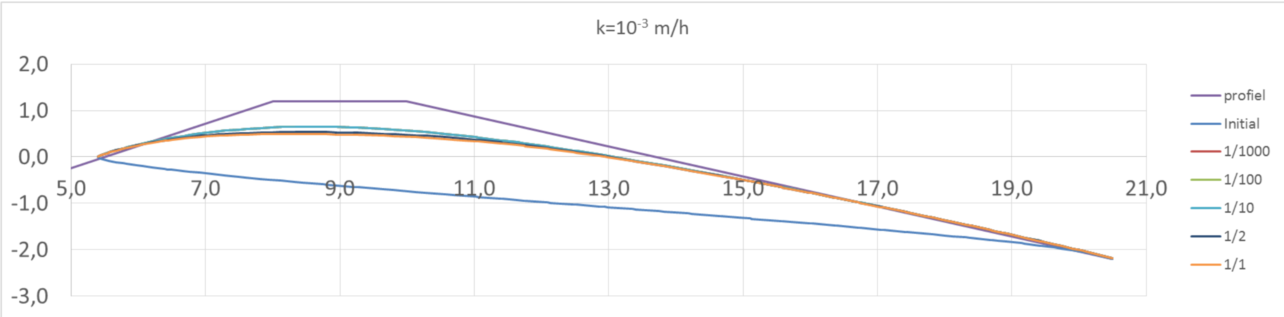


*Figure 87 Rise phreatic surface k=2,8*10-^{7} m/s for different return periods*

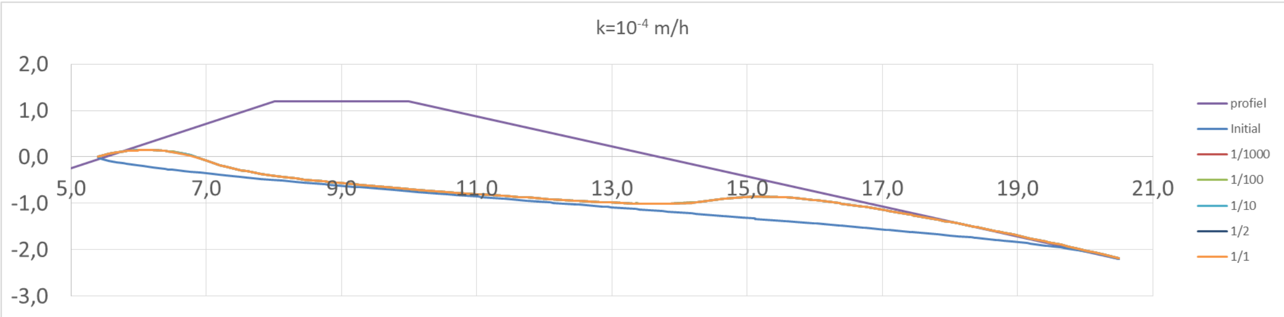


*Figure 86 Rise phreatic surface clay k=2,8*10-^{8} m/s for different return periods*

Appendix: E Sensitivity study

The computations are based on 1 single basic profile; in order to validate the sensitivity, a sensitivity study is performed on the following two parameters: relative crest height and width of the dike.

The results from the study are presented as exact value and as a percentage of the ´basic´ situation; the comparison with the basic situation enables the possibility to scale the phreatic surface when the dike has different dimensions than the basic dike; therefore the increase of the phreatic surface is divided by the length of the phreatic surface and compared to the basic situation. The sensitivity study is performed for all three soil types regarding a critical 1/100 year^{-1} precipitation event.

Sand; sensitivity crest width and water height

For a dike consisting of sand the width of the dike is important for the rise of the phreatic surface. Not only is the increase of the phreatic surface (per running meter) of the phreatic surface smaller for a wider crest but also the total increase is smaller.

Sand	k=2,8*10^{-5} m/s	k=2,8*10^{-6} m/s	k=2,8*10^{-7} m/s
Basic (crest width 2 m)	2,16	3,76	4,42
Crest width 4 m	2,12	3,76	4,40
Crest width 6 m	2,02	3,85	4,35

Table 40 Rise of the phreatic surface in m² depending on the width of the dike for a precipitation event with a return period of 100 years; dike of sand

Relative rise phreatic surface (sand)	k=2,8*10^{-5} m/s	k=2,8*10^{-6} m/s	k=2,8*10^{-7} m/s
Basic (crest width 2 m)	1,00	1,00	1,00
Crest width 4 m	0,87	0,89	0,88
Crest width 6 m	0,74	0,81	0,78

Table 41 Rise of the phreatic surface compared to the basic computation depending on the width of the dike for a precipitation event with a return period of 100 years; dike of sand

The relative crest height for sand gradations with a lower permeability <10^{-2} /h does not affect the rise of the phreatic surface significantly. The closer the phreatic surface is to the surface the larger the influence of the relative crest height will be.

Sensitivity relative crest height; sand	k=2,8*10^{-5} m/s	k=2,8*10^{-6} m/s	k=2,8*10^{-7} m/s
Relative crest height 0,9m	2,38	4,04	4,55
relative crest height 1,2m (basic)	2,02	3,85	4,35
Relative crest height 1,5m	2,01	3,50	4,30

Table 42 Rise of the phreatic surface in m² depending on the relative crest height of the dike for a precipitation event with a return period of 100 years; dike of sand

Relative sensitivity relative crest height; sand	k=2,8*10^{-5} m/s	k=2,8*10^{-6} m/s	k=2,8*10^{-7} m/s
Relative crest height 0,9m	1,07	1,04	1,00
relative crest height 1,2m (basic)	1,00	1,00	1,00
Relative crest height 1,5m	0,97	0,97	1,01

Table 43 Rise of the phreatic surface compared to the basic computation depending on the relative crest height of the dike for a precipitation event with a return period of 100 years; dike of sand

Peat

For a dike consisting of peat the results are very different than for a dike consisting of sand.

For:

- $K=10^{-1}$: the larger the dike is the lower the rise of the phreatic surface (both relative and absolute) The result is comparable with sand;
- $K=10^{-2}$: the rise of the phreatic surface decreases with growing width, however the decrease is less than with sand or with peat $k=10^{-1}$;
- $K=10^{-3}$: The relative rise of the phreatic surface grows with growing dike width; this is due to the fact the dike will be fully saturated after the precipitation event, a larger dike has a larger volume, therefore the absolute and relative rise is larger than in other cases.
- $K=10^{-4}$: For this value the relative rise of the phreatic surface is comparable with sand.

Peat	$k=2,8*10^{-5}$ m/s	$k=2,8*10^{-6}$ m/s	$k=2,8*10^{-7}$ m/s	$k=2,8*10^{-8}$ m/s
Basic (crest width 2 m)	4,29	9,48	12,02	2,54
Crest width 4 m	4,47	10,22	16,03	2,48
Crest width 6 m	4,52	10,87	19,93	2,42

Table 44 Rise of the phreatic surface in m² depending on the width of the dike for a precipitation event with a return period of 100 years; dike of peat

Relative rise phreatic surface (peat)	$k=2,8*10^{-5}$ m/s	$k=2,8*10^{-6}$ m/s	$k=2,8*10^{-7}$ m/s	$k=2,8*10^{-8}$ m/s
Basic (crest width 2 m)	1,00	1,00	1,00	1,00
Crest width 4 m	0,92	0,95	1,18	0,87
Crest width 6 m	0,84	0,91	1,32	0,76

Table 45 Rise of the phreatic surface compared to the basic computation depending on the width of the dike for a precipitation event with a return period of 100 years; dike of peat

Sensitivity rel. crest height; peat	$k=2,8*10^{-5}$ m/s	$k=2,8*10^{-6}$ m/s	$k=2,8*10^{-7}$ m/s	$k=2,8*10^{-8}$ m/s
Relative crest height 0,9m	4,814	9,80	9,86	2,68
relative crest height 1,2m (basic)	4,29	9,48	12,02	2,54
Relative crest height 1,5m	4,236	8,49	14,17	2,44

Table 46 Rise of the phreatic surface in m² depending on the relative crest height of the dike for a precipitation event with a return period of 100 years; dike of peat

Sensitivity relative crest height; peat	$k=2,8*10^{-5}$ m/s	$k=2,8*10^{-6}$ m/s	$k=2,8*10^{-7}$ m/s	$k=2,8*10^{-8}$ m/s
Relative crest height 0,9m	1,09	1,00	0,80	1,02
relative crest height 1,2m (basic)	1,00	1,00	1,00	1,00
Relative crest height 1,5m	1,00	0,93	1,22	1,00

Table 47 Rise of the phreatic surface compared to the basic computation depending on the relative crest height of the dike for a precipitation event with a return period of 100 years; dike of peat

Clay

In the phreatic surface rises less high when the crest width is larger; however if the critical situation approaches a saturated situation the phreatic surface must be scaled following the basic shape.

The relative crest height is important as well; the larger the relative crest height the smaller the rise of the phreatic surface; in the sensitivity study the difference can be as large as 10 %.

For clay with a permeability of 10^{-2} and 10^{-3} the scaling needs special attention. The dike is either saturated or almost saturated; therefore the relation between the crest width and the rise of the phreatic surface (negative relation) and the relation between the relative crest height and the rise of the phreatic surface (negative relation) does not hold.

Clay	k=10^{-1} m/h	k=10^{-2} m/h	k=10^{-3} m/h	k=10^{-4} m/h
Basic (crest width 2 m)	4,29	9,48	12,02	2,54
Crest width 4 m	6,33	14,33	16,22	3,07
Crest width 6 m	6,42	18,40	20,53	2,98

Table 48 Rise of the phreatic surface in m² depending on the width of the dike for a precipitation event with a return period of 100 years; dike of clay

Relative rise phreatic surface (clay)	k=10^{-1} m/h	k=10^{-2} m/h	k=10^{-3} m/h	k=10^{-4} m/h
Basic (crest width 2 m)	1,00	1,00	1,00	1,00
Crest width 4 m	0,86	1,09	1,19	0,91
Crest width 6 m	0,78	1,26	1,35	0,79

Table 49 Rise of the phreatic surface compared to the basic computation depending on the width of the dike for a precipitation event with a return period of 100 years; dike of clay

Sensitivity relative crest height; clay	k=10^{-1} m/h	k=10^{-2} m/h	k=10^{-3} m/h	k=10^{-4} m/h
Relative crest height 0,9m	7,45	8,770	9,92	3,25
relative crest height 1,2m (basic)	4,29	9,48	12,02	2,54
Relative crest height 1,5m	5,90	12,25	14,31	2,85

Table 50 Rise of the phreatic surface in m² depending on the relative crest height of the dike for a precipitation event with a return period of 100 years; dike of clay

Sensitivity relative crest height; clay	k=10^{-1} m/h	k=10^{-2} m/h	k=10^{-3} m/h	k=10^{-4} m/h
Relative crest height 0,9m	1,11	0,73	0,80	1,05
relative crest height 1,2m (basic)	1,00	1,00	1,00	1,00
Relative crest height 1,5m	0,94	1,10	1,23	0,98

Table 51 Rise of the phreatic surface compared to the basic computation depending on the relative crest height of the dike for a precipitation event with a return period of 100 years; dike of clay

Appendix: F Field study; effect of artificial precipitation on the phreatic surface in peat dikes

In-situ research

Data from an infield research can give valuable information on the type of behaviour one can expect from the phreatic surface. For this research the data gathered by the 'Hoogheemraadschap Hollands Noorderkwartier', further referred to as 'HHNK', is used to give information in the phreatic surface.

The research by the HHNK is conducted in 2012 and resulted in the report 'The effects of artificial precipitation on the phreatic surface in peat dikes' (Alterra, 2013). The target of this research was to gain insight in the size of the effects of precipitation on the pore pressures inside the dike.

In this chapter is described how the test was conducted, which parameters are measured and gives a frame work in order to be able to draw conclusions from the gathered data.

- How can the data gathered by the HHNK be used to gain insight in the behaviour of the phreatic surface influenced by extreme precipitation?
- What is the size of the effect on the phreatic surface which can be expected due to extreme precipitation on a peat dike?
- At what timespan does precipitation influence the pore pressures underneath the dike?
- Which special physical aspects have a role in the development of the pore pressures?'

Figure 90 location test site

10.1.2 Test lay out

The location of the embankments which are subjected to the test is near the city of Purmerend. The two dikes which are the subject of the study are called the 'Zuiderringdijk' on the north-west side, and the 'Oudelandsdijkje' on the south-east side of the drainage canal. Both dikes consist largely of clayey and peaty materials and is founded on a sand layer at roughly NAP -6 meters.

A land survey is conducted in order to schematize the geometrical characteristics. Cone penetration tests are performed in 2 section lines with 12 CPTs per line. At the locations of the cone penetration tests head monitoring wells are installed with an average depth of 4 to 6 meters below surface level to measure the effects of precipitation. The filters at the bottom of the well are 1 meter long and are situated in clay or peat; some of the filters are partially situated in the sand layer underneath the clay. Two of the head monitoring wells in the Zuiderringdijk are only 2 to 3m long; the reason for this is unknown.

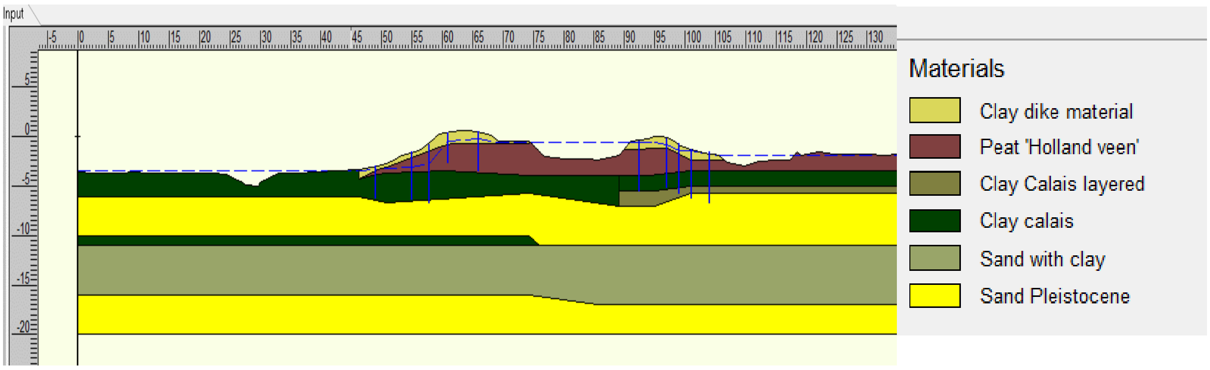


Figure 91 Schematization test site

10.1.3 Natural and artificial precipitation

The main test is a simulation of a precipitation event which has a probability of occurrence of 1/100 per year. Statistics on the precipitation intensity versus duration have been conducted by the STOWA. This includes information on the amount of precipitation in a certain time interval, which ranges from 4 hours to 9 days which all have a probability of occurrence of 1/100 per year.

In order to simulate these events the test site is prepared with pumps with spray systems. To ensure a proper execution of the test, the spraying system is tested before the actual test started. This testing results in a pre-wetted test site. In the test phase the system was calibrated to deliver the correct amount of precipitation on the desired area. The area in which the experiment was set up was 248m^2 for the Oudelandsdijkje and 488m^2 for the Zuiderringdijk.

The duration of the extreme event is chosen to be between 1 and 9 days. The wide spread of duration is chosen because it is not known which duration will have the most critical impact on the dike. The test is constructed in such a way that time and resources are used in a most efficient way. The following test procedure is applied: on day one the 1/100/year event of 1 day is imposed (79mm). In an extreme event of 2 days the amount of precipitation is 92mm. To simulate the 2-day event the precipitation of day 1 is used (79mm) and on the second day the difference is added as precipitation, so 92-79=13mm. With this approach there is implicitly chosen for an event with a high peak at the start and a long tail with low precipitation. For each successive day the same principle is applied.

The amounts of precipitation gifts per day are summarized in the Table 52.

Day	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**
1/100 event [mm]	79	92	102	109	118	125	131	137	143
Irrigation gift [mm]*	79	13	10	7	7	7	6	6	6

Table 52 Amount of artificial precipitation for each day during the test

The second part of the test is a simulation of an extreme precipitation event together with a high water level. For this test the same approach as mentioned above is used. Between the two tests, a six day period of rest is included in order to let the effects of the first test diminish.

Because the test site is relatively small, a third test is conducted to check if the 3D effects are negligible. In the third test a part of the control area is irrigated as well. This test is aborted due to the unexpected high response in the control area.

Because the spray system can get clogged the actual water gifts are monitored during the test and summarized in Table 53 and Figure 92.

Besides artificial precipitation also the natural precipitation influences the phreatic surface. During a part of the period the natural precipitation is measured. Furthermore information from a KNMI weather station in Purmerend is used to complete the information on natural precipitation. The hourly intensities correspond with the by Alterra measured precipitation intensities, disregarding small differences.

Date	**Irrigation Oudelandsdijkje [mm]**	**Natural precipitation [mm]**	**Notes**
4-10-2012	78,7	15,5	Initial test
5-10-2012	13,8	9,0	
6-10-2012	10,3	21,0	
7-10-2012	7,8	0	
8-10-2012	7,8	0	
9-10-2012	7,8	0	
10-10-2012	7,1	0	
11-10-2012	7,7	0	
12-10-2012	6,1	16,8	
13-10-2012	0,0	6,7	Rest period
14-10-2012	0,0	11,4	
15-10-2012	0,0	6,4	
16-10-2012	0,0	1	
17-10-2012	0,0	0,2	
18-10-2012	80,6	3,4	Water level in drainage canal raised with 0,3m
19-10-2012	13,2	10,2	
20-10-2012	10,3	1,2	
21-10-2012	6,9	2,4	
22-10-2012	7,5	0	
23-10-2012	7,5	0	

Table 53 Measured amount of artificial and natural precipitation during the test

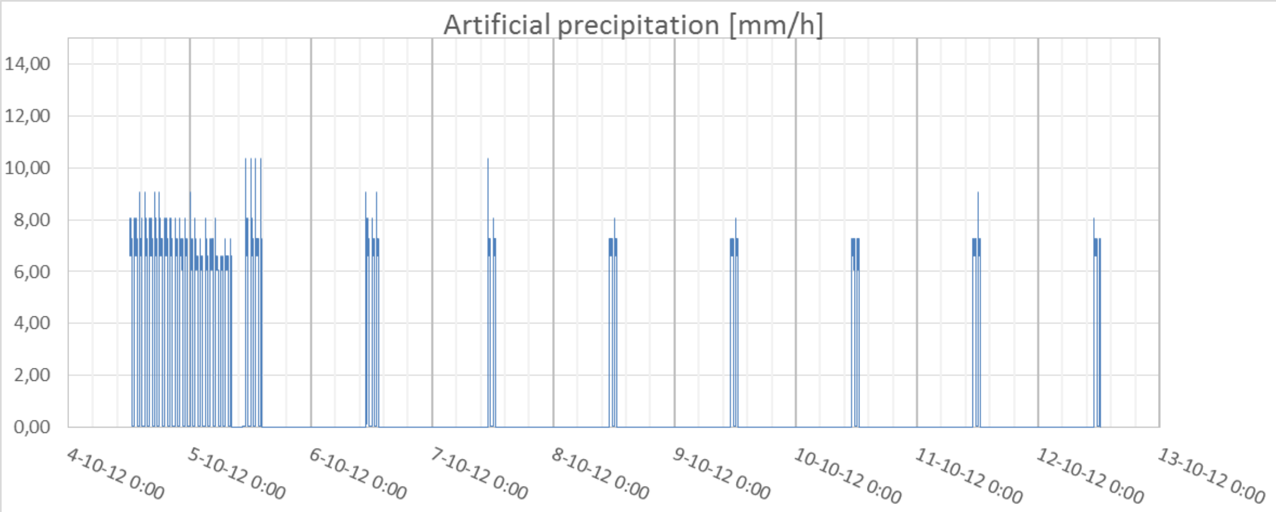


Figure 92 pattern artificial precipitation test Oudelandsdijkje

Analysis in-field tests

To gain insight in the reaction of the phreatic surface due to an extreme precipitation event, two field tests are conducted by the HHNK. Two dikes are equipped with a series of head monitoring wells, in 2 different cross sections per dike order to capture the fluctuation of the head in space and time. With the monitoring wells the head is measured during several weeks in which both natural and artificial precipitation influence the phreatic surface. The results are plotted in the next chapters and noteworthy aspects are listed. With the results from the test the following questions can be answered:

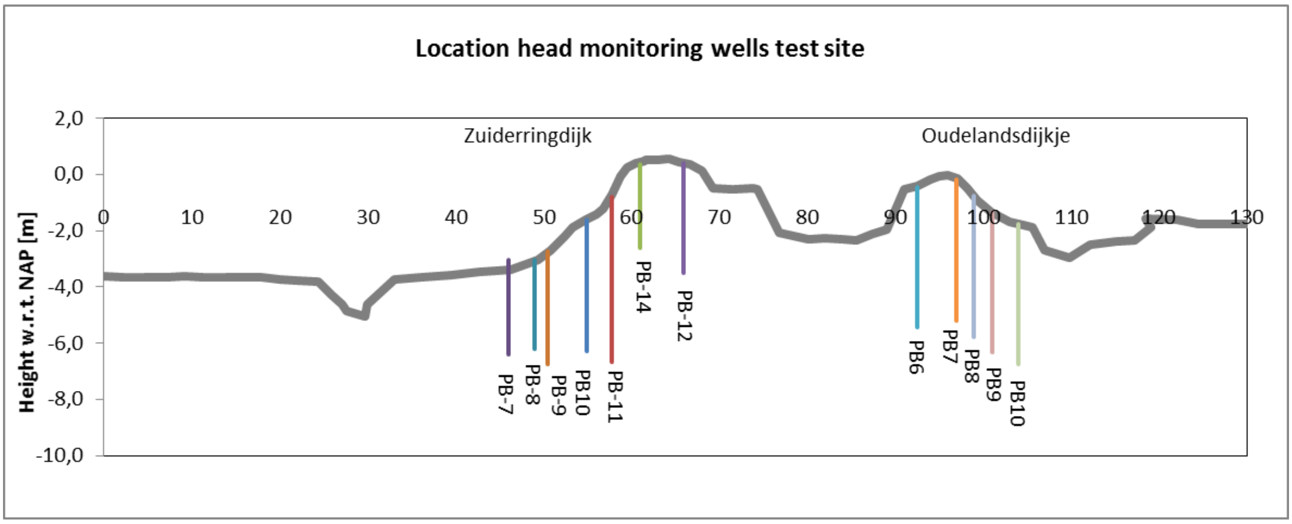


Figure 93 Location head monitoring wells test site

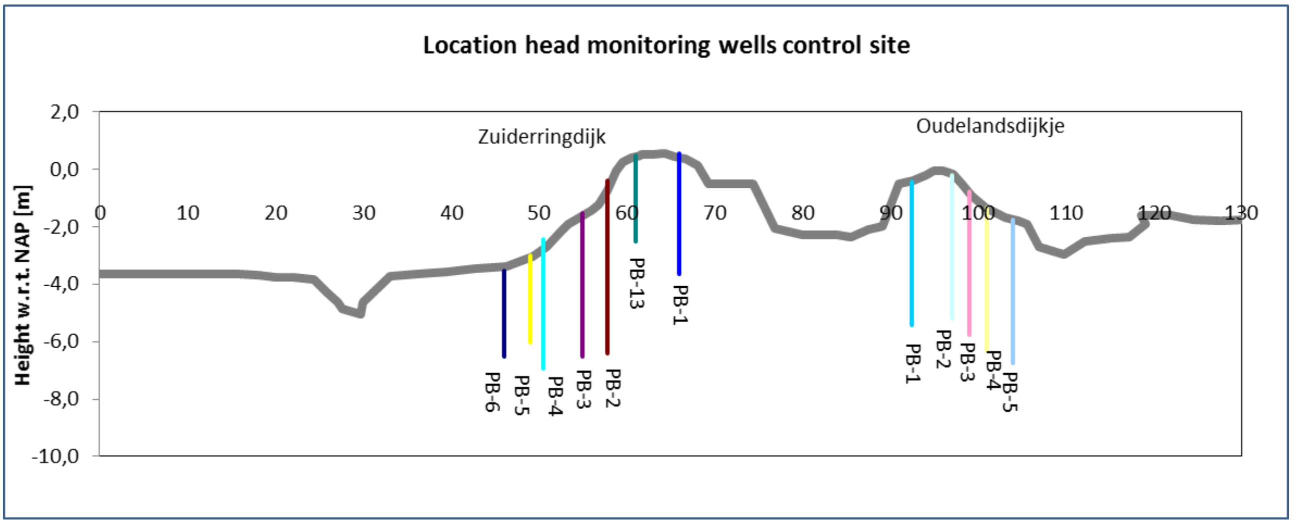
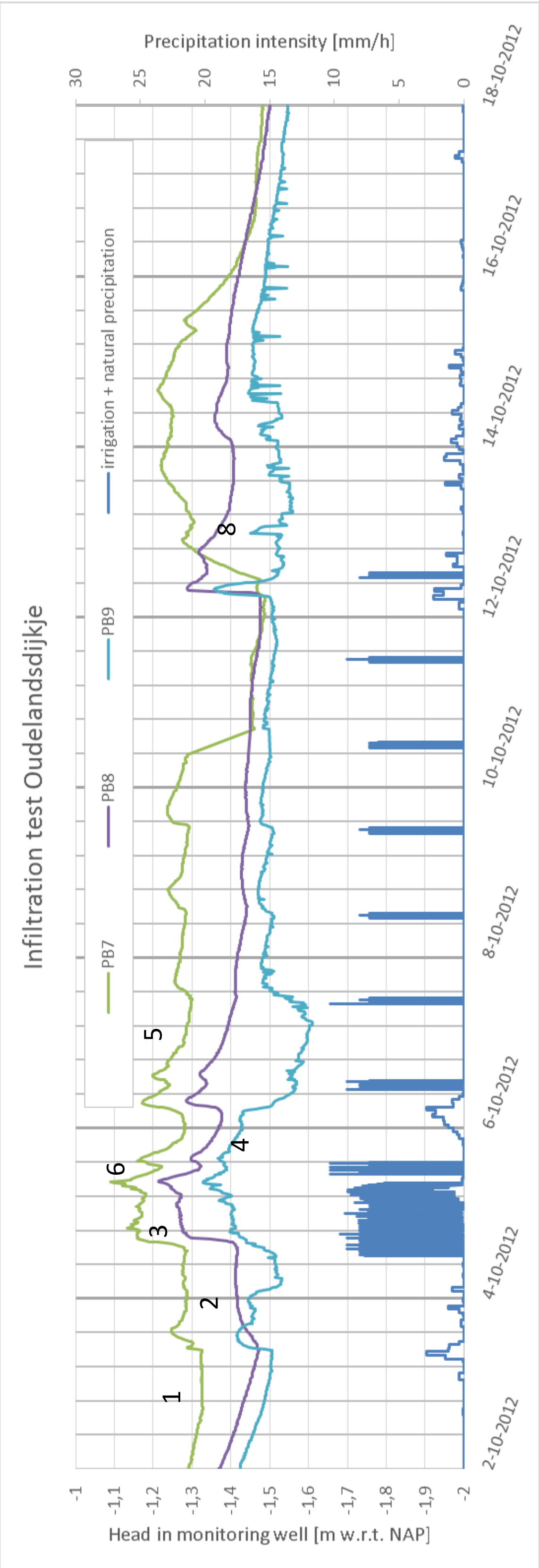
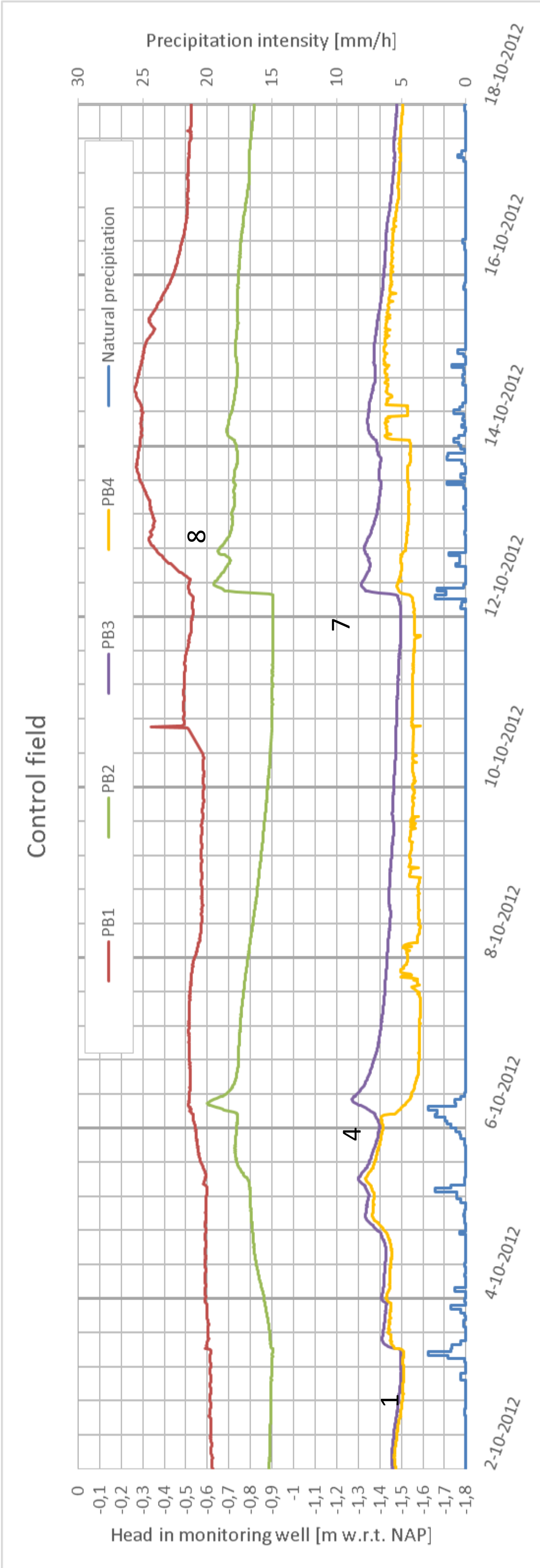


Figure 94 Location head monitoring wells control site



Infiltration test Oudelandsdijkje
Precipitation intensity [mm/h]
Head in monitoring well [m w.r.t. NAP]
PB7
PB8
PB9
irrigation + natural precipitation
2-10-2012
4-10-2012
6-10-2012
8-10-2012
10-10-2012
12-10-2012
14-10-2012
16-10-2012
18-10-2012



Control field
Precipitation intensity [mm/h]
Head in monitoring well [m w.r.t. NAP]
PB1
PB2
PB3
PB4
Natural precipitation
2-10-2012
4-10-2012
6-10-2012
8-10-2012
10-10-2012
12-10-2012
14-10-2012
16-10-2012
18-10-2012

In this graph the total precipitation intensity is plotted over time together with the measured heads in the monitoring wells.

For this specific dike the following aspects can be noticed:

1 Due to a natural precipitation event of 10,5mm the pore pressures in (some of the) monitoring wells rose with 0,1m.
Duration: 5hrs; averaged intensity: 2mm/h
2 The water level in PB-8 rises much slower than the rest; possibly due to lower hydraulic conductivity.
3 Due to the first artificial event, the pore pressures rose with ± 0,2m. The maximum water level reached is NAP-1,1.
PB-3 and PB-4 in the control field react on the artificial precipitation with a head rise of 0,1m.
Duration: 19hrs; averaged intensity: 4mm/h; maximum intensity 7 to 8mm/h
4 Due to a natural even of 15,9mm the pore pressures rose with ± 0,1mm in both the infiltration and the reference field.
An abnormality can be seen in the pore pressure measurements in the toe of the infiltration field (PB-9) and at the toe of the control field (PB-4), the pore pressures drop with no apparent reason. The head in PB-9 rises to almost its old level in 1 day. The head in PB-1 does not rise to its old level.
Duration: 15 hrs; averaged intensity: 1,06mm/h; maximum intensity 2,5mm/h
5 The small artificial events of 7 to 8 mm result in a small rise in pore pressures of 0,05m. During the rest of the day (21,5hrs) the pore pressures drop with 0,05m.
Duration: 1,5hrs; averaged intensity: 5,2mm/h; maximum intensity: 8mm/h
6 An abnormality can be seen in the pore pressure measurements in the crest of the infiltration field. With no apparent reason it drops with 0,2m and within two days it is back at the original level.
7 During a dry period of 5 days the pore pressures in the control site drop with 0,20m - 0,30m towards a stable level.
Duration: 5 days; averaged intensity: 0 mm/h
8 A natural event of 6mm causes a sharp rise in pore pressures in both the infiltration and the reference field of 0,1 to 0,3m. The effect occurs with a delay of ±4hrs from the start of the event. The head in PB-9 does reacts different than the rest, after a sharp rise it drops just as fast towards its old level.
Duration: 3 hrs; averaged intensity: 2,06 mm/h

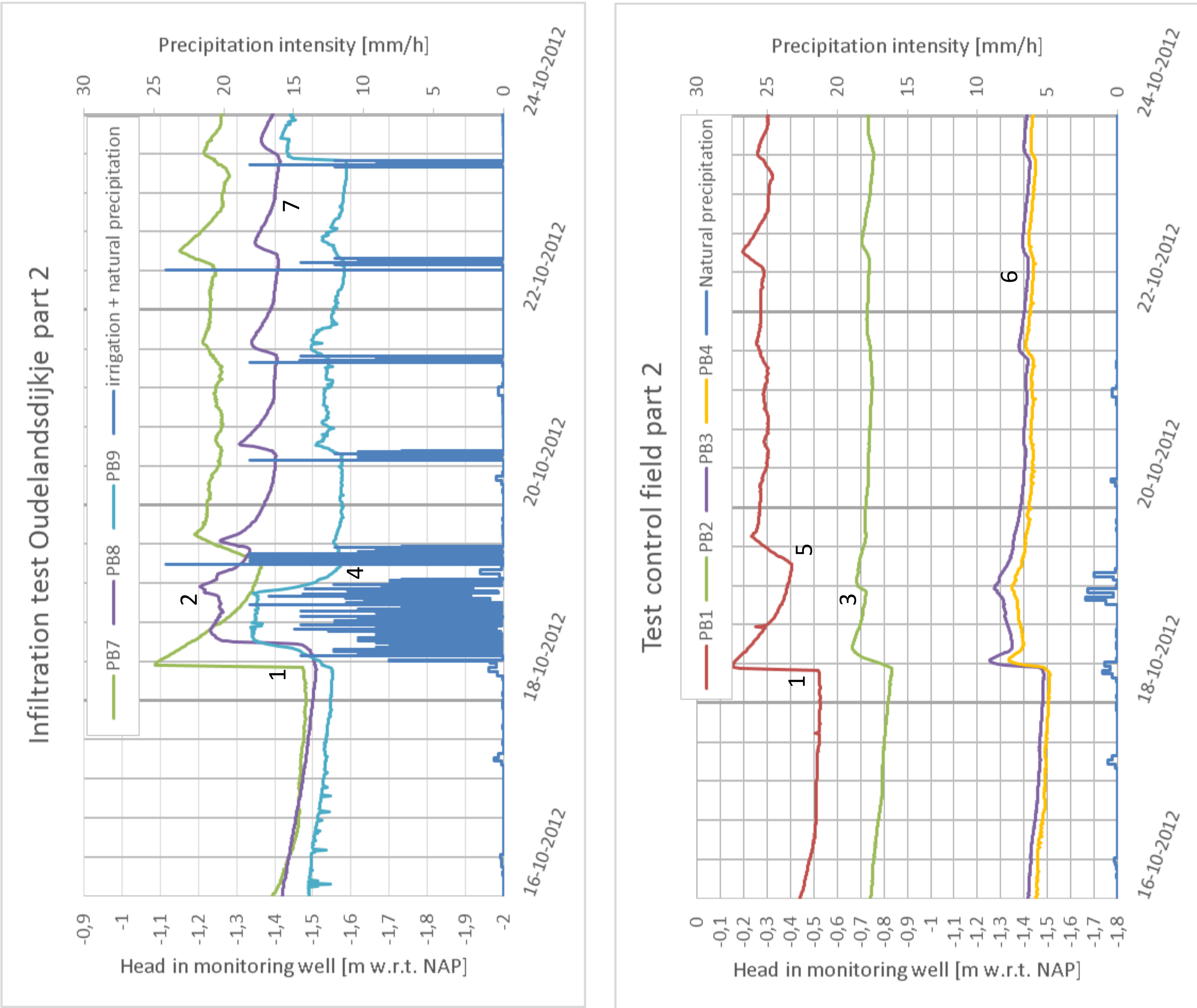


Figure 95 Response pore pressure Oudelandsdijkje part 2

Infiltration test Oudelandsdijkje Part 2: Increased water level (+0,3m)*

The exact time of the increase of water level is unknown

1. Due to natural precipitation (3,3mm), the start of the artificial precipitation and a rise of the boezem level of 0,3m the pore pressures increase sharply. The pore pressure rises with 0,2-0,3m
   Duration: 4 hrs; averaged intensity: 0,75mm/h; maximum intensity 1,05mm/h
2. The artificial precipitation event of 80mm causes a rise in pore pressures of a similar size as the effect test with the normal boezem level. (additional increase of 0,05m)
   Duration: 20 hrs; averaged intensity: 4 mm/h; maximum intensity 8mm/h
3. The pore pressures in the reference field decrease slowly towards 0,1m above the old level. In this result the heightened boezem level isn't clearly visible.
4. Two hours before the end of the artificial event the pore pressures in the toe starts to lower significantly.
5. The small precipitation events cause a rise of 0,1m underneath the landside slope; *Duration: 1,5 hrs; averaged intensity: 5,2 mm/h*
6. The effect of the artificial precipitation on the test area can be seen in PB-3 and PB-4 in the control area.
7. The reference field keeps a heightened water level for the rest of the test period due to the increased outside water level. The difference is 0,10m to 0,20m.

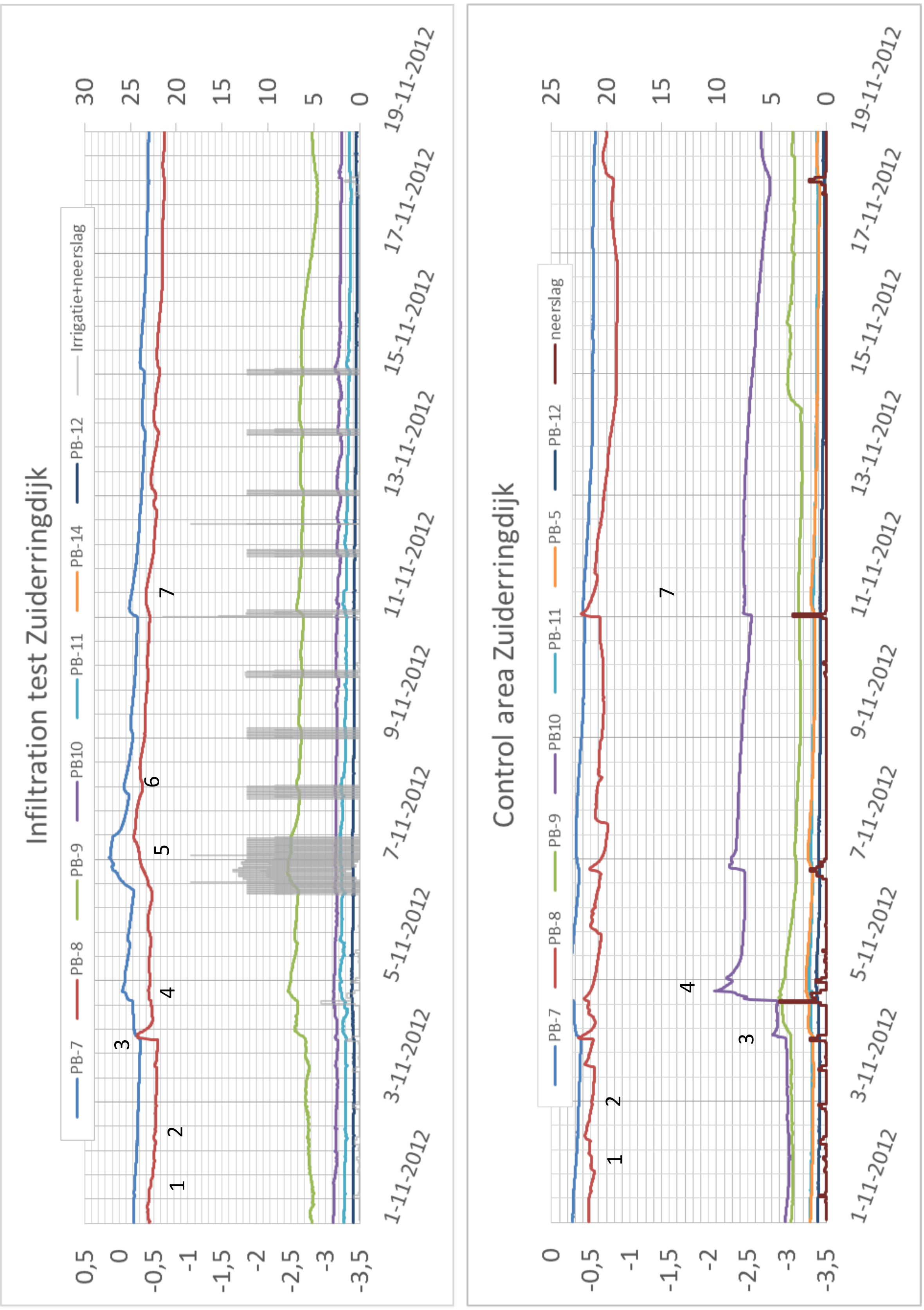


Figure 96 response pore pressure Zuiderringdijk

In this graph the total precipitation intensity is plotted over time, together with the head measurements in the monitoring wells in the Zuiderringdijk.

For this specific dike the following aspects can be noticed:

1 The response on natural precipitation in PB 1, 13, 12 and 14 is without delay. This is possibly due to the high placing of the filter.
2 A natural even with a low amount of precipitation, 8mm in 64 hrs, the pore pressures underneath the crest have decreased with 0,1m. The pore pressures underneath the landside slope have reached a stable level with one exception; PB 11 displayed a rise in pressure of 0,1m.
Duration: 64 hrs; averaged intensity: 0,125mm/h; maximum intensity 0,80mm/h
3 A natural even of 9mm causes a sharp rise in pore pressures in both the crest of the test site as the control field. Furthermore PB 2 in the control field reacts strongly with an increase in pore pressure of 0,5m to 0,7m.
Duration: 6 hrs; averaged intensity: 1,5mm/h; maximum intensity 4,2mm/h
4 After the natural even the increased pressure drops with about 0,1m in 28hrs in PB-1, 13, 12 and 11. In PB-2 and PB-14 a faster drop in pressures is monitored.
5 The first and largest artificial precipitation event of 81,5mm causes a rise of 0,3m in the crest of the test site. The response is more gradual than with the natural events the days before. The reaction in PB-11 in the upper side of the slope is 0,15m and the other wells do not display a change in pressure.
The natural precipitation during this artificial event causes no extra changes in pore pressures. The control area has no signs of reaction on the artificial precipitation on the area next to it.
Duration: 22 hrs; averaged intensity: 3,7mm/h; maximum intensity 9 mm/h
6 The following small precipitation events of 13mm to 6mm result in a small rise in pore pressures underneath the crest (PB12, 14). The dry period in between is too long to keep the pore pressures at a high level. After each following event the pore pressures drop to a lower level.
Duration: 4 hrs; averaged intensity: 3,25 mm/h; maximum intensity 8 mm/h.
7 The natural event on the 11th of November causes a rise in pore pressures in several wells of 0,1m to 0,2m.
Duration: 3 hrs; averaged intensity: 1,17 mm/h; maximum intensity 3,1 mm/h