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The effects of anisotropy and heterogeneity
in the piping sensitive layer

master thesis

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By

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Preface

This thesis completes the master Hydraulic Engineering at Delft University of Technology. The research has been carried out in collaboration with HKV lijn in water.

It was great to work at HKV both in Delft and in Lelystad. It was a motivational and inspiring environment, where interesting research is done and the employees are really involved with their field of work. Besides the personal conversation, I really enjoyed the lunch sessions. I have learned a lot from all colleagues. I would thank my colleagues for sharing their knowledge and fun stories. I would especially like to thank HKV for giving me this great graduation opportunity.

I would like to thank my entire thesis committee, Matthijs Kok, Wouter ter Horst, Bram van den Eijnden, Wim Kanning, Jan Jaap Heerema for their guidance and valuable feedback during this research. Matthijs, thank you for your optimism, but above all for keeping the rest of the committee members in line during the committee meetings. Bram, thanks for your critical view on my thesis structure, writing and results, this has definitely improved my thesis. Wim, thanks for sharing your knowledge, even when you was abroad I could always reach you for questions via email or Skype. In addition, you always adequately helped when I had some D-Geo Flow issues. Your passion for piping inspired me and kept me motivated. Jan Jaap, it was an honour that after my intership at Rijkwaterstaat, you also supervised me during my master thesis. Not only your theoretical knowledge, but especially your practical knowledge about piping had increased the quality of the debates during the committee meeting and thereby improved my thesis. Specifically, I would like to thank Wouter for being my personal supervisor. We had lots of interesting and valuable conversations. Thanks to your supervision, I could carry out my thesis within a foreseeable period of time.

Besides my committee, I would like to thank my fellow students at HKV, in particular Mick van Montfoort and Pieter Verhey for the funny, personally, helpful conversations during coffee and lunch breaks, but also for listening to my thesis issues and for being very good sparring partners.

Furthermore, I would like to thank my close family and friends. My family always supported, helped and believed (in) me during my entire study time in Delft. I want to thank my friends for these pleasant years. Together we have experienced beautiful, fun and memorable events. Besides I could also count on you in tough times. Specially, thanks to the friends which I have bothered for reading my report, you really improved my writing. Finally, I would like to thank my amazing girlfriend. She always gives me power, love and happiness.

Nick Stoop
Delft, December 2017

Summary

This thesis outlines improvements for the piping assessment of levees by investigating the effects of anisotropy and heterogeneity in the piping sensitive layer. Levees are constructed along bodies of water such as rivers , seas and lakes to protect land from flooding. A levee is commonly composed of impervious soils and built on sandy foundations. These sandy foundations are sensitive to an erosion process that is known as piping. Piping refers to the development of shallow channels in the top of the sandy foundation underneath the dike body and is the main topic of this thesis. The erosion process can only operate if the pressure underneath the inland cover layer exceeds the weight of the cover layer and thus results in a hydraulic fracture. Due to the presence of the fracture, sand particles are washed out by the outflowing water. A pipe starts to develop at the hydraulic fracture and grows backwards to the water body. This process tends to control itself until the hydraulic head exceeds the critical level. The erosion process becomes progressive so that the structure will eventually collapse.

The phenomenon of piping has been intensively investigated over the last 100 years. Several simple empirical rules for the design of a standard dike were created with minor theoretical background. In addition, soil properties are not involved in these empirical design rules. A more tangible design rule was therefore obtained by J. Sellmeijer (2006) by curve fitting specific results of a conceptual model. This rule includes soil properties of the foundation layer.

Although this rule seems to be fine for homogeneous and uniform subsoils conditions, it is not explicitly derived for heterogeneous and anisotropic conditions, especially stratified aquifers. The question is therefore: *whether heterogeneous and/or anisotropic conditions will lead to an overestimation or an underestimation of the piping resistance of an aquifer.* In the case of heterogeneous and/or anisotropic aquifer, a numerical model is required to assess the piping resistance of a levee. The recently developed model D-Geo Flow (Deltares, version 1.0.37967) was therefore used to analyse the effects of heterogeneity and anisotropy. The D-Geo Flow model consists of three computations: the groundwater flow in layers, the water flow in the erosion channel and the state of limit equilibrium of the particle at the bottom of the channel. The disadvantage of a numerical model is that the computations can be time-consuming, especially when the aquifer composition consists of very thin elements and/or many (different) elements. In addition, the computations require a lot of preparation, while the consultancy prefers analytical approaches with for instance design rules.

In order to analyse the effects of anisotropy and heterogeneity such as stratification, resistances and conductors, two model studies are performed; an elementary study and a realistic study. In the elementary study, heterogeneous or anisotropic aquifer compositions were constructed by adding artificial elements with different physical properties to the body or by varying physical properties of the body depending on the direction. The realistic study is a more realistic and complex version of the elementary study since stratification has also been implemented. In that way, more realistic cases were modelled. After each simulation, the critical head computed in the elementary and realistic study were first compared to the critical head computed in accordance with Sellmeijer's approach. As a result, the over- or underestimation of the piping resistance regarding Sellmeijer's design rules becomes clear.

Moreover, an alternative method to determine the representative bulk permeability was derived in order to improve the assessment of anisotropic and/or heterogeneous subsoils with Sellmeijer's formula. The alternative method assumes that the groundwater flows curved or radially towards the exit or head of the pipe instead of only horizontally. For that reason, the bulk permeability is calculated as the geometric or logarithmic mean of both the horizontal and vertical permeability. To verify whether this alternative method improves the analytical approach with Sellmeijer's formula, the critical head computed in the elementary and realistic study were also compared to critical head computed with the analytical approach with an alternative bulk permeability.

Based on both studies it can be concluded that the currently used analytical or one layer approach in Sellmeijer's formula, with only one exception, leads to great underestimations of the piping resistance of aquifers. In the case of a stratified and anisotropic aquifer, the underestimation is even 45%, which means that this aquifer composition can resist a 45% higher water level. In addition, both studies show that if the alternative method to determine the representative bulk permeability is used in the analytical approach, the analytical approach, with only one exception, approaches the numerical outcome of heterogeneous and/or anisotropic aquifer considerably more accurate. However, both above mentioned exceptions arise when a gravel element is completely located at the front of the critical pipe length. In that case, the currently used analytical approach is considerably more accurate and almost approaches the numerical outcome.

In conclusion, the current analytical approach with Sellmeijer's formula, with only one exception, greatly underestimates the piping resistance of a heterogeneous aquifer. This underestimation mainly occurs because the vertical groundwater flow is excluded. To improve the current assessment, the vertical permeability of soils should also be determined, simulated and calculated. In that way, the effects of anisotropy and heterogeneity in the aquifer can be included in the safety assessment.

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symbol	**description**	**unit**
A	cross-section area of the column	m
C	Hazen empirical constant	$(\mathrm{cm} \cdot \mathrm{s})^{-1}$
C_{creep}	creep ratio or percolation coefficient (Bligh)	-
$C_{w,creep}$	weighted creep ratio (Lane)	-
D	aquifer depth	m
D_i	thickness of a layer	m
d_{10}	grain diameter for which 10% soil passing through a particular sieve	m
d_{50}	grain diameter for which 50% soil passing through a particular sieve	m
d_{70}	grain diameter for which 70% soil passing through a particular sieve	m
d_{70m}	mean d_{70} in the small scale tests ($2.08{\cdot}10^{-4}$)	m
d_b	thickness of blanket (or cover layer)	m
F_G	geometrical shape factor	-
F_R	resistance factor	-
F_S	scale factor	-
g	gravitational acceleration (9.81)	$\mathrm{m/s^2}$
g_j	gravitation acceleration in j-direction	$\mathrm{m/s^2}$
h or H	hydraulic head	m
h_a	height of the aquifer	m
h_b	height of the blanket ($=d_b$)	m
h_l	height of the levee	m
h_p	free water level or ground level	m
i_c	critical heave gradient	-
i_o	occurring vertical gradient	-
K	uniform hydraulic permeability	m/day (or m/s)
K_h or K_x	horizontal (bulk) hydraulic permeability	m/day (or m/s)
K_v or K_y	vertical (bulk) hydraulic permeability	m/day (or m/s)
K_r	the bulk permeability for radial of curved flow	m/day (or m/s)
K_s	hydraulic conductivity in the direction of s	m/day (or m/s)
k_r	relative permeability	-
L	length (or seepage length)	m
L_h	horizontal seepage length	m
L_v	vertical seepage length	m
l_a	length of the aquifer	m
l_b	length of the blanket	m
l_e	length of the exit	m
$l_{l_{bottom}}$	bottom length of the levee	m
$l_{l_{crest}}$	crest length of the levee	m
n	porosity	-
p	pore pressure	$\mathrm{m^2/N}$
Q	discharge	$\mathrm{m^3/s}$
Q_s	discharge in the direction of s	$\mathrm{m^3/s}$
q_s or q_i	specific discharge	m/s
RD	relative density	-
RD_m	mean relative density in the small scale tests (0.725)	-
S	degree of saturation of the liquid phase in the void space	-

s	distance	m
T	transmissivity	m^2/day
t	time	s
α	compressibility of the soil skeleton	m^2/N
β	compressibility of the pore water	m^2/N
γ_p'	unit weight of particles	kN/m^3
γ_s	unit weight of saturated soil	kN/m^3
γ_w	the unit weight of water	kN/m^3
ΔH	hydraulic gradient or head difference	m
ΔH_c	critical hydraulic gradient	m
η	White's constant	-
θ	bedding angle	$^\circ$
κ	intrinsic permeability of the aquifer	m^2
κ_{ij}	intrinsic permeability	m^2
λ, ν	Lame's constant	N/m^2
μ_l	the dynamic viscosity of liquid	kg/(m·s)
μ_w	viscosity of water	$(N \cdot s)/m^2$
ρ_l	density of liquid	kg/m^3
ρ_s	density of submerged particles	kg/m^3
ρ_w	density of water	kg/m^3
ϕ	slope of the channel	$^\circ$
ϕ_o	hydraulic head underneath the blanket	m
Ω	flow domain	m^2

1 | Introduction

1.1 Research introduction

In this thesis, the effects of anisotropy and heterogeneity are investigated in order to improve the assessment and the design of a levee against piping. Levees are often constructed along rivers, artificial waterways, lakes and coasts to protect land from flooding. They are predominantly earthen structures composed of a sand body with an impermeable cover layer and built on sandy subsoils. The sandy subsoils are vulnerable to an internal erosion process that is known as piping. Piping refers to the progression of shallow pipes in the sandy subsoil underneath the body of a levee. This progress can only initiate if the difference between the outer and inner water level is of significance so that the pressure underneath the cover layer exceeds the weight of the inland cover layer. As a consequence, a hydraulic fracture arises after which sand particles can be washed out by the flowing water. A pipe starts to grow backwards to the water body. This process tends to stop automatically until the hydraulic head is above the critical level. The erosion process becomes progressive so that an open connection occurs and the structure will eventually collapse. Over the last 100 years several rules are formulated for assessing and designing levees against piping. This thesis focusses on the design rules of J. Sellmeijer (2006).

1.2 Project context

As a result of new insights, the design rule of Bligh is expired for the safety assessment of piping and is replaced by the design rules of J. B. Sellmeijer (1988) which were adopted in 2011. These design rules are based on fundamental and theoretical principles that describe the mechanics of the equilibrium of the soil particles and the mechanics of grain transport. Sellmeijer assumes in the design rules a standard geometry with an idealized homogeneous soil composition for calculating the seepage length. However, in practice the subsoil is highly variable on scale of metres over depth, width and length, resulting in substantial variations of for instance the intrinsic 'bulk' permeability(k) and grain size(d_{70}). In recent research it is discovered that these two key parameters are correlated and highly influencial to the seepage length which is required to obtain a safe flood defence

In January 2017 the Dutch government introduced new standards to assess the safety of the primary flood defences. The assessment of the flood defences includes the Sellmeijer's formula for piping. An exploratory assesment performed by Koopmans and de Visser (2016) showed that a dike section is assessed as very insufficient with the new design formulas. Even if the assessment is executed with the occurred high water levels of 1995 instead of the leading high water levels. However, during the high water of 1995 the dike sections did not show clear signs of potential failure (Koopmans & de Visser, 2016). In addition, researchers state that likelihood of piping is less, if there is a heterogeneous subsoil with for instance natural filters (Robbins & van Beek, 2015). Leuvenink et al. (2017) states more specifically that the calculated seepage length for the surveyed dike section was reduced with 50% by using better information of the permeability with respect to a calculation performed with the standard values for the soil profile and permeability (on basis of REGIS). This reduction is the consequence of counting in of the stratification and permeability variations in the aquifer.

The determination of the intrinsic permeability of the entire aquifer proves to be quite difficult in practice. Various methods exist to determine the permeability or hydraulic conductivity ranging from empirical equations, small scale and relative simple laboratory methods, like pumping test, to large scale and more advanced field test like, falling-head test and piezometer test (Förster et al., 2012). Due to these variations, the outcomes can differ significantly. Furthermore, some methods come up with a single value, so the effect of spatial variabilities and anisotropy is invisible or absent. As a result a conservative homogeneous assumption of the K-value (high representative value) should still be used in the Sellmeijer's formula. Consequently used in the current safety assessment.

1.3 Research objective

The research objective is to improve the assessment of the bulk permeability of the piping sensitive aquifer, used in Sellmeijer's formula, by modelling and analysing the effects of anisotropy and heterogeneity in the subsoil.

1.4 Research questions

The research objective will be achieved by answering the following research question:

- What kind of spatial variabilities are present in the subsoil in practice?
- What are the effects of anisotropy and heterogeneity, in particular variations of hydraulic conductivity, in the aquifer on the critical hydraulic head or gradient?
- What are the effects of anisotropy and heterogeneity, in particular variations of hydraulic conductivity, in the aquifer on the hydraulic head underneath the cover layer at the presumed exit point?
- How can heterogeneity and anisotropy in the subsoil be included in the safety assessment of piping, especially in the bulk permeability used in Sellmeijer's formula?

1.5 Definitions

The title of the report 'the effects of anisotropy and heterogeneity in the piping sensitive layer' includes two crucial definitions; anisotropy and heterogeneity. These two definitions are explicitly described in this section, since they are sometimes confused with each other. Figure 1.1 shows an illustrations of anisotropy and heterogeneity.

- **Heterogeneity**
  In general, heterogeneity describes difference in physical properties between two or more points or elements (Ikelle & Amundsen, 2005). In this thesis, heterogeneity describes variations of physical properties between two or more elements or layers. Hence, a heterogeneous subsoil consists of at least two elements or layers with different physical properties. Furthermore, if a heterogeneity is added in this thesis than one or more soil element(s) or layer(s) are added with different properties. This does not necessary result in the addition of different soil materials. However, the soil material must at least have different physical properties.

- **Anistropy**
  In general, a material, element or body is anisotropic when the physical properties are directional-dependent, which implies different physical properties with direction. The physical properties should therefore differ with direction at a given point (note that a point can represents a particle, element or body) (Ikelle & Amundsen, 2005). In this thesis, anisotropy describes difference in the physical properties, especially the permeability, with direction of a given soil element or layer. To put it simply, an anisotropic soil body, element or layer is direction-dependent, so the vertical property is unequal to the horizontal property. On the contrary, if the physical properties are similar with direction than the soil body, element or layer is **isotropic** or direction-independent. In addition, anisotropy on a large scale(m) is commonly the result of heterogeneity on a small scale(cm or mm). For example, tidal deposits often consist of predominately fine sandy soils

with thin and small clay layers. By assigning anisotropy to elements or layers, heterogeneity on a small scale can also be studied and examined with for instance numerical models. Furthermore, anisotropy only applies to direction-dependent properties such as permeability, but do not apply to density or grain size.

It can be concluded that anisotropy and heterogeneity coexist. In addition, heterogeneous and anisotropic subsoils are commonly observed in practice (Ikelle & Amundsen, 2005).

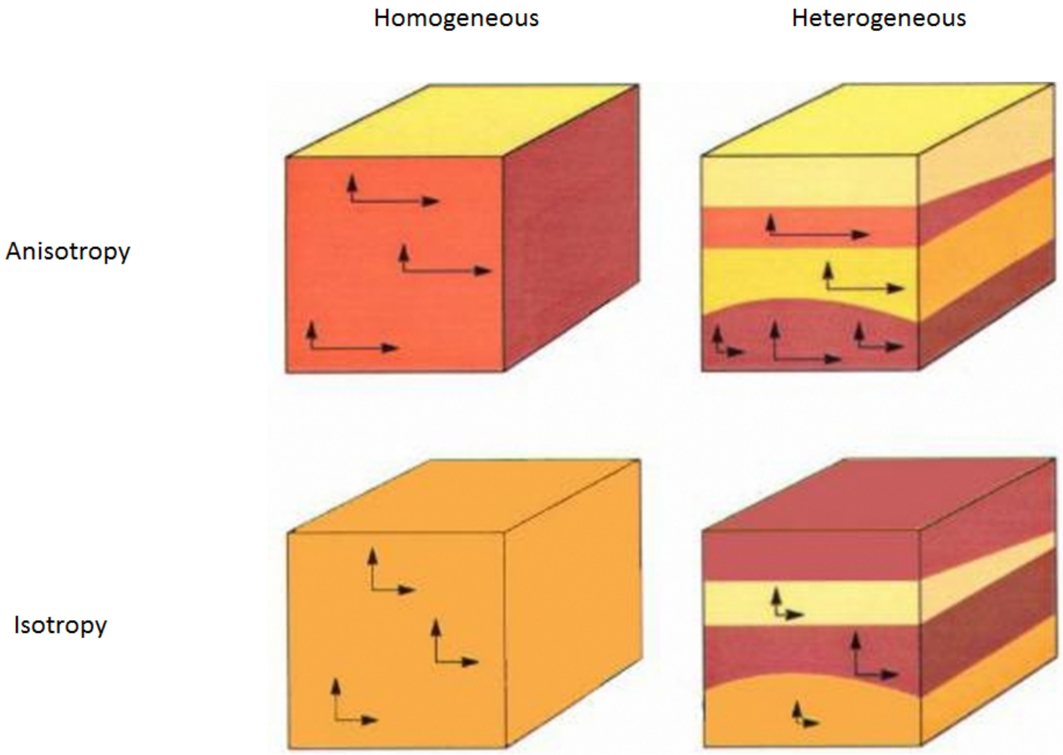


Figure 1.1: An illustration of anisotropy and heterogeneity (Ikelle & Amundsen, 2005)

1.6 Methodology

This thesis is an elementary modelling study to investigate the influence of heterogeneity in the aquifer on the safety of the levee, focusing on anisotropic behaviour, stratification ,and natural hydraulic resistances and conductors. Before building an elementary piping model, knowledge needs to be amassed from literature. The literature study provides insights into the mechanism piping, the parameters that influences piping and, the determination and variability of the hydraulic conductivity. To asses and discuss the effect of heterogeneity several steps are followed.

The second step is to set-up a D-Geo flow model with a standard configuration of levee described by J. Sellmeijer (2006) to verify the critical hydraulic head computed by the model with the Sellmeijer's design rules. Furthermore, the sensibility of the assumptions such as the boundary conditions, parameter and geometry choices are observed. After the sensitivity analysis, the initial model is adjusted, to modify the composition of the aquifer. This adjusted model is referred to as the elementary model and has an identical geometry and boundary conditions.

In order to create sensible compositions of the aquifer, several soil types are deduced on basis of the literature. The soil types range from very fine to very coarse. In general it is supposed that finer soils resist or hinder water, whereas coarser soils conduct water. Furthermore for defining the compositions of aquifer, physical properties of elements of layers are changed. These changes in physical properties of soil commonly arise from geomorphological process such sedimentation. Eventually, several compositions are defined, simulated and evaluated, whereby a distinction has been made between the sub-mechanisms pipe development, and uplift and heave. After each simulation is decided; to maintain the composition, to adjust the composition or to establish a new composition. The results of the elementary model are fundamental for the establishment of the complex and realistic model.

The elementary model is modified to a more realistic model by implementing stratified aquifer compositions. These compositions represent typical Dutch subsoil, but they are not extracted from specific Dutch sites. In order to establish these representative aquifers, Dutch specialist and literature data are consulted. Besides stratification, the realistic compositions also contain one additional heterogeneous property or anisotropy. The choice of the additional property is mainly based on the results of the elementary study and conversations with specialist. After the establishment of the realistic aquifers, the results of realistic aquifers are compared with the result of a representative homogeneous aquifer. In that way, the model provides insights into the underestimation or overestimations of the current assess and design rules. Based on these result, it will be discussed how the design rules can be improved so that anisotropy and heterogeneity addition, the process causing the underestimation or overestimation are revealed. Within the realistic study, a distinction has also been made between pipe development, and uplift and heave.

The results of the elementary and realistic study are explained in the discussion. The discussion also outlines the limitations of this thesis. After the discussion, conclusions and recommendations are drawn. Figure 1.2 shows the schematical overview of the methodology.

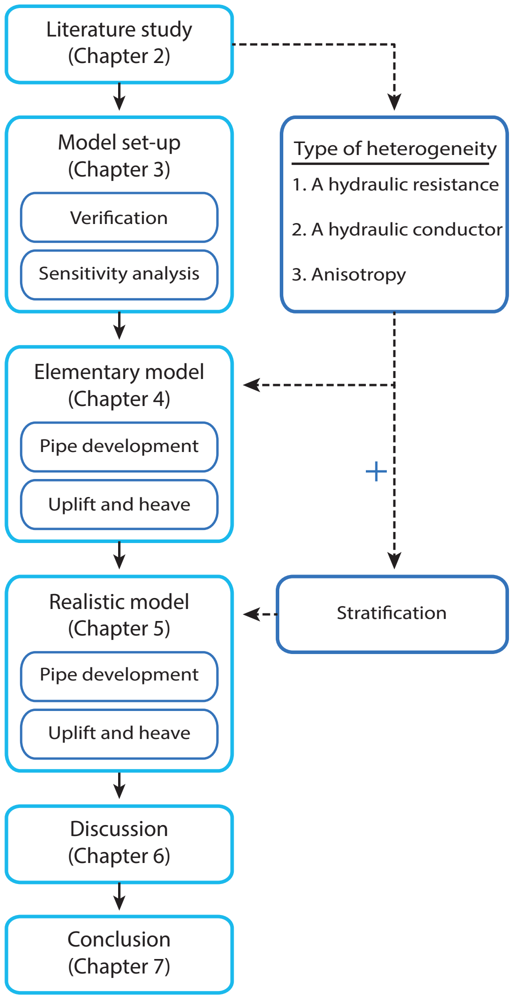


Figure 1.2: Schematized methodology

2 | Background and literature study

2.1 Introduction

This chapter summarizes the preliminary research and literature to underwrite the thesis problem. The literature study starts with a theoretical and practical background of piping processes followed by a historical review of piping developments. After a detailed elaboration of the Selmeijer's design rules, the influences of the parameters are discussed. Followed by the methods of determining the hydraulic conductivity of soils are described. In conclusion the literature study finished with a overview of local spatial variability in the subsoil.

2.2 Piping

This section is divided in three parts: (1) a theoretical description of the piping process, (2) a small overview of historical observation of piping in practice and (3) a historical review of piping development including the design rules.

2.2.1 Piping in theory

Internal erosion is one of the major causes of dike and dam failure. Failure is defined as the inability to achieve a defined performance threshold (to response a given loading), for a given function. (Morris, 2008). In this report a more tangible definition is used, especially for flood defences. Therefore, failure is defined as the inability to prevent unintentional inundation for flood susceptible areas, this commonly implies the initiation or development of a breach. Internal erosion refers to any process that results in soil particles being eroded from within or beneath an embankment due to seepage flow (Robbins & van Beek, 2015) (ICOLD, 2015). There are four major mechanisms of initiation of internal erosion:

1. Concentrated leaks
2. Backward erosion piping (BEP)
3. Contact erosion
4. Suffusion

This thesis mainly investigates the effects of anisotropy and heterogeinity for backward erosion piping (BEP) also refered to as simple piping. Detailed description of the other mechanismes are given in (ICOLD, 2015). Piping is one distinct internal erosion process which occurs in the aquifer or sandy foundation covered by a cohesive impermeable cover layer. An aquifer is a permeable region or layer in the saturated zone (Fitts, 2013). The backward erosion process affects the stability of the structure and can eventually lead to failure. The phenomenon of piping can occur if there is a high water level in the outer water, for instance rivers, canals and lakes, and relative low inland water or inland water table, such that the hydraulic gradient (outer water level minus inland water level) is of significance. For the sake of clarity, the outer water is identified as river in this thesis.

The hydraulic gradient induces seepage flow or ground water flow through the aquifer or foundation. The water seeping through the pores increases the hydraulic forces inside the aquifer or deep sand layer. A hydraulic fracture or crack arises, when pore pressures exceed the weight of the covering layer and water starts to flow out. This is the second phase of the piping process which is commonly called *uplift*. In the next stage, the water flow from the aquifer to the surface level increases so that detached sand particles are washed out and a channel starts to form a channel or pipe underneath the dike(ENW, 2010) (Kanning, 2012). The sand particles are deposited around the boil resulting in a sand crater or sand boil, with fluizing boiling sand at its centre (Van Beek et al., 2011). The initial erosion of sand is called *heave* (Kanning, 2012). Once, the erosion has been initiated the particles may continue to gradually erode backward along the interface of the cover layer until a pipe has formed that will eventually break into the river (Robbins & van Beek, 2015). This phase is called *the backward erosion process(BEP)*. The erosion process tends to stop automatically. Since the flow decreases when the pipe develops towards the river (ENW, 2010). If the flow is insufficient to transport the particles, the pipe will become blocked and piping will stop, so a new equilibrium is reached. The equilibrium will hold until the hydraulic gradient is beneath the maximum hydraulic gradient or critical gradient. Otherwise, the eroding process becomes progressive and the pipe develops until an open connection is created between the inner and outer side of the dike (Kanning, 2012). In the case of an open connection, the water flow increases inside the pipe, causing the pipe to widen (phase 4). This is followed by subsidence, and finally by breaching of the dike. Figure 2.1 illustrates the phases of the development of the piping process.

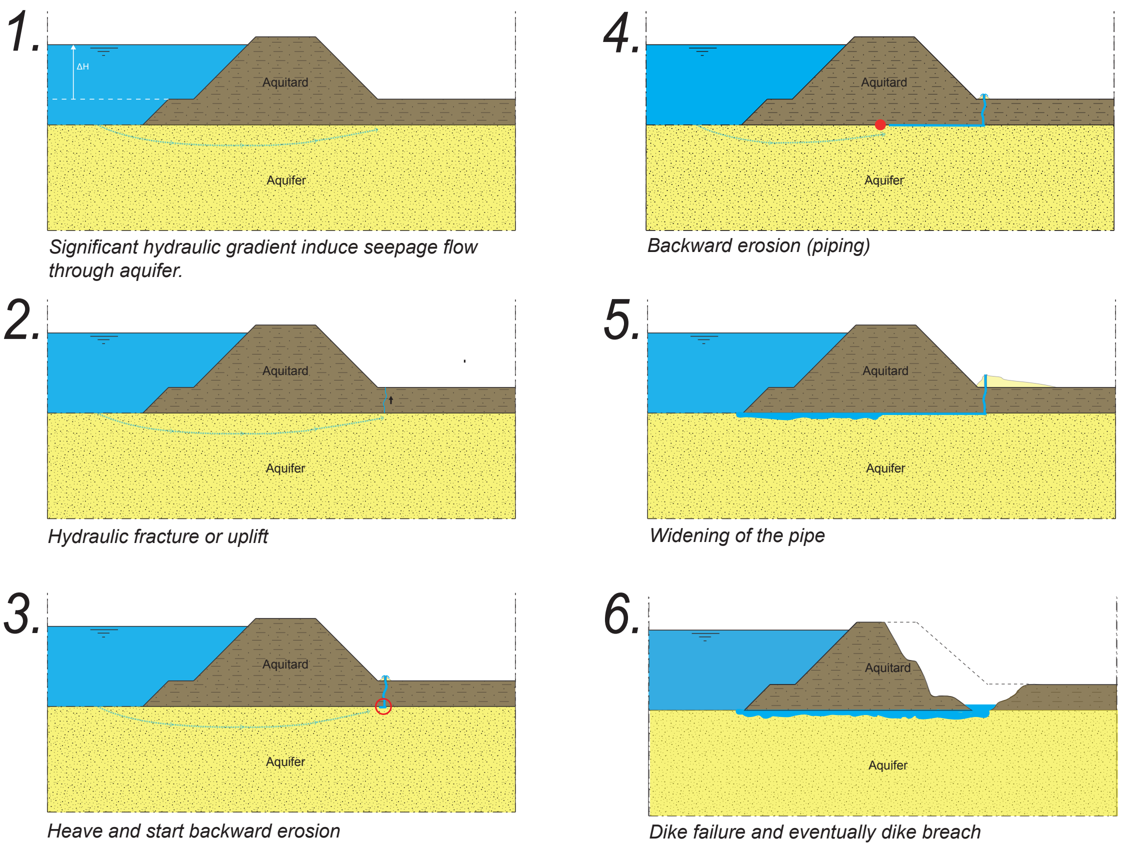


Figure 2.1: Schematic representation of the process of piping

2.2.2 Piping in practice

This section describes some historical cases of piping in practice, which were documented in the Netherlands and abroad, especially in the US. The cases are extracted from (Förster et al., 2012) and (ENW, 2010) and (Kanning, 2012).

2.2.2.1 Piping in the Netherlands

Fortunately, there are no recent cases of dike failure due to piping in the Netherlands. However, three historical cases of dike failure may be attributed to the phenomenon of piping (Förster et al., 2012):

- Dike breach in the Heidijk at Nieuwkuijk(1880)
- Dike breach polder Nieuw-Strijen at Tholen(1894)
- Dike breach at Zalk(1926)

At that moment, the phenomenon of piping was still unknown. Beside this, the documentation of these failures were described by laymen (Förster et al., 2012). As a consequence, these dike breaches are not describe in this report. During the high water in 1993 and 1995, sand boils were observed in Netherlands, although failure did not occur.

High water in 1993 (TAW, 1994)

During high water in 1993, one or more (sand) boils were noticed on 120 places, of which about 40 along the Rhine, 40 along the Waal, 30 along the IJssel and 10 along the Meuse. These (sand) boils have not caused any dangerous situations. The implemented mitigating measures, for instance ringing the sand boil, have been effective. Figure 2.2 shows a sand boil occurred in ditch near Ciesbeek on bank of the IJssel during the high water and Figure 2.3 shows a ringed boil that provides back pressure in order to prevent washing out of soil particles.

Figure 2.2: A sand boil in a ditch (TAW, 1994)

Figure 2.3: A boil ringed with sand-bags (TAW, 1994)

High water in 1995 (TAW, 1995)

During the high water in 1995, sand boils were noticed at about 180 place. Many boils that were observed in 1993 returned. However, in some cases the boils did not return or occurred in new places. Most of (sand) boils were observed along the Rhine and the IJssel. Boils were also observed along the Waal and the Lek. In many cases piping was contested by creating back pressure due to ringing of the sand boils or damming of the waterways. The construction of inland berms were also used to prevent sand boils.

Figure 2.4: A ringed sand boil in 1995 (TAW, 1995)

Figure 2.5: Ringed boils during high water of 1995 (TAW, 1994)

Piping sensitivity

Zwang and Bos (2009) investigated the piping sensitivity areas in the Netherlands. The results of this study is complemented with observations of sand boils from several Water Boards. An overview of sand boil observations is given in Figure 2.6. Figure 2.6 does not provide a nationwide view, since several Water Boards did not participate. Nevertheless it can be noticed that most of the sand boils were observed in the river-area.

Figure 2.6: Observations of (sand) boils in the Netherlands (Zwang & Bos, 2009)

2.2.2.2 Piping abroad

In 2005 the flood defence system in New Orleans collapesed due to Hurricane Kartrine. New Orleans is the biggest city in the southern state Louisiana in the United States of America. The flood therefore caused ernormous damage and social disruption. One of the breaches along the London Avenue Canal was most likely the consequence of piping (Kanning, 2012). The breach in the southern part of the London Avenue Canal exhibits clear indications of piping (ENW, 2010):

- The water level never exceed the height of the floodwall;
- The floodwall sank which indicates piping underneath the floodwall;
- Sand boils were observed in the vicinity of the breach;
- The geology of New Orleans is composed of a thick sand layer covered by an impermeable layer.

The piping process is executed during a high water level lasting several hours, while piping commonly causes problems during relatively long lasting loads that last for days or weeks. Hence the breach revealed that, if it was piping, it can occur very rapidly for specific conditions as well.

2.2.3 Historical review

This historical review outlines the most common methods to design or to assess dams and levees over the years. Clibborn and Beresford (1902) were perhaps the first that discovered a relation between the seepage length and the head at which piping could occur. Still, little information was provided regarding guidelines for designing to prevent piping.

2.2.3.1 Bligh

Bligh (1910), a British engineer for the Royal Navy, published the first method for designing to prevent piping. He came up with a empirical rule, which is also known as the creep theory, based on his experiences with numerous weir failures in India (Van Beek et al., 2011):

$$\Delta H \leq \Delta H_c = \frac{L}{C_{creep}} \tag{2.1}$$

Bligh supposes a relation between the minimal seepage length underneath the weir (L), the critical hydraulic gradient over the weir (ΔH_c) and the quantity called the percolation coefficient (also known as a creep ratio C_{creep}). The percolation coefficient depends on a qualitative characterisation of soil types in the piping sensitive layer, see Table 2.1. However, the basis of these values is unknown (Robbins & van Beek, 2015). During the next 80 year, Bligh's formula provide, while crude, the basis for dam and dike designs throughout the world.

2.2.3.2 Terzaghi

Terzaghi (1929) considered a vertical equilibrium (vertical gradient) between the upward seepage forces acting on a body of soil and the downward gravitational forces. The vertical critical gradient occurs when the seepage forces exceeds the downward gravitational forces and the particle movement will initiate at the exit point (Van Beek, Bezuijen, Sellmeijer, & Barends, 2014). This process had been referred to as heave. Terzaghi proposed the following equation for determination of the critical heave gradient:

$$i_c = \frac{(\gamma_s - \gamma_w)}{\gamma_w}(1-n) \tag{2.2}$$

Where γ_s is the unit weight of saturated soil, γ_w the unit weight of water and n is the porosity. It should be noted that the vertical gradient does not indicate that sufficient horizontal gradient exist for a pipe to progress beneath a structure, hence inition only (Van Beek et al., 2014). Besides, it is limited to homogeneous material.

In the Netherlands, it assumed that when the vertical exit gradient is greater than 0.5 sand boils arise and the development of pipe needs to be verified. The occurring vertical heave gradient(i_o) is calculated with:

$$i_o = \frac{(\phi_o - h_p)}{d} \leq i_c \tag{2.3}$$

Where ϕ_o is the hydraulic head at the bottom of the cover layer where the vertical gradient is maximum, h_p is the free water level or ground level and d_b is the thickness of the cover layer. The ϕ_o can be calculated with differential equations which assume a stationary ground water flow in the sand layer. The differential equations can be found in Förster et al. (2012).

2.2.3.3 Lane

Lane (1935) argued that the vertical seepage length has more significance on the safety than the horizontal seepage length. He revised the Bligh's creep ratio concept to account for the increased vertical seepage lengths provided by vertical cutoffs (Robbins & van Beek, 2015). The adjusted creep ratio known as weighted creep ratios on basis of a review of 278 dams. Lane (1935) proposed the following empirical calculation rule:

$$\Delta H \leq \Delta H_c = \frac{\frac{1}{3}L_h + L_v}{C_{w,creep}} \tag{2.4}$$

Where ΔH_c is the critical hydraulic gradient over the structure, L_h is the total horizontal seepage length, L_h is the total vertical seepage length and $C_{w,creep}$ the weighted creep ratio. The weighted creep ratios are various for different soil types as shown in Figure 2.1. Although weighted creep ratio factors are a significant improvement, the values still provide limited information on the relative safety factor against backward erosion piping (BEP). In addition, it is impossible to take site-specific conditions into account (Robbins & van Beek, 2015).

Table 2.1: Creep ratios for emperical rules Bligh and Lane (Förster et al., 2012)

soil types	median grain size(μm)[1]	C_{creep}(Bligh)	$C_{w,creep}$(Lane)
very fine sand, silt	< 105		8.5
fine sand	105 - 150	18	
fine sand (mica)		18	7
moderate fine sand (quartz)	150-210	15	7
medium sand	210-300		6
coarse sand	300-2000	12	5
fine gravel	2000-5600	9	4
medium gravel	5600-16000	-	3.5
coarse gravel	> 16000	4	3

[1] indications conform NEN 5104 (September, 1989)

2.2.3.4 Blanket theory

Bennett (1946) presented a detailed analytical approach for accounting the effect of semipervious blankets for assessing levee under-seepage. The analytical framework was later adopted by the United States Army Corps of Engineers (USACE) on the basis of a detailed field study of levee under-seepage in the Mississippi River levee system (USACE, 1956) (Meehan & Benjasupattananan, 2012). This analytical solution is commonly known in the United States as 'the blanket theory'. The blanket theory assumes; vertical flow in the confining layer(blanket), horizontal flow through the aquifer is horizontal and laminar seepage. The USACE (2000) adopted this analytical methodology as their primary approach for analysing levee under-seepage, as described in the their manual "Design and construction of levees". For situations where the confining layer permeability is significant lower (factor of 10 or more) than that of the foundation sands, the blanket theory equations accurately culculate under-seepage quantities and vertical exit gradient at the landside embankment toe (Robbins & van Beek, 2015). The exit gradient through the blanket is chosen as a criteria for designing levee against BEP in the United Stated (Kanning, 2012). For details regarding the analytical solution and the usage of the analytical methodology is refered to (USACE, 2000).

2.2.3.5 Sellmeijer

The first analytical method that theoretically accounts for the process of piping was derived by J. B. Sellmeijer (1988). In order to develop his analytical model, Sellmeijer first simplified the problem of piping. Within the simplification Sellmeijer assumed; a relative simple geometry with a homogeneous aquifer beneath an impermeable (clay) levee, constant head boundaries both upstream and downstream of the levee and a steady state seepage solution for the piping process (Robbins & van Beek, 2015). In addition, he assumed that the back erosion process is dominated by the sediment transport conditions in the bottom of the pipe. Sellmeijer's model is basically an equilibrium model that calculates whether a maximum head, better known as the critical head, is reached. His general idea, validated with observations and numerical computations, is that the piping channel can reach an equilibrium if the head does no exceeds the critical head. When the critical head is exceeded the process of piping will progressively increase until failure (Kanning, 2012). Sellmeijer established the design rules by curve fitting the numerical results. The original rules are given in Appendix A. The design rules were adjusted through a multivariate anylisis of the numerous test conducted by H. Sellmeijer et al. (2011) and van Beek et al. (2011). To this day, Sellmeijer's model remains the official design standard for levees in the Netherlands and has received increasing attention abroad (Robbins & van Beek, 2015). Similar to Bligh and Lane, the critical hydraulic head is defined in the adjusted design rules (Förster et al., 2012)

$$\Delta H_c = L\ F_S\ F_R\ F_G \tag{2.5}$$

$$F_R = \eta \frac{\gamma_p'}{\gamma_w} tan\theta \left(\frac{RD}{RD_m}\right)^{0.35} \tag{2.6}$$

$$F_S = \frac{d_{70m}}{\sqrt[3]{\kappa L}} \left(\frac{d_{70}}{d_{70m}}\right)^{0.4} \tag{2.7}$$

$$F_G = 0.91 \left(\frac{D}{L}\right)^{\frac{0.28}{\left(\frac{D}{L}\right)^{2.8}-1}+0.04} \tag{2.8}$$

Where:

ΔH_c	critical hydraulic head [m]
L	the horizontal seepage length under the levee [m]
F_R	resistance factor
F_S	scale factor
F_G	geometrical shape factor
D	aquifer depth [m]
η	White's constant [-]
γ'_p	unit weight of particles [kN/m^3]
γ_w	unit weight of water [kN/m^3]
θ	bedding angle [°]
d_{70}	grain diameter for which 70 percent of particles are smaller [m]
d_{70m}	mean d_{70} in the small scale tests ($2.08{\cdot}10^{-4}$) [m]
κ	intrinsic permeability of the aquifer [m^2]
RD	relative density [-]
RD_m	mean relative density in the small scale tests (0.725)

0.3d-rule

The hydraulic gradient used in the rule of Sellmeijer is reduced by a factor of 0.3 times the thickness of the blanket(d_b) in situations with a covering layer (e.g. clay and / or peat) on a piping-sensitive sand or soil layer. By this reduction, The additional resistance of the fluidized sand grains in the uplift channel is taken into account. This reduction is known as the '0.3d-rule'.

Besides, it should be noted that not all sand boils indicate whether the critical condition is reached, so it can still be a safe situation. The degree of safety ,if a sand boil has appeared , can theoretically be determined by Sellmeijer's rules. However, the model is derived by assuming perfectly uniform foundation conditions. Recent research has proved that variations from the assumptions influence the behaviour and critical values and thereby the safety of the structure. The influential factors are discussed in next section.

2.3 Factors influencing the critical values

This section briefly discusses the influential factors for the piping process. The influential factors discussed are exit configuration, aquifer depth and soil characteristics. This list contains factors for which experimental evidence is available (Robbins & van Beek, 2015). All parameters are discussed separately. However, it is difficult to be conclusive about the individual influence of parameters, since some parameters are highly correlated , which can amplify or weaken the impact on piping process. Most of the information in this section is extracted from Robbins and van Beek (2015).

2.3.1 Exit configuration

The exit configuration, or shape of the exit dominates both the flow pattern in the subsurface and the resistance near the exit (Robbins & van Beek, 2015). If an exit has a confine circular perimeter the seepage flow converges towards the exit, resulting in high local gradient. On the other hand, the convergence of the seepage flow is less for large shaped exits, resulting in lower local gradients, which makes pipe initiation less likely. de Wit (1984) proved this phenomenon by performing experiments with different shapes of exit. Additionally, Van Beek, Van Essen, Vandenboer, and Bezuijen (2015) showed in more recent experiments with circular exit that the 2D Sellmeijer model overestimates the critical gradient by a factor of 2. The outcome of Van Beek et al. (2015) research is shown in Figure 2.8a.

2.3.2 Aquifer depth

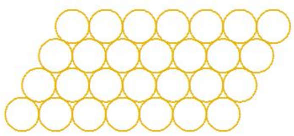
The depth of the aquifer highly influences the piping process. A larger depth results in a larger flow area. Consequently, more water is flowing towards the exit or head of the pipe, so therefore induces higher local gradients. This kind of scale effect is taken into account in Sellmeijer's model. Van Beek et al. (2015) and Hanses (1985) performed several scale experiments. The results of both scale experiments were compared by Robbins and van Beek (2015) and are shown in Figure 2.8b. The aquifer thickness D is not overly important for the resistance; it is, however, important for the flow patterns and velocity.

2.3.3 Pipe Path

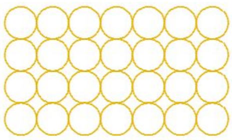
The pipe path in laboratory test is generally perfect horizontal and smooth. This pipe development can be clarified by the fact that most experiments are performed under homogeneous conditions. However, natural barriers in the top of aquifer will influence the pipe path. This effect is already noticed by Kanning (2012) in the 2D (width and length) plane. But, this effect can also occur in the 3D plane (width, length and depth) depending on the site specific conditions.

2.3.4 Relative density

The relative density (RD) defines how loose or dense an aquifer is packed. The relative dense packed grains have more resistance due to interlocking of grains and dilatancy. As a result it's more difficult to erode dense packed soils. In addition, dense packed soils are less permeable and less flow enters the pipe, see Section 2.3.7. Figure 2.7a and Figure 2.7b show a very dense and a very loose packing.



(a) very dense packing



(b) very loose packing

Figure 2.7: An illustration of a loose and a dense packing (Verruijt, 2001)

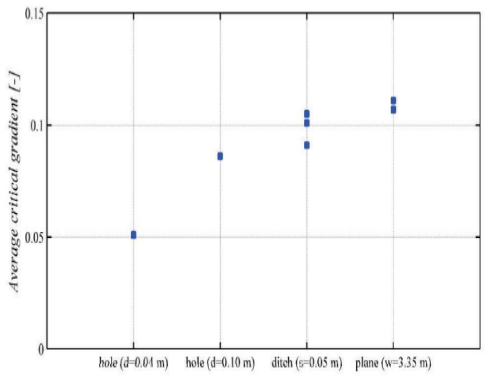
The relative density is directly dependent on the porosity of the soil. The porosity is defined as the ratio of the volume of the pore space and the total volume of the soil. For most soils the porosity is ranged between 0.30 and 0.45 (Verruijt, 2001). Low values of porosity are associated with densely packed soils, while large porosity is associated with loose packed soils. The influence of porosity is investigated by Robbins and van Beek (2015) and Van Beek et al. (2015) and shown in Figure 2.8c.

2.3.5 Grain size

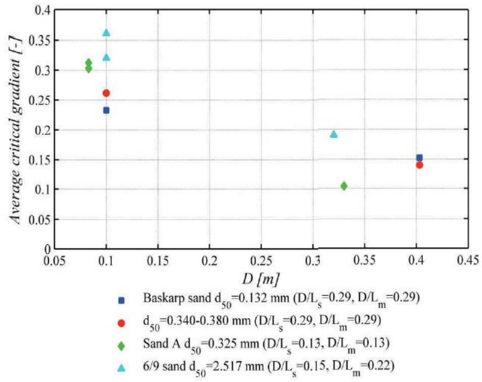
The grain size has a influence on critical gradient. Since, the coarser grains are more difficult to erode and transport due to the higher mass. The d_{70} grain size, which is the 70^{th} mass percentile of a sand passing through a sieve with mesh d, is implemented in Sellmeijer's model. According to Sellmeijer's rule the critical gradient will increase with grain size. However, in scale experiments with uniform sand by Van Beek et al. (2015) the influence of grain size on the critical gradient was limited or compensated by other properties in soil characteristics such as the permeability. The erosion process occurs in the top of the aquifer underneath the impervious layer. Thus, the grain size d_{70} in the top of the aquifer mainly determines the resistance against backward erosion piping, whereas the grain size distribution in the entire aquifer mainly affects the seepage flow and therefore also affects the likelihood of the uplift and heave.

2.3.6 Uniformity coefficient

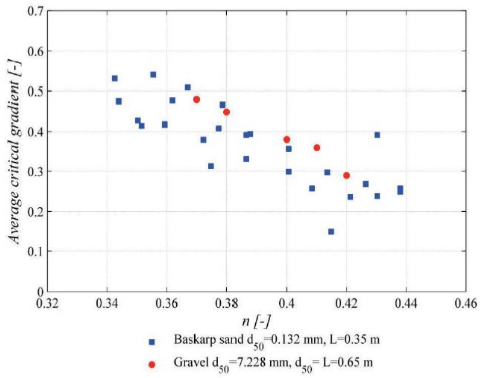
The coefficient of uniformity is the ratio of the large grain fraction and smaller grain factor, $C_u = \frac{d_{60}}{d_{10}}$, where d_{60} and d_{10} are the grain diameters of 60 % and 10 % passing through a sieve. The influence of the uniformity coefficient is mainly advocated by Schmertmann (2000). The experiments performed by Van Beek et al. (2015) also prove that critical gradient increases with the uniformity and contributes to the piping resistance. Figure 2.8d shows the effect of uniformity coefficient found in the experiment by Van Beek et al. (2015).



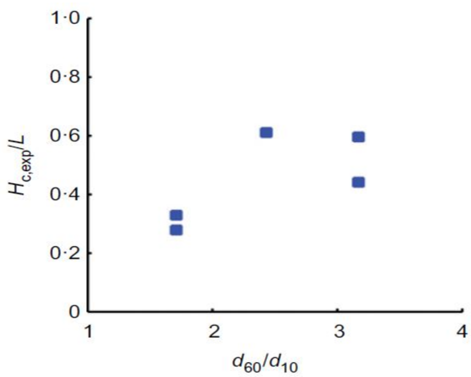
(a) The influence of the exit configuration



(b) The influence of the aquifer depth



(c) The influence of the porosity



(d) The influence of the uniformity coefficient

Figure 2.8: The influence of parameters on the critical gradient extracted from (Robbins & van Beek, 2015) and (Van Beek et al., 2015)

2.3.7 Permeability

The permeability of the aquifer highly influence the critical gradient. A highly pervious aquifer allows more water to be conveyed to the exit of head of the pipe. This effect was also found in Sellmeijer's numerical model, see Figure 2.9. However, the permeability is positively correlated to several other soil characteristics like, grain size, uniformity and porosity (van der Zee, 2011). This correlation makes it difficult to assess the distribution of the permeability.

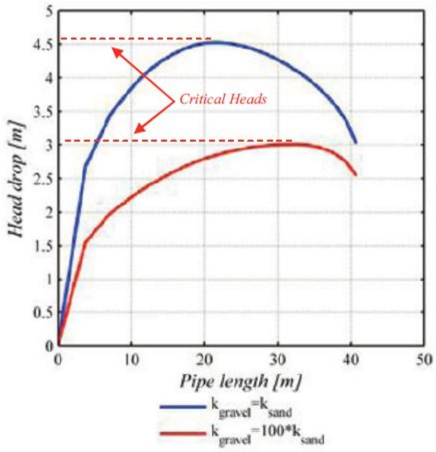


Figure 2.9: Influence of permeability on the (critical) head (Robbins & van Beek, 2015)

The permeability is essential to calculate the amount of ground water flow in Darcy's Law. Henry Darcy, one of the first engineers who examined the amount of ground water flow, derived the diffential equation from laboratory experiments (Fitts, 2013). The differential equation for the one dimensional ground water flow is given below,

$$Q_s = -K_s \frac{dh}{ds} A \tag{2.9}$$

$$q_s = -K_s \frac{dh}{ds} \tag{2.10}$$

where h is the hydrualic head, s the distance, A the cross-section area of the column, Q_s is the discharge in the direction s and K_s is the hydraulic conductivity in the direction s and has historically been used synonymous with permeability. The permeability is nowadays associated with the *intrinsic permeability*, so that is not specifiek to the fluid water. The intrinsic permeability κ is proportional and related to hydraulic conductivity as follows (Fitts, 2013):

$$\kappa = \frac{K\mu_w}{\rho_w g} \tag{2.11}$$

The effect of heterogeneity in the aquifer is mainly researched by varying the permeability, despite the correlation to several other soil characteristics, since the ground water flow or seepage flow is the main driver of piping. Additionally, the permeability is the only parameter in the groundwater differential equation which represents the soil characteristics of the aquifer. The next section outlines the hydraulic conductivity in more detail, especially the spatial variation and establishment of the hydraulic conductivity.

2.4 The establishment of the hydraulic conductivity

As is stated in previous section, the progression of piping depends to a great extent on the hydraulic conductivity (K) of the aquifer. Therefore, it is important to determine the K-value as accurately as possible. However, the permeability varies in space and time (Section 2.4.1), which makes it difficult the determine an accurate representative value (Ritzema, 1994). In addition, determining the K-value is costly and time-consuming. As a result, a balance will often be made between budget limitations and desired accuracy. Various methods are developed to determine the K-value of soils and are briefly described in Section 2.4.2. These different methods are compared to show the merits and constraints of each method.

2.4.1 Variability of the hydraulic conductivity

The hydraulic conductivity is mainly depending on the shape and the size of the soil particles and the distribution of the pores. Additionally, it is also influenced by the soil temperature, viscosity and density of the water. The K-value of saturated soil represents the average hydraulic conductivity and is usually less than 10 m/day or 2.74 * 10^{-2} m/s (Ritzema, 1994). In few sandy soils, the hydraulic conductivity is the same in all directions (isotropic). However, the hydraulic conductivity is generally anisotropic, so it varies with the direction of flow. The vertical hydraulic conductivity of soil (layer) often differs from the horizontal mainly due to vertical difference in texture, structure, and porosity due to layered depositions and biological activities (Ritzema, 1994).

The groundwater is highly influenced by anisotropy. Due to anistropy, ground water flow alters per direction. The hydraulic conductivity in horizontal and vertical direction are expressed in K_h and K_v. Wit (1967) performed some experiments, where he measured both the vertical and horizontal hydraulic conductivity of soil samples. Examples of the vertical and horizontal hydraulic conductivity of soil samples are shown in Figure 2.10 and Table 2.2 (Wit, 1967), in which the anisotropy is clearly visible.

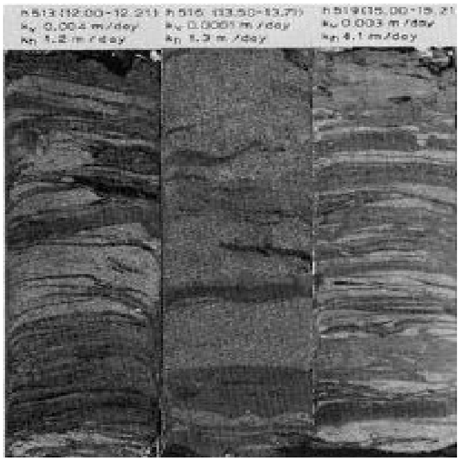


Figure 2.10: Soil samples from old tidal flat deposits in polder "De Oude Korendijk."

Table 2.2: Hydraulic conductivity of the soil samples from the old tidal flat deposits from left to right

sample numbers	K_h [m/day]	K_v [m/day]
h513	1.2	0.004
h516	1.3	0.0001
h519	4.1	0.003

Todd (1980) noted that anisotropic behaviour is principally characteristic for undisturbed alluvial material. Alluvial material or alluvium is loose and unconsolidated soil or sediment that has been deposited by water in non-marine environments. The alluvial sediment includes sands, silts, clays or gravel (Geology Dictionary, 2017). Todd came up with two explanations for anisotropy in unconsolidated soils for both small and large scale (Freeland, 2013):

1. Soil particles are rarely spherical. If soil particles settle down in water, they land on their long and flat sides. In addition, most clay minerals have a platy habit and would land on their flat side as well. This process occur at particle level and therefore is a small scale process.

2. Alluvium has the tendency to consist of layers with different grain sizes and hydraulic conductivity. This stratification occurred due to changes in flow patterns or sediment sources. The process can lead to heterogeneous subsoils composed of thick and long different layers and therefore is a large scale process.

Todd concluded that hydraulic conductivity in stratified sediment tends to be higher in the horizontal direction. The soil structure like, size of soil, distribution of pores and depth are key characteristics in determining the anisotropy of the soil. In addition, Whipkey and Kirkby (1978) presume that the weight of overlying material or "geostatic pressure" tends to reduce the porosity and the hydraulic conductivity with depth.

2.4.1.1 Variability within soil types

Smedema and Rycroft (1983) came up with a generalised table with ranges of K-values for certain soil types, see Table 2.3. Smedema and Rycroft warn that: 'Soils with identical texture may have quite different K-values due to difference in structure' and 'Some heavy clay soils have well-developed structures and much higher K-values than those indicated in the table'. So, this table should be used to get an indication of the permeability.

Table 2.3: The range of K-values by soil type (Smedema & Rycroft, 1983)

soil types	K [m/day]
gravelly coarse sand	10 - 50
medium sand	1 - 5
sandy loam, fine sand	1 - 3
loam, clay loam, clay (well structured)	0.5 - 2
very fine sandy loam	0.2-0.5
clay loam, clay (poorly structured)	0.002 - 0.2
dense clay	$<$ 0.002

2.4.1.2 Variability between soil layers

If a subsoil is stratified, it is often perceived that the hydraulic conductivity differs across the different layers. In general, the more sandy layers have a higher K-value then more clayey layers, but there are exceptions (Ritzema, 1994). The representative value of K depends on the direction of the groundwater flow. If water flows parallel to soil layers, the representative value can be determined by a summation of the transmissivity of the layers (Equation 2.12). The transmissivity is defined as the hydraulic conductivity multiplied with the thickness of the layer ($T = KD$). The representative value of K for groundwater flow perpendicular to the soil layers can be determined by a summation of the hydraulic resistances (Equation 2.13). The hydraulic resistance is defined as thickness of the layer divided by the hydraulic conductivity (Ritzema, 1994).

$$K^* D_t = \sum_{i=1}^{n} K_i D_i \qquad \text{parallel flow} \tag{2.12}$$

$$\frac{D_t}{K^*} = \sum_{i=1}^{n} \frac{D_i}{K_i} \qquad \text{perpendicular flow} \tag{2.13}$$

Where:

K^* weighted average hydraulic conductivity $[\frac{m}{d}]$
D_t total thickness of the soil layers $[m]$
i number of the soil layer
n total number of soil layers

2.4.1.3 Geomorphology

An interesting geomorphological process occurs in the flood plains. During floods, the coarser soil particles (sand and silt) are deposited as natural levees near bank of the river, while the finer particles (silt and clay) are deposited further away from the river. The fine particles in the flood plains are also called the basin soils. The hydraulic conductivity of these basin soils (from 0.1 to 0.5m/day) can be a factor of 10 lower than the hydraulic conductivity of the natural levees soil(from 2 to 5m/day) (Ritzema, 1994). Due to the meandering of rivers, irregular and complex patterns of natural levee and basin soils have emerged in river deltas. In addition, the basin soils can become considerably more permeable due to the presence of organic material, which can be found at various depth. The relationship between the hydraulic conductivity and geomorphological characteristics is not clear for every site.

2.4.2 Hydraulic conductivity determination methods

The most common methods to determine the hydraulic conductivity are correlation methods and hydraulic methods. The hydraulic methods are divided into relatively small-scale laboratory test and field (in-situ) methods (Ritzema, 1994). The field method are subdivided into small scale and large scale methods. The small scale method are fast and mobile, whereas the large scale are time-consuming and expensive. However, the large scale methods guarantee a more reliable representative K-value of the aquifer. Firstly, all methods are discussed individually. These discussions reveals some pros and cons of each method. The conclusion contains an overview of these pros and cons. The different hydraulic conductivity determination methods are presented in a dendrogram in Figure 2.11.

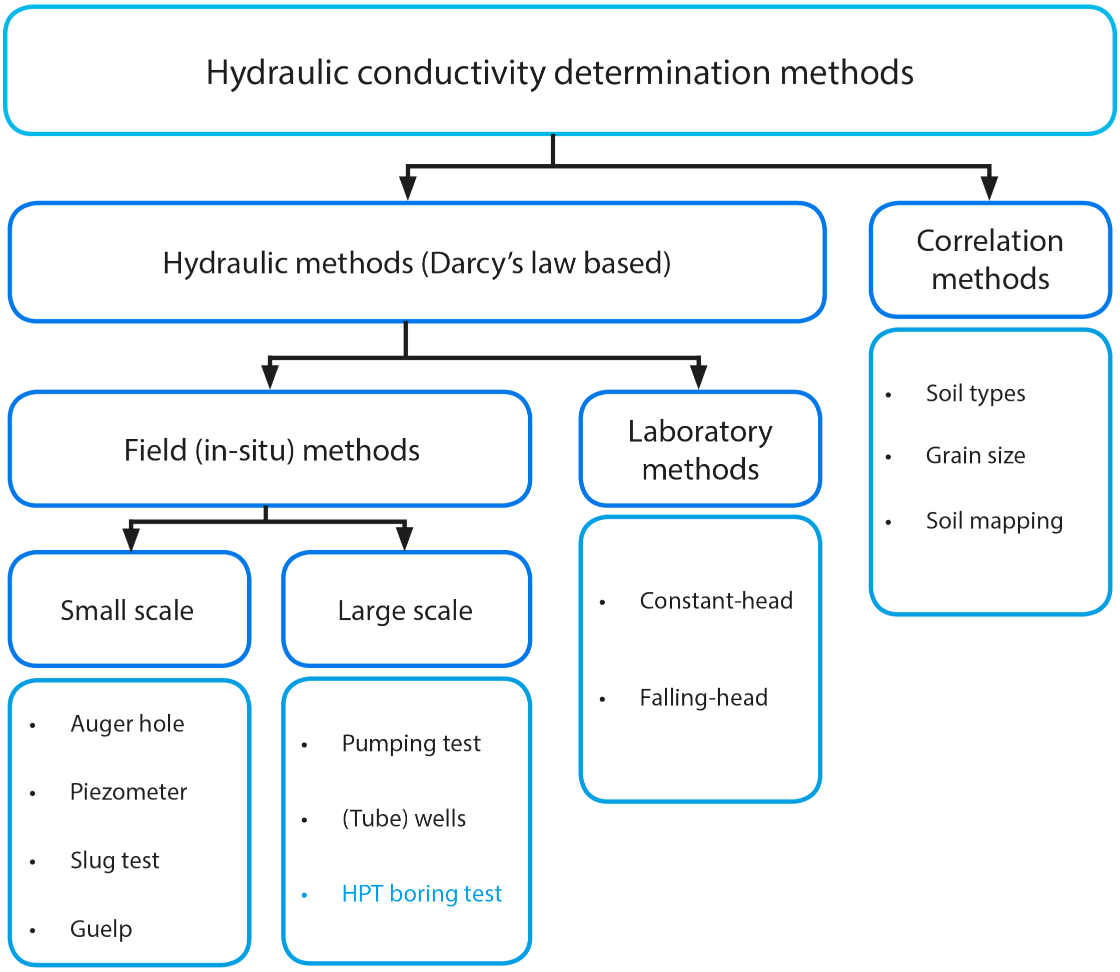


Figure 2.11: An schemetical overview of methods to determine the hydraulic conductivity (Ritzema, 1994)

2.4.2.1 Correlation methods

The correlation methods are based on relations between the hydraulic conductivity and one of more easily determined soil properties, like the soil types (Section 2.4.1.1), grain size distributions or soil mappings. The advantage of the correlations methods is that the K-value can be determined quite simple and quick. However, the correlations method are based on rough relations between soil properties and can therefore result in incorrect assessment of the soils. In addition, Berbee, van Goor, and

Martac (2017) revealed that results of different correlation methods considerable differ. The difference will be roughly a factor of 10 in permeability.

Grain size distribution

The hydraulic conductivity is a function between the size and distribution of pores in a material. It makes sense that there is some correlations between the hydraulic conductivity and the grain size, yet the grain size is not a prefect measure of the size, orientation and connectedness of the pores (Fitts, 2013). About a century ago, Hazen (1911) developed the following empirical relation between the hydraulic conductivity and grain size, based on experiments with different sand samples.

$$k = C(d_{10})^2 \tag{2.14}$$

where K is hydraulic conductivity in cm/s, C is the Hazen empirical constant with units of $(\text{cm} \cdot \text{s})^{-1}$ and d_{10} is the diameter grain size in centimetres where 10% soil passing through a particular sieve. The constant C varies from 40 to 150 for most sand. Low values of C are related to fine and widely graded sands, whereas high values of C are related to coarse and narrow graded graded sands (Fitts, 2013).

About half a century thereafter, Kozeny and Carmen developed a more widely semi-empirical and semi-theoretical equation used to determine the hydraulic conductivity of porous media (Carrier, 2003) (modified from (Fitts, 2013)):

$$K = \left(\frac{\rho_w g}{\mu}\right)\left(\frac{n^3}{(1-n)^2}\right)\left(\frac{(d_{50})^2}{180}\right) \tag{2.15}$$

wherein $\rho_w g$ is the weight/viscosity of water, n is the porosity and d_{50} is the grain diameter size where 50% soil passing through a particular sieve.

It should be noted that the determination of the hydraulic conductivity based on the grain size correlation on a large scale(m) is quite inaccurate, since heterogeneity and anisotropy will often not be considered. However, at local level or very small scale heterogeneity do not exist and anisotropy is only present in grain orientation and packing so that the grain correlation will be quite suitable. Nevertheless, the values derived with empirical formula should mainly be used as indication values.

Soil mapping

In the U.S.A., soil mapping is done on the basis of soil series, in which various soil properties are combined. These soil series are often correlated to a certain range of the hydraulic conductivity (Ritzema, 1994). The Netherlands Organisation for applied scientific reseach (TNO) also constructs soil maps. For instance, REGIS, a lithographic and hydrogeological map, which is mainly based on approximately 14.500 selected moderate deep boreholes from the database DINO (TNO, 2005). Within REGIS, permeabilities are deduced for each soil layer with the same hydraulic characteristics on the basis of their lithographic class. In REGIS, regional boreholes can be selected, which shows vertical sequences of model units with respect to the ground level for a user-selected location. The user gets insights into the depths, thickness and hydraulic properties of these units, such as the vertical and horizontal permeability. An example is shown in Figure 2.12.

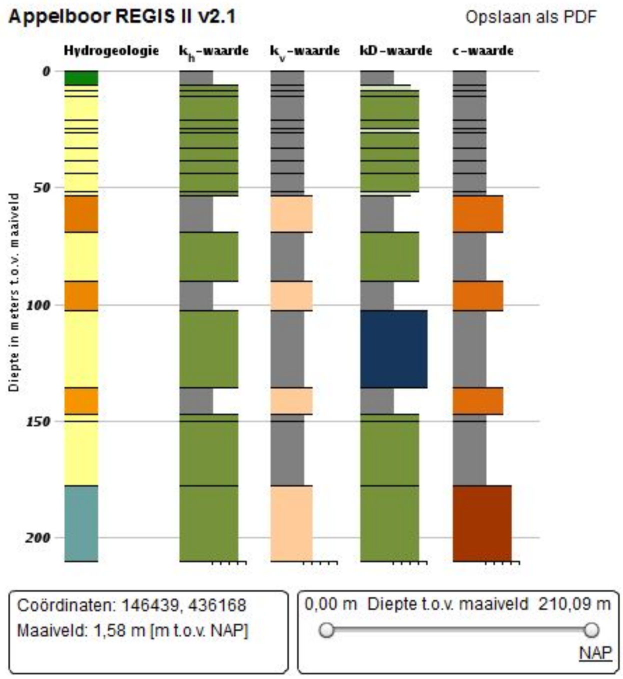


Figure 2.12: An example of a visualistion in REGIS (TNO, 2005)

2.4.2.2 Laboratory tests

Laboratory measurements of the hydraulic conductivity are generally performed on small soil samples, with dimensions in the order of 10-50 cm, contained in metal cylinders or cast in gypsum (Fitts, 2013; Ritzema, 1994). The samples retrieved from borings are usually in line with borehole, so the lab tests measure the vertical hydraulic conductivity K_v (Fitts, 2013). Nevertheless, samples can be taken in horizontal direction to establish the K_h value. In most cases, the hydraulic conductivity can be determined quite accurate. However, the laboratory tests are not suitable for extremely permeable or impermeable soil samples. In addition, a sample represents only a small areas of the subsoil, so many samples should be obtained to get a representative value of K. In conclusion, laboratory tests are good methods to measure the hydraulic conductivity of a certain small layer. Hence, laboratory test only provide local value for K_v and K_h. The two most laboratory tests used are described below: the constant-head test and falling-head test.

Constant-head test

In constant head test, a constant head difference is maintained across the soil sample, while water flows through the sample. During the test, the amount of water flowing through the soil samples is measured for given time intervals. Knowing the water discharge Q, the height of the sample column L, the sample cross section A normal to the flow, and the constant head difference ΔH, one can calculate the hydraulic conductivity with the following relation (Fitts, 2013):

$$K = \frac{Q}{A}\frac{L}{\Delta H} \tag{2.16}$$

Falling-head test

The falling head test differs from the constant head test in that the head difference and discharge decrease in time, while the test setup is practically similiar (Fitts, 2013). The hydraulic conductivity can be calculated from the velocity of total flux through the sample, which is more complicated to calculate because of the inconstant water head.

2.4.2.3 Small scale field methods

The small scale tests, as the name suggested, are unsuitable to great depths. However, the measured hydraulic conductivity at shallow depth can be used as indication of the hydraulic conductivity at greater depths, when it is proven that their results are representative for the entire aquifer and the vertical and horizontal hydraulic conductivity are nearly isotropic (Ritzema, 1994). The small scale tests are distinguished in tests applied in saturated or unsaturated aquifers. The hydraulic conductivity of unsaturated aquifers is obtained by infiltrating sufficient water in the aquifer. Such methods are therefore called **infiltration methods**. The hydraulic conductivity must be determined by using the relationship between the measured infiltrated water and the hydraulic head (Ritzema, 1994). In saturated soils water can be extracted from the soil, by for example creating a hole to observe the flow rate. These methods are called **extraction methods** and use the relationship between the extracted water and the induced hydraulic head (Ritzema, 1994). In the both methods, the equations describing the relationships should be formulated according to the boundary conditions. The most common methods small scale test are briefly explained below.

Auger-hole method

The Auger hole method is a widely extraction method used to determine the hydraulic conductivity of soils. The auger hole method is rapid, simple and reliable for shallow aquifers (van Beers, 1983). The general principle is very simple: a hole is bored into a certain depth below the water table (van Beers, 1983). A part of the water in the hole is exacted, when the equilibrium is reached with the groundwater. Water again flows into the hole and fills the hole. The rate of water rise in the hole is measured and thereafter converted by an appropriate equation to the hydraulic conductivity. The Auger method provides the average permeability of the soil layers governed below the water to a small distance below the bottom of the hole (van Beers, 1983). The main disadvantages of this method are: measurement of mainly the horizontal conductivity and unsuitable for highly stratified soils or for soils with an irregular pore distribution. In addition, the use of this method is limited to areas with a relative high ground water level and to soils where a stable hole can be maintained (van Beers, 1983).

Piezometer method

The piezometer method is also an extraction method and in principal based on the Auger-hole method, except a tube is always inserted into the hole, leaving cavity of limited height at the bottom (Ritzema, 1994). The hydraulic conductivity can basically be measured by flow rate into the cavity of the piezometer (Luthin & Kirkham, 1949). The advantage is that the method measured the hydraulic conductivity of separate layers at relatively great depth. The disadvantage is that the result is a single value which represents the hydraulic conductivity of the aquifer in the vicinity of the cavity.

Slug tests

The slug test can be either an extraction or infiltration method in which the head in a well is changed suddenly, and the subsequent variation of the head with time is measured as the head returns to the equilibrium level that exist before the modification of the head (Fitts, 2013). The modification is created by inserting or withdrawing a solid cylinder ('slug') from the water column of the well. The slug test is relatively simple, quick and inexpensive. A disadvantage is that the slug test produces an average of the horizontal hydraulic conductivity of small area surrounding the well (Fitts, 2013).

Guelp Method

The Guelph method (Reynolds & Elrick, 1985) is an infiltration method. The Guelp permeameter is an in-hole constant-head permeameter that measures a composite of vertical and horizontal hydraulic conductivity (Mohanty, Kanwar, & Everts, 1994). The permeameter is placed above a cylindrical well that is bored into the ground. The method involves measuring the steady-state rate of water recharge into unsaturated soil from the cylindrical hole, in which the constant depth (head) of water is maintained. When a constant well height of water is established, the outflow of water from the well reaches a steady-state flow rate, which can be measured. The rate of this constant outflow of water, together with the diameter of the well, and height of water in the well can be used to accurately determine the saturated hydraulic conductivity. The disadvantage of the Guelp method is that it only measures the conductivity of the top (measured) layer.

2.4.2.4 Large scale field methods

With large scale field methods, the hydraulic conductivity can be measured in larger and deeper aquifers. The advantage of these methods is that the flow path of the seepage water and the natural resistances in the aquifer are represented in the (overall) K-values produced by these methods. Therefore, it is not always essential to determine local variations of the K-values in horizontal and vertical directions. Besides, the (overall) K-values can be easily adopted in the design formulas. The disadvantages of these methods are that they are relatively time-consuming, complex and expensive (Ritzema, 1994).

Pumping test

In the pumping test, large constant volume of water is pumped from a well into an aquifer. The changes in the water table or head are measured at the pumping well and/or nearby the observation wells (Fitts, 2013). A typical pumping test continues for hours, days or even month. The pumping test produces an average horizontal conductivity and storage parameter of the aquifer. The flow path of seepage and hydraulic resistance are overall incorporated, but local variations of K-values can not be observed. In addition, the results of the pumping test apply most to the near vicinity of the pumping well, and to a lesser degree to the region surrounding the observation wells (Fitts, 2013).

Parallel drains method

In the parallel drain method, the hydraulic conductivity is determined on basis of observation on drain discharges and corresponding heads at some distance from the drains (Ritzema, 1994). The hydraulic conductivity can eventually be calculated with formulas appropriate for site conditions under which the drains are functioning. These tests are commonly performed in experimental fields, pilot areas or on exiting drains. The disadvantage of this method is that the theoretical formalas frequently induce random deviations, so statistical confidence analysis is required to substantiate the calculation (Ritzema, 1994). In addition, local variations of K-values can not be observed.

HPT boring test

The Hydraulic Profile Tool (HPT) is recently developed method that maps the geohydrological conditions of the subsoil. The HPT boring test registers the cone resistor, the local shear friction and the water pressure. During probing, a constant flow water is injected in the soil. The pressure that is needed to injected the constant flow of water is also registered. After which, the relative hydraulic conductivity is determined by dividing the amount of infiltrated water by the simultaneous measured pressure (Lubbers & Wolfs, 2014). This method provides comparable values of the hydraulic conductivity with regard to the classical pumping test. In contrast to the pumping test this method provides a detailed and continuous permeability profile of the subsoil (Leuvenink et al., 2017), which gives more insights in the heterogeneity.

2.4.2.5 Scale-effects in hydraulic conductivity determination method

Because both the literature data and the local subsoil research provide a permeability that is related to a large soil volume (or even the entire aquifer), there seems to be a scaling effect (Berbee et al., 2017). This scaling effect can be observed in Figure 2.13.

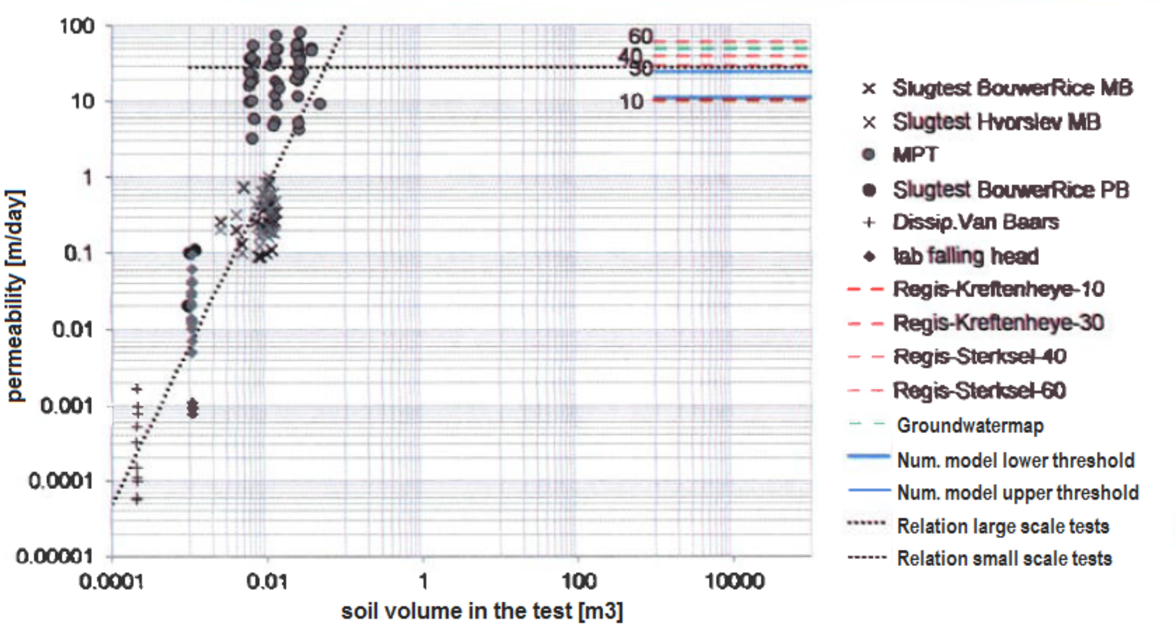


Figure 2.13: The permeability measured by the determination method versus the soil volume measured (Berbee et al., 2017)

In Figure 2.13, the soil volume involved in the test or method is plotted against the permeability results from the test or method. The soil volume is defined as the size of the laboratory sample examined or, in the case of an in situ test, the volume of water displaced divided by the porosity. The permeability of the small scale test is positively correlated to the soil volume involved in the test.

A clarification for the scaling effect suggested by Schulze-Makuch et al. (1999) is that the large-scale tests are dominated by the local presence of more permeable soil layers, while this does not apply in the small scale tests. If this clarification is completely correct, then occasional higher permeabilities should also be observed in the small scale tests. However, these occasionally higher permeabilities are not observed in the research of Berbee et al. (2017).

2.4.2.6 An overview of pros and cons per method

In the previous paragraph, several hydraulic conductivity determination methods were briefly described, whereby the main pros and cons were appointed. This paragraph gives an overview of the pros and cons, see Table 2.4. For the substantiation, reference is made to the preceding paragraphs.

Table 2.4: A brief overview of pros and cons per hydraulic conductivity determination method

method of determination	pros	cons
Correlation methods		
soil types	quick and simple	hard to classify every soil
grain size distribution	quick and simple	based on rough relation, which can result in inaccurate assessment
soil mapping	quick, simple, cheap and insights in some hydraulic properties	inaccurate, since these maps make use of classification.
Laboratory test		
constant head test	simple and quite accurate for small areas	limited usage, represents only small areas and a single K-value in usually the vertical direction only.
falling head test	simple and quite accurate for small areas	limited usage, represents small areas, a single K-value of usually the vertical direction only and more complex then the constant head test.
small scale field methods		
auger-hole method	quick, simple, relatively inexpensive and reliable	a single value for horizontal hydraulic conductivity and limited usage: applicable for shallow and nearly homogeneous aquifer with a high ground water level.
piezometer method	simple, relatively inexpensive, reliable, possible at greater depth and/or in stratified soil	a single value of the horizontal hydraulic conductivity in the vicinity of the cavity.
slug test	relatively simple, quick and inexpensive	an average of the horizontal hydraulic conductivity of small area surrounding the well
Guelp method	relatively simple, quick and inexpensive	only the hydraulic conductivity of the top (measured) layer
large scale field methods		
pumping test	evaluates larger volumes of aquifer, including seepage flow path and natural irregularities (heterogeneity)	time-consuming, expensive, single K value and represents mainly the surrounding of the pumping well.
parallel drain method	large scale determination, including seepage flow path and natural irregularities (heterogeneity)	time-consuming and based on theoretical relationships formulas
HPT boring test	detailed and continuous permeability profile	expensive and little experience in practice

3 | Model set-up

3.1 Introduction

The literature study provides several rules to design flood defences, especially levees, against piping. Despite of continuous research, it is still impossible to accurately assess heterogeneous subsoils with relative simple design rules. The Dutch government decided to apply the adjusted Sellmeijer's design rules for the legal assessment of the flood defences in spite of some imperfections. Firstly, the design rules contain multiple parameters with a certain dependency. Furthermore, parameters are determined by different methods ranging from simple and quick correlation methods to time-consuming and expensive hydraulic methods. These methods produce deviant outcomes. As a consequence, the deviant input values of the hydraulic permeability will be used in the assessment or design. Besides, these methods generally provide few insights into the heterogeneity of the subsoil. Another problem regarding the design rules is that they are derived by assuming perfectly uniform foundation. Recent research shows that this assumption highly influences critical values (Robbins & van Beek, 2015) and thereby the safety of the structure. This thesis continues on recent research by investigating the effects of anisotropy and heterogeneity in a model study.

In order to perform a proper model study, a basic model is set-up in D-Geo Flow. This chapter discusses the settings of the basic model, including the arrangement of soil types. Firstly, addition information of D-Geo Flow is provided. The next section outlines the required input parameters and presents several soil types deduced from literature (Section 3.3). These soil types are of importance, because they are used later on to create heterogeneous aquifer compositions. In order to verify the D-Geo Flow model with Sellmeijer's design rules, a standard geometry of a levee is generated. The standard configuration consists of a homogeneous layer with a clay levee on top. The boundary conditions assigned to standard configuration are also discussed within this section. The standard configuration and applied boundary conditions are discussed in section 3.4. Next, a verification is carried out in which the result of basic model is compared with the result of the design rules. The homogeneous aquifer compositions are established by varying the hydraulic conductivity K and the grain size d. This chapter concludes with a sensitivity analysis in order to investigate the effect of the parameter, geometry and boundary-choices.

3.2 D-Geo Flow

D-Geo Flow is a graphical user interface for the finite element computing platform DGFlow of Deltares. In D-Geo Flow, it is possible to perform 2D transient and steady-state groundwater flow calculation with complex and stratified foundations. The model includes time-dependent hydraulic load, compressibility of the soil skeleton and the groundwater, and change of the phreatic line. In addition, the model includes a piping module, based on Sellmeijer's model, to assess whether piping can occur at a given water level. The piping module is based on a force balance of the top grains along the erosion channel. The piping module and the possibility to include complex and stratified foundations are the main reasons for the selection of D-Geo Flow. Despite these assets, D-Geo Flow has a couple of limitations, see section 3.2.3. The finite element

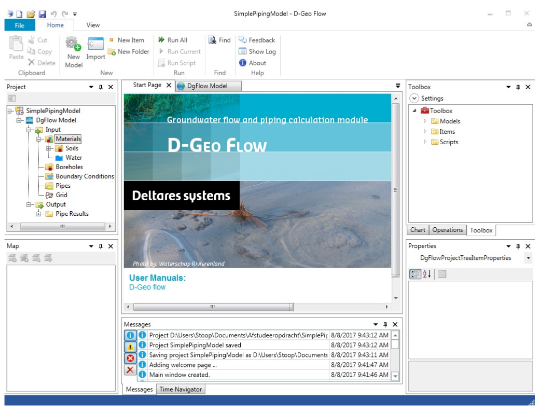


Figure 3.1: The interface of D-Geo Flow

method(FEM) and scientific background are also elaborated in next sections.

3.2.1 Finite element method

The finite element method (FEM) is a numerical method to solve engineering problems such as groundwater problems. The method solves a set of algebraic equations in which the unknowns are the heads at finite numbers of nodal points. In order to solve the problem, the problem domain is divided into triangular elements. The triangular elements are defined by three nodes, one at each corner. The nodes are the proposed location of unknown heads. Futhermore, the nodes are computed within the problem domain. The value within each element is computed by interpolating the nodal heads (Wang & Anderson, 1995). After defining the unknowns, the values would be computed. Besides, finite element method contains additional functions to approximate a solution by minimizing an associated error function. For more information regarding FEM reference is made to Wang and Anderson (1995). Figure 3.2 shows an example of a FEM grid produced by D-Geo Flow.

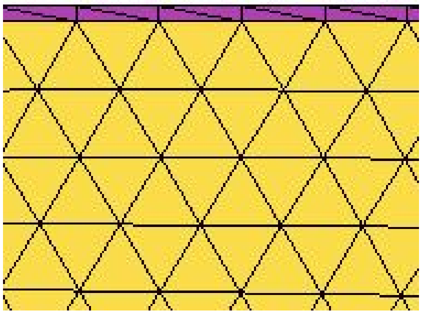


Figure 3.2: An example of a FEM grid

3.2.2 Scientific background

This section briefly outlines the two most important equations that are implemented in D-Geo Flow; The subsurface flow equation and the boundary condition along the erosion channel. More information regarding the scientific background can be found in (Deltares systems, 2017).

3.2.2.1 Subsurface flow equation

The flow through the saturated porous medium is modelled by considering continuity of fluids and a generalisation of Darcy's Law, which is described as follows:

$$(\alpha + n\beta)S\frac{\delta p}{\delta t} + n\frac{dS}{dp}\frac{\delta p}{\delta t} + \frac{\delta q_i}{\delta x_i} = 0, \quad \alpha = \frac{1}{\lambda + 2\nu}, \quad q_i = -\frac{k_r \kappa_{ij}}{\mu}\left(\frac{\delta p}{\delta x_i} - \rho^l g_j\right) \quad \text{on} \quad \Omega^p \tag{3.1}$$

Where:

α	the compressibility of the soil skeleton [m^2/N]
β	the compressibility of the pore water [m^2/N]
n	porosity [-]
S	the degree of saturation of the liquid phase in the void space [-]
p	pore pressure [m^2/N]
λ, ν	Lame's constant [N/m^2]
q_i	specific discharge [m/s]
k_r	relative permeability [-]
κ_{ij}	intrinsic permeability [m^2]
μ	the dynamic viscosity of liquid [kg/(m·s)]
ρ^l	the density of liquid [kg/m^3]
g_j	gravitation acceleration in y-direction[m^{2}s]
t	time [s]
Ω	the flow domain [m^2]

The problem can be solved by applying two types of boundary conditions; Dirichlet conditions prescribe the pressure on parts of the boundary and Von Neumann boundary conditions prescribe the derivative of the pressure flux on the boundary (Deltares systems, 2017).

3.2.2.2 Boundary along the pipe

The appropriate expression for the boundary condition along the erosion considers the continuity condition in the erosion channel and the limited stress state of the bottom particle. If the yet unknown height of the channel is eliminated, then the expression depends only on flow features. The expression is written by J. Sellmeijer (2006) as:

$$\sqrt[3]{\frac{Q}{kL}p_c^2} = \frac{\pi}{3}\eta\frac{sin(\theta+\phi)}{cos(\theta)}\frac{d}{\sqrt[3]{12\kappa L}} \tag{3.2}$$

where Q is the discharge in the channel; p_c is the gradient in the channel; L is the length of the pipe; η is White's constant; γ_p' is the unit weight of particles; γ_w is the unit weight of water; θ is the bedding angle; ϕ is the slope of the channel and d is the particle diameter. All other parameters were already mentioned in the previous equation.

Since the erosion channels are shallow and mainly arise in the sandy sublayer, the flow in the channel can be modelled by the Navier Stokes equations. These equations assume that the flow is laminar and are valid for sandy layer. If the sublayer consists of coarse gravel, the flow tends to be turbulent so that these assumptions of laminar flow are not justified.

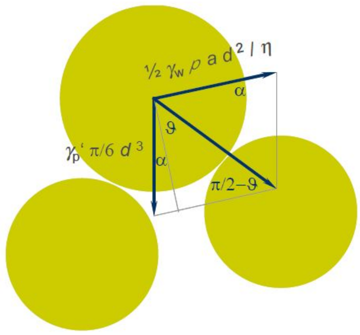


Figure 3.3: The balance of a top grain

The condition of limiting stress state imposes a balance between the forces on the top grain particles. J. Sellmeijer (2006) supposed a balance between the shear stress force along the sloping channel and weight of a spherical particle, see Figure 3.3.

3.2.3 Limitations

D-Geo Flow is a recently developed software by Deltares. As a consequence, the software has still some initial flaws. Besides the initial flaws, there are also substantial limitations. The substantial limitations are mentioned below:

- The pipe can only be defined in the top of the aquifer directly beneath the levee body. Besides, the model is not able to calculate vertical erosion. So the results of progression of the pipe are only reliable, if it is likely that the pipe progresses in a straight line.
- The software has a relativity simple user-interface which makes it easy to operate. However, implementation of complex problems becomes time-consuming or even impossible.
- The software is a kind of a black box. One can view the final results, but calculations performed by the model are not visible. Consequently, errors are difficult to detect or can not be observed.
- The model can only performs 2D groundwater flow calculations. So, the effect of variation over length are excluded in the calculations. Kanning (2012) recently showed that these variation over length assuredly influences the progression of the piping.

3.3 Input parameters

In order to perform a calculation in D-Geo Flow a couple of input parameters are required. This section outlines the most important input properties.

3.3.1 Soil parameters

The soil properties are the essential input parameter in a seepage flow problem. It is possible to add every type of soil in D-Geo Flow. As is stated before, D-Geo Flow calculates the flow through the saturated porous medium with a subsurface flow equation, see section 3.2.2. The following properties may to be inserted in order to perform a calculation:

- hydraulic conductivity, K [m/day]
- 70%-fractile of the grain size distribution, d_{70} [m]
- porosity, n [-]
- compressibility, α [m^2/N]
- submerged particle density, ρ_s [kg/m^3]
- White's constant, η [-]
- bedding angle, θ [°]

The hydraulic conductivity should be inserted separately for the x- and y-direction. In that way, anisotropic properties of soils can be expressed.

It should be noted that **not** all parameters of each soil type will be used in the calculation, since the pipe equilibrium expression is only applied in the soil directly beneath the levee body. As a result, the grain diameter, White's constant and the bedding angle are only required for the upper aquifer layer. In addition, one can perform a *steady state* or a *transient* calculation. A steady state calculation is a calculation without storage of water, which means that the groundwater is time independent. A steady state calculation can be performed by setting the compressibility of all soil materials to 0.

Several soil types with potential corresponding properties are deduced from Table B.1. These properties are in not yet validated, since the main aim of this chapter is to set-up a standard model. The impact of the supposed parameter and boundary conditions are examined in Section 3.6. The indicative soils are depicted Table 3.1. If the value of the parameter is not applied in the D-Geo Flow model, then a dash is depicted. The levee and blanket materials are assumed to be (very) impermeable.

Table 3.1: Soil types with potential properties, see B.1

soil type & color		hydraulic conductivity [m/day]	grain size [m]	porosity [-]	compressibility, α [m^2/N]
homogeneous					
sand	coarse	25	$8.0{\cdot}10^{-4}$	0.38	$1.0{\cdot}10^{-8}$
	medium	5	$3.0{\cdot}10^{-4}$	0.39	$1.0{\cdot}10^{-7}$
	fine	2	$1.5{\cdot}10^{-4}$	0.37	$1.0{\cdot}10^{-7}$
gravel		$2.5{\cdot}10^{2}$	-	0.32	$1.0{\cdot}10^{-9}$
silt		10^{-1}	-	0.48	$1.0{\cdot}10^{-6}$
clay		$2{\cdot}10^{-3}$	-	0.45	$1.0{\cdot}10^{-6}$
peat		$3.5{\cdot}10^{-1}$	-	0.88	$1.0{\cdot}10^{-5}$
slightly heterogeneous					
sand	silty	1	$6.3{\cdot}10^{-5}$	0.4	$1.0{\cdot}10^{-6}$
	clayey	0.5	$6.3{\cdot}10^{-5}$	0.4	$1.0{\cdot}10^{-6}$
Others	levee	0.01	-	0.4	$1.0{\cdot}10^{-7}$
	blanket	0.002	-	0.4	$1.0{\cdot}10^{-6}$

The particle density is the density of the solid particle that collectively represent the soil sample. The common range of soil is 2.55 g/cm^3 to 2.70 g/cm^3 (Chesworth et al., 2008, p. 504). A value of 2.65 g/cm^3 is generally used, exceptions being made where a great accuracy is required or where soils are known to deviate highly from the common range. The particle density is not a dominant parameter in the piping progress. As a result, a general value of 2.65 g/cm^3 is justified (Table 3.2).

Practical recommendations based on Sellmeijer's formula regarding parameter selection are provided by TAW (1994). The practical recommendations are developed in order to show how uncertainties can be discounted in the parameter selection. Table 3.2 depicts the recommended representative values of the White's constant and bedding angle. These input values are, if required, the same in each soil.

Table 3.2: Representative parameter values used in D-Geo Flow

parameter	representative value	unit
White's constant, η	0.25	-
bedding angle, θ	37	°
submerged particle density, ρ_s	$2.65{\cdot}10^{3}$	kg/m^3

3.3.2 Water properties

The water properties are already assigned in D-Geo Flow, see Table 3.3. These values correspond to fresh water. Fresh water is naturally occurring as groundwater in an aquifer, so these values are justified.

Table 3.3: Water properties used in D-Geo Flow

water type	compressibility, β [m^2/N]	density, ρ_l [kg/m^3]	viscosity, μ_l [(N·s)/m^2]
fresh water	$5{\cdot}10^{-10}$	$1{\cdot}10^{3}$	$1.3{\cdot}10^{-3}$

3.4 Standard configuration

This section outlines the implemented geometry of a levee and the applied boundary conditions. The geometry of the levee should correspond to the geometry of a standard levee in order to verify the model with Sellmeijer's design. The piping rules of (J. Sellmeijer, 2006) are also obtained by calculation of a conceptual model that considers a levee with a standard geometry. The conceptual model is a groundwater flow program that is described by the boundary conditions. This conceptual model contains a special iteration module to make predictions for piping design. The boundary conditions should be applied in such a way that the model approaches the physical process of groundwater flow.

3.4.1 Geometry of the levee

The seepage length is an essential parameter for assessing a levee against piping. The seepage length is directly dependent on the geometry of the levee. Because of this, the establishment of the geometry is a crucial step in the model set up. As is stated before, the geometry should correspond to the geometry of a standard dike in order to verify the model. So, the geometry of the levee consist of at least a horizontal impermeable inland blanket and clay levee on top of a sandy homogeneous aquifer. The dimensions of the geometry are derived by taking the mean dimensions of several geometry of levees located in the Netherlands. The geometries are extracted from the `HKV profielengenerator`. The `HKV profielengenerator` generates levee geometries on basis of the Actueel Hoogtebestand Nederland(AHN), which is a file that contains the elevation data of the Netherlands. The selection of the geometry is not completely randomized, because piping is not likely by a geometry with a long seepage length. The following conditions are presumed by the selection of the geometries:

1. Piping sensitive region.
2. Absence of relative wide inland berm and wide foreshore, since these structures significantly increase the seepage length and thereby reducing the likelihood of piping.

On basis of Table 2.6 and the `HKV profielengenerator`, three geometries of levees are selected within levee path 43_6. The three selected geometries and the mean geometry are depicted in Figure 3.4

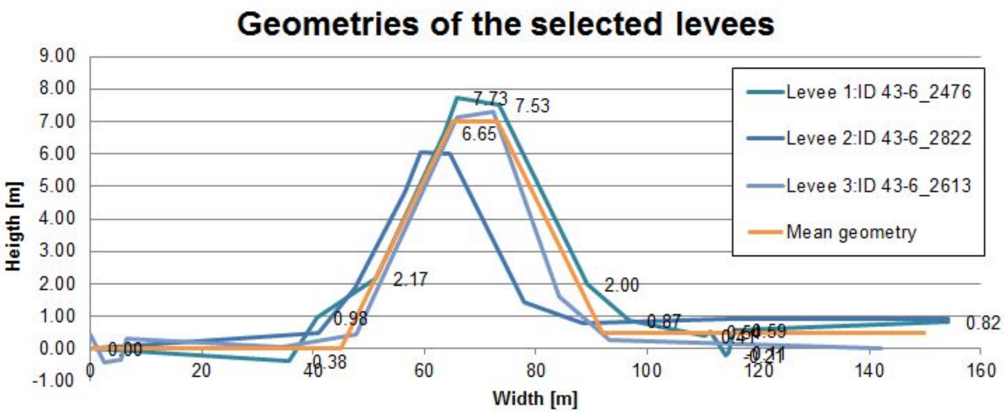


Figure 3.4: Geometries of the 3 selected levees and the mean geometry

The geometry or cross section of the levee can be created by adding several boreholes. The boreholes can be composed of different layer with a minimum thickness of 0.1 m. It should be noted that very thin layers will induce grid errors. Besides this, simulations whereby the geometry consists of very small layer are time-consuming, since a fine grid is required. Once the boreholes are defined a cross section can be generated in D-Geo Flow. D-Geo Flow linearly interpolates between the boreholes to generate a cross section. Figure 3.5 shows the added boreholes and Figure 3.6 shows the generated cross section. In this case, a homogeneous subsoil is added. An overview of the dimensions is given in Table 3.4. Furthermore, it is more likely that the critical head can be observed in deeper aquifer. As a consequence, an aquifer with depth of 30 m is considered. The influence of the aquifer depth on the piping process was already mentioned in section 2.3.2.

As is stated before, this thesis focusses on the backward erosion piping process. This process only develops when the uplift and heave criteria are exceeded. So there will be a (sand) boil or exit. The exit is realized in the model by implementing a small ditch at the toe of the levee, see Figure 3.6. In Figure 3.7 is zoomed at the exit point. The influence of exit configuration in elementary model has been investigated in the sensitivity analysis.

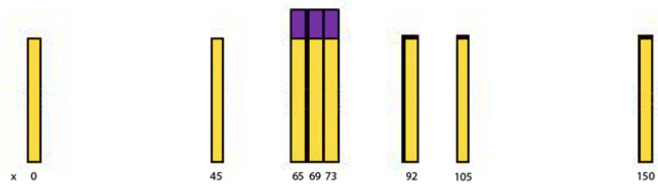


Figure 3.5: Added boreholes in D-Geo Flow

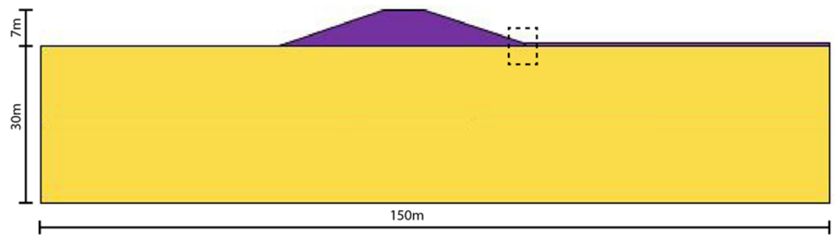


Figure 3.6: The generated cross section in D-Geo Flow

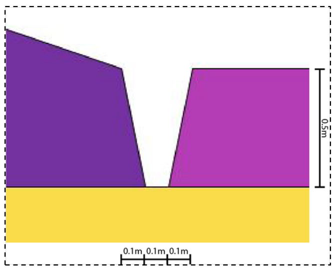


Figure 3.7: The exit configuration constructed in D-Geo Flow

Table 3.4: Dimension of the (standard) geometry of a levee

dimensions	value [m]
aquifer	
height (h_a)	-30
length (l_a)	100
levee	
height (h_l)	7
bottom length ($l_{l_{bottom}}$)	47
crest length ($l_{l_{crest}}$)	8
blanket	
height (h_b)	0.5
length (l_b)	58
exit	
length (l_e)	0.1

3.4.2 Boundary conditions

The boundary conditions are required to describe the model and represent the influence of the outside world. Two common types of boundary conditions are the **Dirichlet** and **Von Neumann** conditions. As stated before, a Dirichlet condition usually corresponds to setting the value, whereas a Von Neumann condition usually specifies a flux condition on the boundary. In ground-water problem, the solution is usually expressed in terms of head (h), since the head is usually the dependent variable in the governing partical differential equation (Franke, Reilly, & Bennett, 1987). This section describes five types of external boundary conditions; a constant head, a specified head, a streamline (no flow), specified flux and free surface or seepage surface boundary condition. These boundary can also be assigned in D-Geo Flow. However, the boundary condition can have different naming. The D-Geo Flow names are depicted in italic and between brackets.

1. **Constant head boundary** (*head boundary condition*)
   The hydraulic head (h) in a seepage system is the sum of the elevation head (z) and pressure head ($\frac{p}{\gamma_w}$), where p is the pore pressure and γ_w is the unit weight of water. Physically, the hydraulic head represents the water level above datum in the point of question. The constant head boundary occurs where a part of the boundary surface of a aquifer system coincides with an constant head. The word 'constant' implies that the value is uniform in space as well as in time (Franke et al., 1987).

2. **Specified-head boundary** (*submerged boundary condition*)
   The specified head boundary is a more general type of boundary condition and occurs wherever the head can be specified as a function of position and time. In a river or a channel the head can change over time due to, for example, difference in discharge. In this case, the water level at specific point along channel can be specified as a function of time, $h(x_n, t) = f(t)$. In more complex (3D) situations, the head changes in relation to the slope of the channel as well as with time. The head at a point along the streambed would be specified as a function of both position and time, $h = f(x, y, t)$ (Franke et al., 1987). A specified-head boundary often occurs at the submerged side of a levee.

3. **Streamline or stream surface boundary** (*no flow boundary*)
   A streamline is a curve that is tangent to the flow-velocity at every point along its length. No flow exits, where the flow components are normal to a streamline and no flow crosses a streamline (Franke et al., 1987). For some parts of a boundary the stream surface remain fixed over time. An example of a fixed stream surface is an impermeable boundary. Despite of the fact that natural materials are never completely impermeable, they may be modelled as impermeable, when their hydraulic conductivity differs by several the order of magnitude (lower) with respect to the aquifer (Franke et al., 1987). The no flow condition requires that q_s is equal to zero. This can only be satisfied when $\frac{\delta h}{\delta x}$, the derivative of the head is zero, see Equation 2.9.

4. **Specified flux boundary** (*flux boundary condition*)
   The specified flux boundary occurs wherever the flux across a given part of the boundary surface can be specified as a function of position and time. The term 'flux' refers to the volume of fluid crossing a unit cross-sectional surface area per time (Franke et al., 1987). The simplest type of specified boundary also known as constant-flux boundary considers a uniform flux both in space and time, $\frac{\delta h}{\delta x} = constant$. In the special case that the flux is zero, this boundary is equal to stream surface boundary. In more general cases the flux can be function of position $\frac{\delta h}{\delta x} = f(x, y)$ or time $\frac{\delta h}{\delta x} = f(t)$ or a function of both positions and time $\frac{\delta h}{\delta x} = f(x, y, t)$.

5. **Free-surface or seepage surface boundary ($h = z$, or more generally $h = f(z)$)** (*seepage boundary condition*)
   The surface of seepage boundary condition occurs at boundary between the saturated flow field and the atmosphere along which ground water discharge. A seepage surface is always associated

with a free surface. The pressure at the water table is atmospheric. So, the total ground water head at the water table is equal to the evaluation head (Franke et al., 1987).

The common designations of the first four boundary conditions are summarized in Table 3.5.

Table 3.5: Designation for several boundary conditions (Franke et al., 1987)

boundary condition name	formal name	mathematical notation
constant head (1) specified head (2)	Dirichlet	h =constant $h = f(t)$
stream surface (3) specified flux (4)	Von Neumann	$\frac{\delta h}{\delta x} = 0$ $\frac{\delta h}{\delta x} = f(t)$

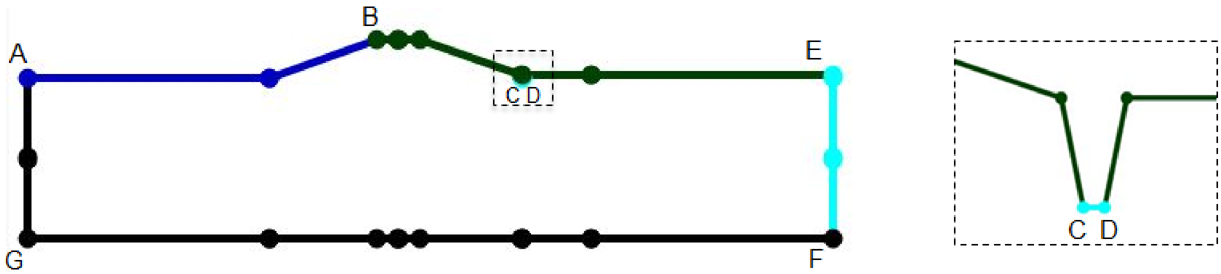


Figure 3.8: Assigned boundary conditions in D-Geo Flow

The boundary conditions assigned to the model are depicted in Figure 3.8. It is assumed that ground water flow is at rest when z=0 and h=0 and is reference level. The line AB is the submerged river side of the levee, so a specified head boundary is assigned, see Figure 3.9. The water level or head will increase with time, such that it represents a river with changing discharge. The gradient of the head is manually reduced, when the head approaches the critical head or the height of the levee. This reduction of slope is applied to determine the critical head more accurate without introducing more time steps, since the $\frac{\delta h}{\delta t}$ will be smaller.

The lines BC and DE are assigned as seepage or free-surface boundaries. However, seepage of water do not occur at the surfaces, since the underlying materials (levee and blanket) are very impermeable. The line CD is the exit point of the model. The line CD is at the top of the aquifer where z=0. The hydraulic head is also 0 such that the ground water can flow out. The line is assigned as with a constant head boundary with h=0. The line GF is at top of a very impermeable layer. So, it is assumed that no flow crosses the boundary.

The lines EF and AG do not correspond to physical boundaries. The natural flow system may extend beyond these boundaries, perhaps for a considerable distance. The questions are; where should these boundaries conditions be located? and which condition should be assigned? Based on experience with general groundwater problems Franke et al. (1987) described the following: the distance to lateral boundaries should be at least three times the depth of the flow system and further increase in distance has only a slight influence on the potential distribution near the structure. In the assignment of the boundary condition, it is assumed that water flows from a specified head toward a constant head boundary. The AG line is therefore assigned as a stream surface or no flow boundary. At line EF is assumed that the body of water is at rest, so there will be hydrostatic pressure. The hydraulic pressure is created by assigning a constant head with h=0. In addition, the selection of boundary conditions involves simplification of the actual hydrological conditions which can induce errors. The effect of boundary conditions on the model response is evaluated in the sensitivity analysis. Table 3.6 provides an overview of the assigned boundaries.

Table 3.6: Assigned boundaries per line in D-Geo Flow

line	boundary type
AB	specified head (2)
BC	free-surface or seepage (5)
CD	constant head (1)
DE	free-surface or seepage (5)
EF	constant head (1)
FG	stream surface or no flow (3)
AG	stream surface or no flow (3)

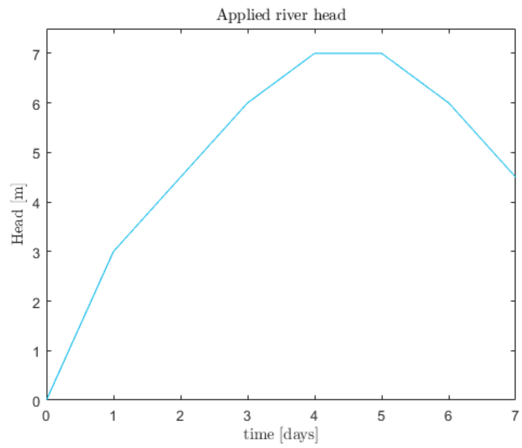


Figure 3.9: Assigned river head boundary

3.5 Verification

The previous sections describe the initial settings of the model. The initial settings are in accordance with the standard configuration proposed by J. Sellmeijer (2006), in order to verify the outcomes. The verification is carried on basis of a comparison between both the adopted design rules of Sellmeijer (Section 2.2.3.5) and the orginal design rules of Sellmeijer (Appendix A). As mentioned before, the design rules are derived by curve fitting a number of specific numerical results performed in MSeep. The original design rules were developed by assuming a 2D field. The 2D approach seems to suited for smaller particles. However, for larger particles, the channels tend to have a restricted width. This implies a smaller critical head due to increased discharge in the channel. On behalf of this conclusion, H. Sellmeijer et al. (2011) has adopted the design rules.

In order to verify the model, several simulations were performed in which the hydraulic conductivity(K) and grain size diameter (d_{70}) are varied. The values of these parameters must correspond to sand, otherwise the design rules are not valid. The results of the simulations are summarized in Table 3.7. The first ΔH_c is the critical head calculated in D-Geo Flow, the second ΔH_c is critical head calculated with the adopted design rules and the third ΔH_c is the critical head calculated with the original design rules. The difference(dif.) between the outcome of the model and the outcome of the design rules are given in m and/or %. If the deviation is less than or equal to 0.5 m or 10% the cell is coloured green, otherwise the cell is coloured red.

Table 3.7: The results of the verification per simulations

			verification					
run data			model	adopted design rules			original design rules	
number	d_{70} [m]	K [m/day]	ΔH_c [m]	ΔH_c [m]	dif. [m]	dif. [%]	ΔH_c [m]	dif. [m]
1	3.00E-04	3	6.0	6.27	0.27	4.50	7.76	1.76
2	2.00E-04	5	4.1	4.49	0.39	9.51	4.65	0.55
3	2.00E-04	10	3.2	3.57	0.37	11.56	3.78	0.58
4	4.00E-04	10	4.4	4.71	0.31	7.05	7.01	2.61
5	1.00E-04	2	4.8	4.62	-0.18	-3.75	3.28	-1.52
6	1.00E-04	10	2.6	2.7	0.1	3.85	2.02	-0.58
7	1.00E-04	1	6.6	5.82	-0.78	-11.82	4.04	-2.56
8	8.00E-04	25	4.2	4.58	0.38	9.05	9.89	5.69
9	3.00E-04	5	4.9	5.29	0.39	7.96	6.67	1.77
10	3.00E-04	25	2.9	3.09	0.19	6.55	4.13	1.23
				Total	0.144	4.45		0.953

Although a total deviation of approximately 0.15 m or 5%, it is concluded that the model corresponds sufficiently to adopted design rules. The deviation of the model with respect to the design rule can be clarified as a consequence of the following aspects:

- **Curve fitting**
  The design rules are obtained by calculation by a conceptual model. After which a number of specific results is curve fitted and collected into rules (J. Sellmeijer, 2006). Curve fitting is process of constructing a mathematical function, which is the best fit to the specific result. As a consequence deviations of the design rule are inevitable.
- **Default setting model**
  D-Geo Flow is implemented with functions such as MPicard, Errlin and ErrNonlin. These functions can slightly influence the outcomes.
- **Boundary conditions**
  The setting of boundary conditions may be different and may induce small deviations. For example, a transient calculation or steady state calculation can be performed.
- **Grid definition**
  The definition of the grid determines both the duration of the calculation and the accuracy of the model. However, a fine accurate grid is time-consuming, whereas coarse grid is quick. In addition, model with coarse grids can becomes unstable resulting wrong or inaccurate solutions. So a consideration is made between the accuracy and the duration of the calculation. This consideration may also induce small deviations.

In addition, all outcomes are obtained by calculation by a (conceptual) model. A model is an approximation of reality. Due to the approximation, the model may encounter the following errors which can also cause difference in results. The errors are listed on the basis of their relevance.

- **Truncation error**
  A truncation error occurs when an infinite sum needs to be approximated by a finite sum.
- **Human error**
  Human are involved in programming, input preparation and output preparation and can induce errors.
- **Rounding error**
  Round-off error occurs because digital computers use floating numbers of fixed length.
- **Measurement error**
  Measurement errors occur if external data is measured imperfectly.

3.6 Sensitivity analysis

This section outlines how the parameter, geometry and the boundary conditions choices influence both the critical gradient and the hydraulic head underneath the cover layer. The mechanism piping consists of three consecutive phase: uplift, heave and backward erosion piping. The next phase can only be executed if the criterion of the previous phase is met. The hydraulic head underneath the cover layer determines whether uplift and heave occur, while the critical head determines whether the backward erosion process becomes progressive so that the levee will eventually collapse. An overview of these relations is given in Figure 3.10. The hydraulic head and critical head are correlated and therefore connected by a dashed line. However, the degree of influence can significantly differ due to the boundary condition along the erosion channel used to determine the critical head. This boundary condition has a highly non-linear behaviour and depends also on additional resistance parameters such as the bedding angle and particle size.

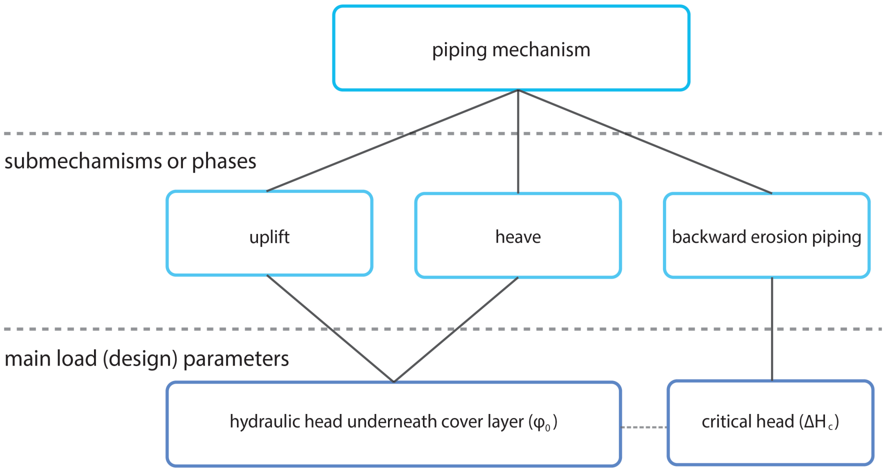


Figure 3.10: The main load (design) parameters for piping

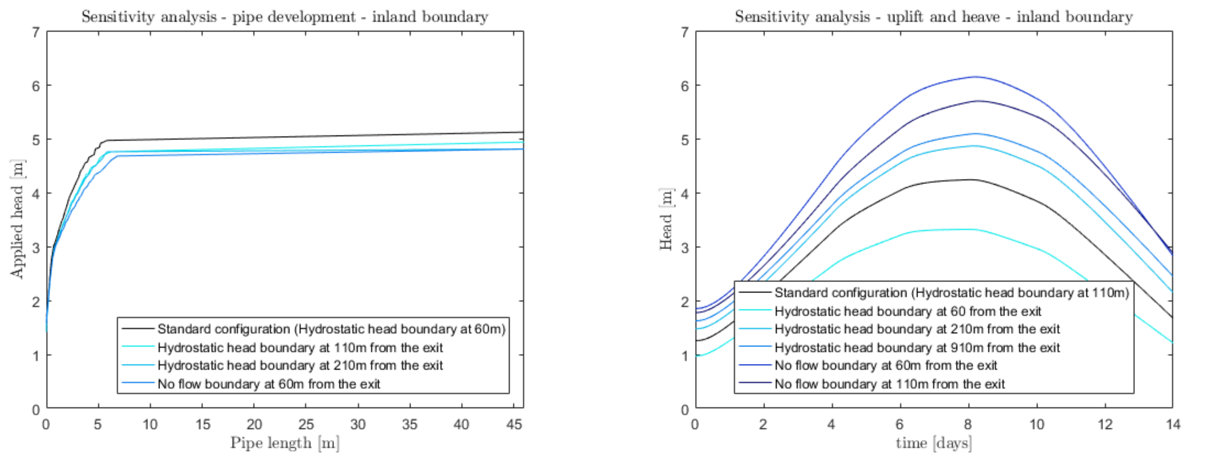
The influence of several assumed parameter, geometry and boundary conditions choices are observed. An assumption is observed if it is known or expected that; the assumption will highly influence the result and/or the assumption is quite difficult to determine in practice. The sensitivity of (influential) factors that are already investigated in previous research are not observed in this thesis, except the exit configuration. The influence of following three assumptions/choices are observed:

- The type and location of **inland boundary**.
- The thickness and hydraulic conductivity of the **blanket or cover layer**.
- The configuration of the **exit**, especially its length

The observations of these three considerations are outlined separately in the following sections. As previously described, a distinction is made between uplift and heave, and pipe development.

3.6.1 Inland boundary

Two type of boundaries can be assigned to inland side of the model. Namely, a constant head boundary or a no flow boundary. A constant head boundary needs to be assigned if it is supposed that the groundwater in the subsoil has reached an stable equilibrium so that the pressure is hydrostatic. A no flow boundary needs to be assigned if a transition from an aquifer to an aquitard can be observed at the presumed location. Nevertheless, it is hard to determine both conditions in practice. Due to this complication, a hydrostatic head boundary is assumed at the inland side in the standard configuration. After which the influence of the boundary conditions are observed and compared with the standard configuration. In Section 3.4.2 was outlined that the distance of the boundary may also highly influences the outcome, especially when the distance is less then three times the depth of flow system. This phenomenon was also observed during the exploration. Namely, it was observed that the distance from the inland boundary to the (presumed) exit highly influence the outcome of model, especially if the hydraulic head underneath the cover layer. Therefore, distance to the inland boundary is enlarged in the standard configuration if the hydraulic head underneath the cover layer was observed. In this case, the inland hydrostatic head boundary is assigned on a distance of 110m from the exit instead of 60m. Several simulations are performed, whereby the condition and the distance of the boundary are changed.



(a) The sensitivity of the inland boundary on critical head (pipe development)

(b) The sensitivity of the inland boundary on the hydraulic head at presumed exit (uplift)

Figure 3.11: The sensitivity of the inland boundary

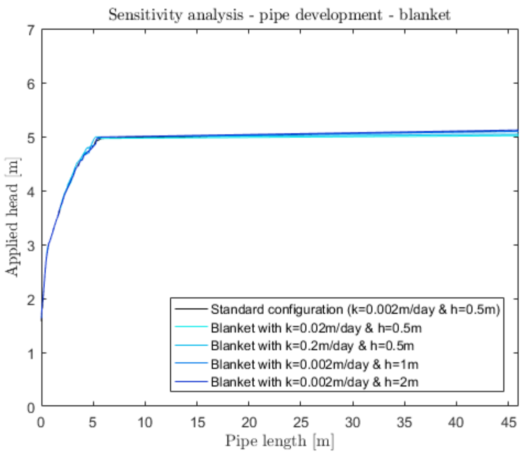
Figure 3.11a shows the pipe development plotted against the applied head. The critical head or critical gradient is the applied head where the pipe length suddenly progress almost horizontal to its maximum. Figure 3.11b shows the hydraulic head underneath the cover layer plotted against time. The applied river water level is given in 4.12. An interesting difference is observed. The influence of the boundary conditions on the critical head is quite limited, whereas the influence on the hydraulic head is substantial. This difference occurs because the calculations are performed with a different geometry. The critical head can only be calculated if exit is constructed or a hydraulic fracture is defined. The boundary assigned to the exit dominates the ground water model so that the influence of the inland boundary is limited. In this situation, most of the water flow is forced towards the exit point instead of flowing to the inland side of the geometry.

However, if the hydraulic head is calculated, the exit is absent and water flows towards the inland side of the geometry. The hydraulic head underneath the cover layer is therefore very dependent on inland boundary. In addition, the hydraulic head is also dependent on the leakage of the blanket and leakage length. The potential hydraulic head underneath cover layer can be determined by means of the hydraulic head course or phreatic line in the aquifer. The hydraulic head course can be calculated with the Dupuit-flow equations. The leakage of the blanket is dependent on its thickness and its hydraulic conductivity. The effect of leakage of the blanket is outlined in next section.

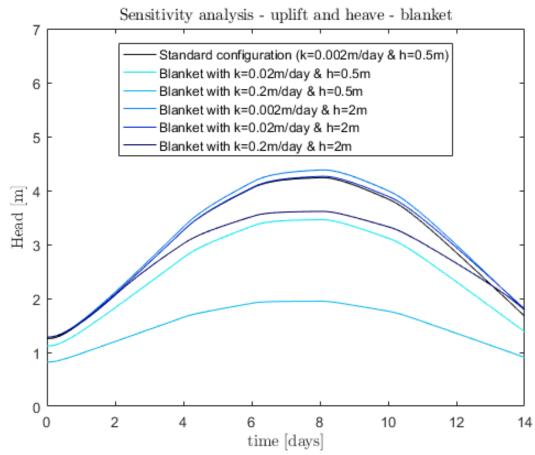
In Figure 3.11b, it is clearly visible that if no flow boundary was assigned, higher hydraulic heads arise. The higher hydraulic occurs due to the reflection of water at the boundary resulting in higher pressure underneath the blanket. In addition, two processes appear depending on the assigned boundary. If an inland head boundary is assigned, the hydraulic head underneath the cover layer increases if the distance between distance the presumed exit and inland constant head boundary becomes longer. This creates a more gradual phreatic line. On the other hand, if no flow boundary is assigned the hydraulic head decreases if the distance becomes larger. Due to the increased distance, more water can leak to the enlarged blanket which result in lower hydraulic heads. These two processes seems to coincide so that there should be a location where the type of boundary condition does not matter. However, this location is hard to determine, since the phreatic line is also dependent on the leakage of the blanket. Besides, it is noticed that models with a no flow boundary run significant longer. To cover both issues, an constant head boundary condition at a sufficient distance to (presumed) exit should be assigned to the inland boundary.

3.6.2 Blanket or cover layer

As is mentioned in previous section, the hydraulic head underneath the cover layer is, besides the inland boundary, also dependent on the leakage through blanket. The leakage of the blanket is a function of both the thickness and the hydraulic hydraulic. If the blanket is permeable it allows more water to leak through the blanket as a result in lower hydraulic pressures occur underneath the blanket. The thickness of the layer determines, among other things, the degree of resistance that leakage water experience. It is more difficult for water to leak through a thicker blanket. The water experience more resistance so that hydraulic pressure arise underneath the blanket. The influence of the blanket is observed and shown in Figure 3.12a and Figure 3.12b. Both the hydraulic conductivity and the thickness of blanket are changed.



(a) The sensitivity of the blanket on the critical head (pipe development)



(b) The sensitivity of the blanket on the hydraulic head at presumed exit (uplift)

Figure 3.12: The sensitivity of the blanket or cover layer

It clearly visible that a change in both the hydraulic conductivity and the thickness of the blanket barely influence the critical head (Figure 3.12a). This negligible effect is caused by the presence of the exit. Just as described in previous section, the exit boundary dominates the model. Furthermore, the hydraulic conductivity and thickness are not incorporated in the boundary along the pipe. However, in the literature study was found that the hydraulic gradient may reduced by $0.3d$ due to the additional resistance of sand grains in the uplift channel. This effect can be D-Geo Flow by adding a internal boundary condition. This internal boundary condition is added to the realistic model.

In contrast to the critical head, the hydraulic head underneath the cover layer is highly influenced by the permeability and thickness of the layer. In addition, Figure 3.12b shows that the hydraulic head underneath the layer can be the same unless the permeability and thickness are different. For instance, a blanket with a permeability of 0.02 $\frac{m}{day}$ and a height or thickness of $2m$ result in the same hydraulic head as a blanket layer with a permeability of 0.002 $\frac{m}{day}$ and a height or thickness of 0.5m. The permeability and the thickness of the blanket are negatively correlated. For this reason, it is difficult to come up with a universal blanket. However, the effect of the thickness the and permeability of the blanket is kept in mind during this thesis.

3.6.3 Exit configuration

Figure 3.7 depicts the dimensions of the exit of the standard configuration. The exit is a very small ditch with very steep walls. The height of the ditch is directly dependent on the height of the blanket. The influence of the blanket is discussed in the previous section. The walls are as steep as possible to represent a bursting channel. The parameter that is adjusted is the length or diameter of the exit. In a 2D problem, the length of the exit is similar to the diameter. The exit of the standard configuration has a length of 0.1 m.

The length is enlarged in order to observe the influence of a larger outflow area. The results are depicted in Figure 3.13. Logically, the changes in critical head are observed only, since a ditch or sand boil implies that uplift and heave occurred.

In Figure 3.13, it can be observed that the critical head increases by enlarging the outflow area. Due to a larger outflow area, the convergence of the seepage flow is less resulting in lower local pressures and a higher critical head. The influence of the exit configuration was also researched by (Robbins & van Beek, 2015). Nevertheless, the influence of the enlargement seem to be limited in this study. Since, a 50 times larger exit increases the critical head by only 3%.

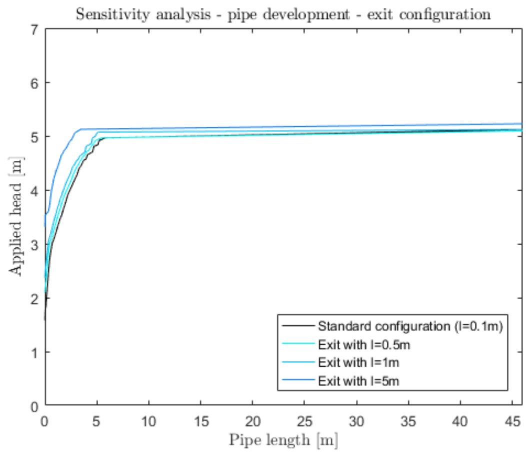


Figure 3.13: The sensitivity of the exit configuration on the critical head

3.6.4 Summary

This section summarizes the main conclusions of the sensitivity analysis, hereby a distinction is made between pipe development and, uplift and heave.

- **Pipe development**
  The influences of the assumptions are limited or even negligible. Because that the model is dominated by the exit boundary. Due to the exit boundary, water is forced to flow towards the exit instead of flowing towards the inland boundary.

- **Uplift and heave**
  Contrary to observations regarding pipe development, the assumptions significantly influence the hydraulic head underneath the cover layer. In general, a no flow boundary and a thick and impermeable blanket increase the hydraulic head, while a inland constant head boundary and a small and permeable blanket decrease the hydraulic head. Therefore these assumptions should be used with care. The purpose of the chapter determines which degree of accuracy is required and it therefore determines the accuracy of the parameter, geometry and boundary conditions selection. A model with a narrow grid will be very accurate, but also time-consuming. On the other hand, a model with very coarse grid can become unstable. Because of this, a consideration should be made between the accuracy and duration of the simulation.

4 | Elementary model

4.1 Introduction

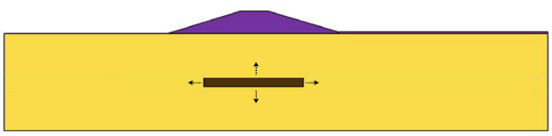
The previous chapter outlines the initial settings of the D-Geo flow model with the geometry of a standard levee. The geometry is in accordance with the standard configuration proposed by J. Sellmeijer (2006), so that the model could be verified according to Sellmeijer's design rules. Before the verification, several soil types with corresponding properties were deduced from the literature. These soil types serve as the main input parameters in this chapter. After the verification, it was concluded that the basic D-Geo Flow model sufficiently corresponds to the design rules. Despite the observation that some parameters, geometry and boundary conditions choices influence the outcome, the standard configuration was used as the starting model for elementary models. However, the influences of crucial assumptions will be taken into account in the realistic model.

The purpose of this chapter is to research the effect of anisotropy and heterogeneity in a broader context by using an elementary model. The elementary model is an customized version of the standard configuration model to enable aquifer modification. This research was carried out by implementing anisotropy ($K_x \neq K_y$) or artificial elements with varying soil types. In addition to varying subsoil types, the effect of widening, extending or moving these elements has also been observed. Section 4.2 discusses the basic aquifer compositions that have been investigated. As stated above, the piping mechanism consists of three phases: uplift, heave and pipe development. Pipes can only develop if uplift and heave have occurred. The likelihood of both uplift and heave occurring depends on the hydraulic pressure (head) underneath the cover layer, while the likelihood of pipe development is defined by the critical head (Figure 3.10). The change in both parameter values due to the addition of anisotropy and heterogeneity are observed separately. Firstly, the change in critical head with respect to standard configuration is observed in Section 4.3, hereby it is assumed that uplift and heave have occurred. Secondly, the change in hydraulic head at the bottom of the cover layer(ϕ_o) was investigated, see Section 4.4. The physical processes or mechanisms that cause the changes in both the critical hydraulic head and the hydraulic head at the bottom of the cover layer are outlined as well. The chapter ends with general conclusions on the elementary study on the basis of the results.

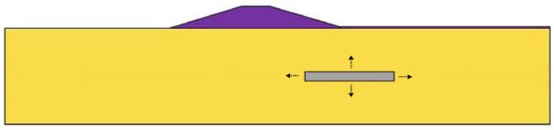
4.2 Aquifer compositions

As mentioned above, the aquifer compositions are created by implementing artificial elements with varying soil types or by changing soil properties, especially concerning the addition of direction dependency (anisotropy). In order to properly determine the effect of conductors and resistances in the aquifer, very permeable gravel layers and very impermeable clay layers are added. All other soil properties remain equivalent to the properties assumed to hold in the previous chapter. The main aquifer material is still medium sand, see Section 3.3 for the corresponding properties. An enormous number of aquifer compositions could be considered and constructed. However, the aim in this phase of the thesis is to research the effect of anisotropy and heterogeneity in a broader context. Consequently, a consideration of several expressive aquifer compositions is sufficient. The following four aquifer compositions are considered:

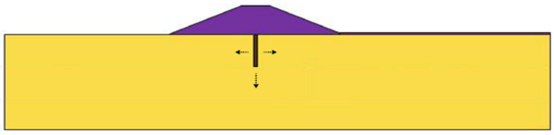
1. A horizontal layer in the centre of the aquifer



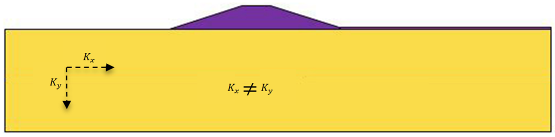
2. A horizontal layer beneath the exit point



3. A vertical layer beneath the levee body



4. An anisotropic aquifer



The aquifer compositions are separately described in the following subsections.

4.2.1 A horizontal layer in the centre of the aquifer

A horizontal layer located in the centre of the aquifer will hinder or conduct seepage flow depending on its properties. For instance, an impermeable clay layer will bifurcate the flow. As a consequence, the flow underneath the clay layer cannot flow directly towards the exit point. But the flow is stimulated to flow in an almost horizontal direction towards the inland boundary. As a consequence, less water converges at the exit point resulting in lower local pressure gradients. On the other hand, a very permeable gravel layer conducts water. As a result, more water flows towards the exit point thereby inducing a higher local pressure gradient. In addition, the effect will be amplified when the dimensions of the layer are larger. Larger clay layers reduce the flow area and increase the resistance, whereas larger gravel layers allow more water to flow through the aquifer towards the exit. Several aquifer compositions with a horizontal layer in the centre are examined. After each simulation, one parameter is changed, while the rest remain constant. The following parameters are changed: the height, the length and the soil material.

4.2.2 A horizontal layer beneath the exit point

It is supposed that an intriguing process will occur if a highly permeable gravel layer is situated beneath the exit point. The seepage will not only flow towards the exit point , but will also be deflected towards the very permeable gravel layer, since water prefers the path of least resistance. The water that flows into the gravel layer will be conducted towards the inland boundary condition. As a consequence, less seepage flow converges at the exit point so resulting in local lower pressure gradients. A very impermeable layer beneath the exit point will initially hinder or block some water from flowing towards the exit. However, if the length of the pipe exceeds the impermeable layer, the water is no longer blocked. From that moment onward, the impermeable clay layer will barely affect the local pressure gradient at the head of the pipe. This is true if the length of the clay layer measured from the exit point is much smaller than the critical pipe length. In this case it is supposed that the length of the layer measured from the exit point is much smaller then the critical pipe length. As a result, the effect of the clay layer is of less importance. Therefore it is only the effect of a gravel layer beneath the exit point that is examined.

4.2.3 A vertical layer beneath the levee body

A vertical layer can be compared to a screen. The effect of the vertical layer or screen is analysed by varying the position, thickness and length, and by thereby mainly focussing on the impermeable layer. It is supposed that such a layer will significantly hinder or block the flow and thereby modify the flow pattern. Besides, the vertical impermeable layer at the top of the aquifer hinders or blocks the progression of the pipe as well.

On the other hand, a vertical gravel layer allows more water into the aquifer so that higher local velocity can be observed. If these higher local velocities arise underneath the levee body, then they will accelerate the erosion piping process. Hence a vertical gravel layer located directly beneath the cover layer could be of significance. However, if a permeable vertical gravel layer is present in the top aquifer, piping failure is not likely. Due to the large grain diameter and weight of the gravel material, the gravel will not erode so that the pipe erosion process stops.

To summarize, it is supposed that interesting processes only occur when a vertical impermeable layer is located beneath the levee body. Accordingly, the effect of the vertical permeable layer is not investigated.

4.2.4 An anisotropic aquifer

An anisotropic aquifer is an aquifer in which the hydraulic conductivity alters depending on the direction. In other words, the hydraulic conductivity is directionally dependent. In Section 2.4.1 it was stated that anisotropic behaviour is principally characteristic for undisturbed alluvial material. In addition, de Wit (1984) observed that the vertical hydraulic conductivity can be a factor of 100 or even more lower than the horizontal conductivity. It is expected that anisotropic behaviour will greatly influences the critical hydraulic gradient, since less water will be flowing vertically into the aquifer, so there is less water supply from the deeper layers of the aquifer. In order to analyse the effect of anisotropic behaviour, the vertical hydraulic conductivity is assumed to be a factor of 10 lower, which is in accordance with the literature.

4.3 Results of the critical head

This section discusses the results of each different type of aquifer composition separately. All aquifer compositions are plotted against the standard configuration to analyse the change in the critical (hydraulic) head. If the aquifer composition is more resistance to piping, the critical head gradient will be higher. In other words, a higher critical gradient is advantageous. The prospects of each aquifer composition were outlined in the previous section. If the prospects were correct, the results are briefly clarified. Otherwise, a more comprehensive clarification is provided. The clarification is sometimes supported by figures. The results are summarized in Table 4.2, whereby the differences in the critical head are expressed in terms of percentage relative to the standard configuration.

4.3.1 A horizontal layer in the centre of the aquifer

As mentioned in Section 4.2.1, three characteristics of the implemented layer are changed; the soil material, the length or the height. One parameter was adjusted after each simulation, while the remaining the parameters remained constant.

4.3.1.1 Varying soil material

Three simulations with different soil types were performed, while the length and height were kept constant. The implemented soil types are: a very permeable gravel(G) layer, a very impermeable clay(C) layer and a moderate permeable sand silty(Ss) layer. The length and height of the layer are 18 m and 5 m The results are depicted in Figure 4.1. The length of the pipe(x-axis) is plotted against the applied head (y-axis). It is clear that increase in head is compensated by a small increase in length until the critical head is reached. The critical head is the head at which the pipe suddenly progresses to its maximum length, which ,in this case, is 47 m. The legend displays abbreviations of the aquifer compositions. The abbreviation is structured as follows. The first upper-case letter, which can be followed by a lower-case letter, represents the soil material of the layer. The second upper-case letter represents the orientation of the layer (horizontal or vertical). The third, and if necessary the fourth upper-case letters represent the position of the layer. These upper-case letters are followed by the dimensions of the layer. For example, CHC L18 H5 is a clay(C) horizontal (H) layer in the centre(C) of the aquifer with a length(L) of 18 m and a height(H) of 5 m. This way of abbreviating is maintained throughout this chapter.

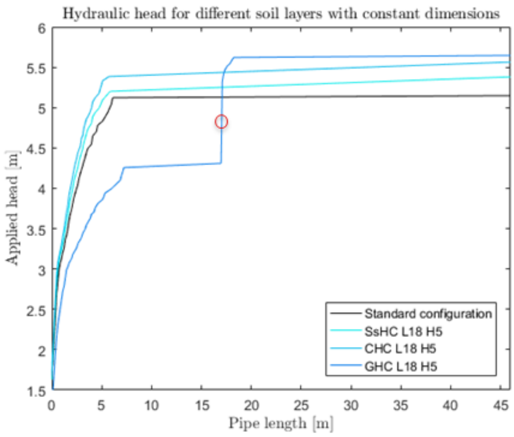


Figure 4.1: The critical head for a horizontal layer with varying soils

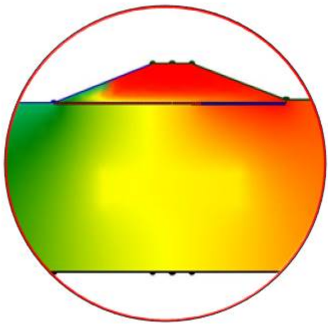


Figure 4.2: The hydraulic head distribution at the point of investigation in the aquifer with a gravel layer in the center

In Figure 4.1 can be observed that the critical head is increased by all aquifer compositions, even when a very permeable gravel layer was implemented. It was expected that a more permeable layer would allow more water to be conveyed to the exit or head of the pipe resulting in a lower critical head. In the gravel layer results it can be observed that the pipe tends to grow progressively in an earlier stage at an applied head of approximately 4.3 m. However, at a length of approximately 18 m the pipe reaches a new equilibrium. In order the clarify this phenomenon, it was investigated to what happened at the point where the pipe reaches a new equilibrium. This point of investigation is depicted with a red circle as shown in Figure 4.1. Figure 4.2 shows the hydraulic head in the aquifer at the point of investigation. It is clearly visible that the hydraulic head is not progressing in an almost vertical line to the exit, see Figure 4.11. The hydraulic head in the gravel layer is therefore higher than its surroundings, near and below the pipe. This implies that not all water is flowing towards the head of the pipe, but the water is also conducted horizontally through the gravel layer. As a consequence, less water is conveyed at the exit or head of the pipe resulting in a lower local pressure gradient. The pipe will be in equilibrium until the local hydraulic (pressure) gradient at the pipe is beneath the local maximum hydraulic (pressure) gradient. It should be noted that if the gravel layer is situated prior to the critical pipe length, then the gravel layer will not diverge the flow, but will allow more water to flow in the aquifer. As a consequence, the progressive erosion process will occur at lower hydraulic gradients. Accordingly, the effect of a gravel layer depends on its location. The effect of a gravel layer beneath the exit point is discussed in Section 4.3.2.

4.3.1.2 Varying dimensions

Several simulations were performed, whereby per simulation the length or the height of the layer was changed. The layer was composed of clay material and remained constant. It is supposed that the impact of an arbitrary layer on the piping process is positively correlated to its dimension. So, layers with larger dimensions have more impact. The degree of impact can be material-dependent. The results are depicted in Figure 4.3 and Figure 4.4. Figure 4.3 shows the applied head versus the length of the pipe for layers with heights of 1 m, 3 m and 5 m, while the length remains constant (18 m). Figure 4.4 shows the applied head versus the length of the pipe for a layer with lengths of 18 m, 45 m and 60 m, whereas the height remains constant (3 m).

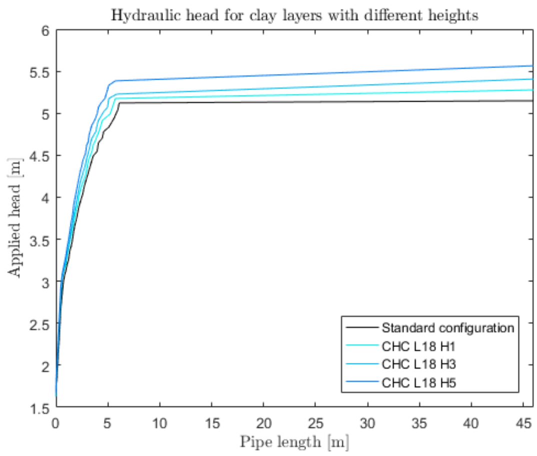


Figure 4.3: The critical head for an aquifer with horizontal clay layer with varying the height

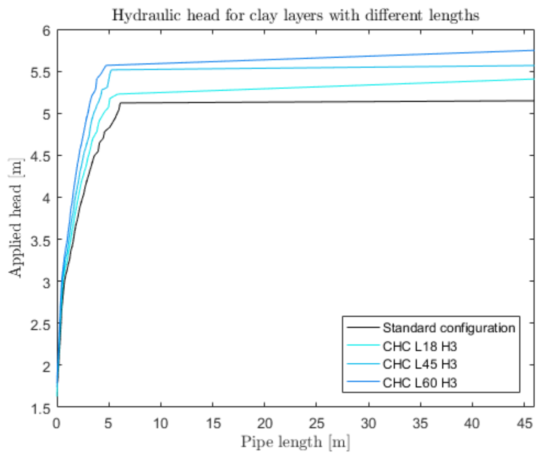


Figure 4.4: The critical head for an aquifer with horizontal clay layer with varying the length

It can been clearly seen that the critical head increases with either the height or the length of the layer. Both outcomes can be clarified. A higher clay layer reduces the flow area resulting in a limited flow of water towards the exit point. Due to the presence of a longer clay layer less water can flow towards the exit, since the water beneath the clay layer can hardly cross the layer. The water beneath the clay layer is stimulated to flow towards the inland boundary instead of being deflected towards the exit. As a consequence, less water converges at the exit point. Finally, it was concluded that the influence of the layer is positively correlated to its dimensions.

4.3.2 A horizontal layer beneath the exit

Two aquifer scenarios with a gravel layer beneath the exit were experimented with, whereby the height of the layer was enlarged. The results are depicted in Figure 4.5. The third and fourth upper-case letters (MR) in the abbreviation represent the location of the layer. The layer is located in the middle(M) on the right side(R) in the aquifer.

It can clearly be seen that the critical head is increased in both scenarios, while the overall hydraulic conductivity of the aquifer is higher. The mechanism that causes the increase in critical head is similar to the process observed in the aquifer compositions with a gravel layer situated in the centre of the aquifer. However, this process is initiated when the water flow approaches the exit. When the aquifer is homogeneous, almost all water flows towards the exit point. However, in this case the water also flows into the gravel layer due to its very high permeability. Because of the high permeability, the gravel layer produces less resistance to water so that the water partly flows into the gravel layer. As a result, the water flow is less concentrated near the exit resulting in lower local hydraulic gradients.

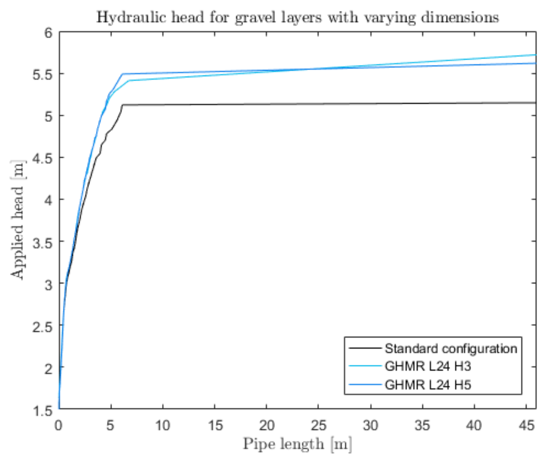


Figure 4.5: The critical head for an aquifer with horizontal gravel layer beneath the exit

In addition, a small increase in critical head can be observed. However, the lines almost coincide which suggests that the enlarge height barely influences the critical head. Based on this outcome, the influence of this specific layer seems to be limited.

4.3.3 A vertical layer beneath the levee body

Five different aquifer compositions are simulated, whereby the position and dimensions of layers are adjusted. The results are depicted in Figure 4.6. The third, and if present fourth, upper-case letter(s) indicate the position of the layer within the aquifer. The C stands for central and TM stand for top middle. The full purpose of the abbreviation is described in Section 4.3.1.1. Two specific aquifer compositions are simulated again with an increased water level or applied head (Figure 4.7).

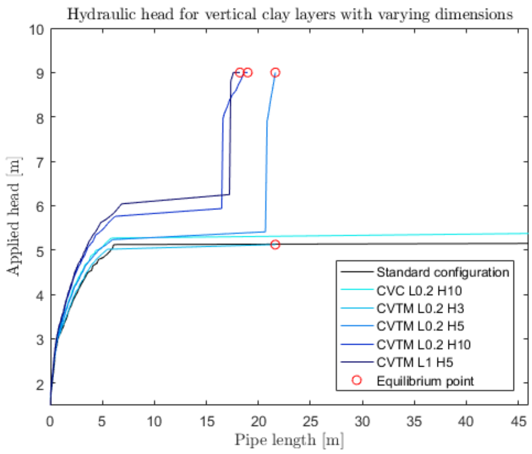


Figure 4.6: The results of the pipe development for a vertical layer with varying properties at the top of the aquifer

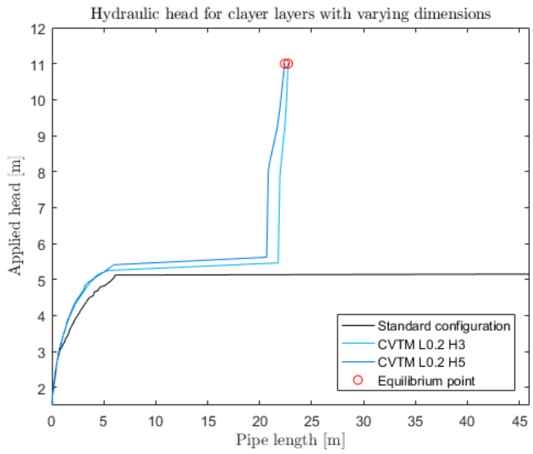


Figure 4.7: Two specific results of the pipe development for a vertical layer with varying properties at the top of the aquifer with an increased applied head

A couple of interesting results can be observed. Firstly, the pipe will reach an equilibrium, if an impervious layer in situated in the top of the aquifer, even if the applied head is increased. Due to the presence of the vertical impervious layer in the top of layer, the water flow is blocked and deflected. Hence, the magnitude of flow decreases behind the layer, whereby a virtually stagnant area arises in the top of the aquifer directly behind the screen. A stagnant area is a area without currents or flows. The pipe progress horizontally towards the stagnant area until the horizontal magnitude of the flow becomes too weak to erode and move the sand particles, so that an equilibrium is reached. However, the vertical component of the flow will become larger behind the layer because of the deflection of the streamlines resulting in diagonal or vertical erosion. Thus, at a certain point before the equilibrium point, the pipe will progress diagonally or vertically around the layer. However, vertical progression cannot be calculated in D-Geo flow. It can only calculate the erosion of the pipe along a horizontal path beneath the levee body. The results are therefore not fully representative.

However, it is very likely that the pipe erodes horizontally over the first few meters. For that reason, the outcome of the first few meters is assumed to be reliable. Over the first few meters it is observed that higher head can be applied if the screen is taller and/or thicker. As a result, the aquifer composition becomes more resistant to piping.

The blockage and the deflection of water is can clearly be seen in Figure 4.8. The flow pattern is shown by the hydraulic head distribution in the aquifer. It can be seen that the color suddenly changes from green to orange at the location of the screen instead of there being a gradual change of color. This implies a blockage of water. In addition, it can be seen that the gradient changes direction, when the flow passes the screen which implies the deflection of the streamlines.

The layer or screen is also shifted towards the centre of the aquifer. The layer located in the centre of the aquifer blocks and bifurcates the flow resulting in more resistance. The flow area furthermore reduces. Both processes cause a lower pressure gradient near the exit point.

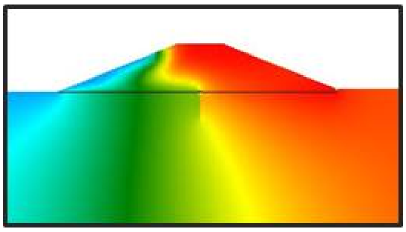


Figure 4.8: The hydraulic head distribution in an aquifer with a vertical layer at top of the aquifer at a specific water level

4.3.4 An anisotropic aquifer

As stated above, the anisotropic behaviour of the soil material is carried out by reducing the vertical hydraulic conductivity (K_v or K_y) by a factor of 10. So the vertical hydraulic conductivity is 0.5 m/day, while the horizontal hydraulic conductivity remains 5 m/day. The result is shown in Figure 4.9.

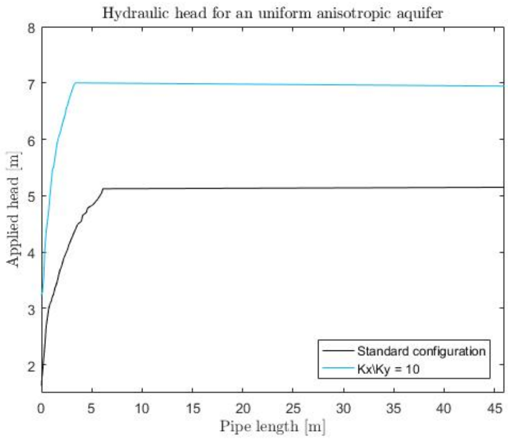


Figure 4.9: The critical head for a homogeneous and isotropic, and homogeneous and anisotropic aquifer

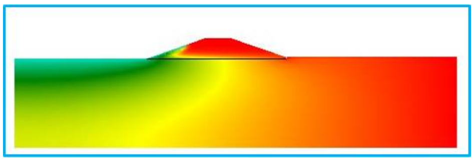


Figure 4.10: The hydraulic head distribution in homogeneous and anisotropic aquifer

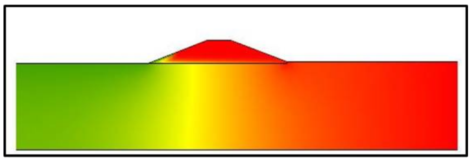


Figure 4.11: The hydraulic head distribution in homogeneous and isotropic aquifer

In Figure 4.9, it is observed that the critical head increases by 40 percent. The increase in critical head is clarified in Figure 4.10 and Figure 4.11. Figure 4.11 shows the flow pattern by means of the hydraulic head distribution in an isotropic aquifer. The water flows almost horizontally through the aquifer. By contrast, the water pattern is curved in the anisotropic aquifer (Figure 4.10). The water flows to a lesser extent vertically through the aquifer. Consequently, less water can flow from the deeper layer of the aquifer towards the exit or head of the pipe. In other words, the effective flow area of the aquifer is reduced thus resulting in a lower local gradient.

Table 4.2: The differences in critical head of all elementary compositions relative to the standard configuration in terms of percentage

	properties of the implemented layer					
name	material	hydraulic conductivity [m/day]	length [m]	height [m]	anisotropy ($\frac{K_h}{K_v}$) [-]	difference [%]
A horizontal layer in the centre						
a. changing soil material						
ScHC L18 H5	sand clayey	0.5	18	5	-	4.1
CHC L18 H5	clay	0.002	18	5	-	7.8
GHC L18 H5	gravel	250	18	5	-	12.5
b. changing the height or the width						
CHC L18 H1	clay	0.002	18	1	-	3.6
CHC L18 H3	clay	0.002	18	3	-	4.7
CHC L45 H3	clay	0.002	45	3	-	10.4
CHC L60 H3	clay	0.002	60	3	-	11.1
A horizontal layer beneath the manually constructed exit						
GHMR L24 H3	gravel	250	24	3	-	8.3
GHMR L24 H5	gravel	250	24	5	-	9.9
A vertical layer in the top						
CVC L0.2 H10	clay	0.002	0.2	10	-	5.5
CVTM L0.2 H3	clay	0.002	0.2	10	-	-
CVTM L0.2 H5	clay	0.002	0.2	10	-	-
CVTM L0.2 H10	clay	0.002	0.2	10	-	-
CVTM L1 H5	clay	0.002	1	5	-	-
An anisotropic aquifer						
Anisotropic aquifer	-	-	-	-	10	40.1

A dash(-) is given when the pipe reaches a solid equilibrium in the front the vertical layer, so the critical head cannot be observed. In reality, the pipe progresses vertically or diagonally around the wall, but D-Geo flow can only calculate horizontal paths. So at a certain point the results are unreliable. However, over the first meters the piping will progress horizontally so that these results are sufficient. Figure 4.6 shows that the aquifer with a vertical screen is more resistant to piping over the first meters. Because of this positive influence, the boxes are coloured green.

4.4 The results of the hydraulic head underneath the cover layer

The development of piping depends on the occurrence of uplift and heave. If uplift and heave do not occur, there is no exit channel and washing out of sand particles. The occurrence of both uplift and heave can be determined by the hydraulic head underneath the cover layer. Section 2.2.3.2 outlines the heave criteria, where the hydraulic head is the only load. Uplift is based on a balance between the weight of the cover layer(resistance) and the pressure expressed in hydraulic head underneath the cover layer(load). In order to verify the hydraulic head, the model set-up is modified. The inland boundary was shifted inland further from the toe of the levee, since it was found that the inland boundary greatly influences the hydraulic head underneath the cover layer. The inland boundary is assigned at 110 meter inland from the exit. In addition, the river head is modified. The high water holds longer and the water level starts from 2 m and increases more slowly towards high water (7 m). The outward head is firstly compared with the hydraulic head occurring underneath the cover layer at the toe of standard levee. A hydraulic fracture will most likely occur at the toe of the levee, since the pressure is still high and the blanket layer is relatively thin. Next, the hydraulic head occurring in the modified aquifer compositions is compared with the head occurring in the standard levee in order to analyse the effect of anisotropy and heterogeneity. The aquifer compositions are equivalent to the compositions that were presumed to exist as outlined in Section 4.2, however, the dimensions of the layer may differ. The changes in hydraulic head with respect to the standard configuration are caused by identical processes that are mentioned in previous sections.

4.4.1 Standard configuration

In this case, the standard configuration is systematized without an exit. The hydraulic head is calculated at the toe of the levee, where uplift and heave is most likely. The assigned submerged hydraulic head and inland hydraulic head are depicted in Figure 4.12. Several changes can be found. Firstly, hydraulic head is reduced by 39.5% from 7 m towards 4.2 m. It should be noted that the reduction is greatly influenced by the inland boundary. However, a free water level at 100 m inland from the toe is assumed to be acceptable and may be conservative. Secondly, the black line is shifted by 0.5 days, since the infiltration and transport of water takes time. Lastly, the black line is horizontal at the top which means that the groundwater flow is in a steady state from day 7.5 until day 8.

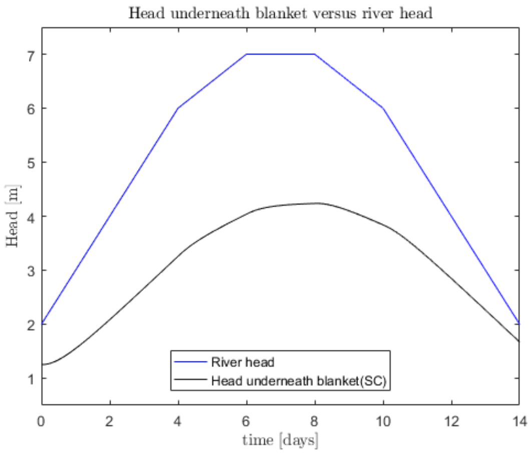
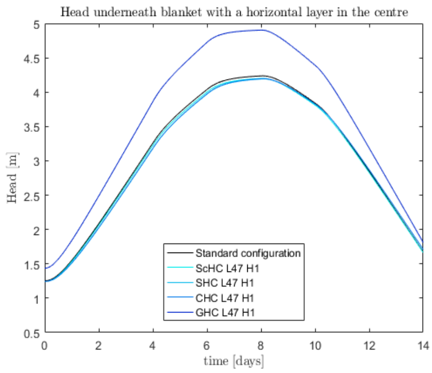


Figure 4.12: The outward (applied) and inland hydraulic head in the elementary study

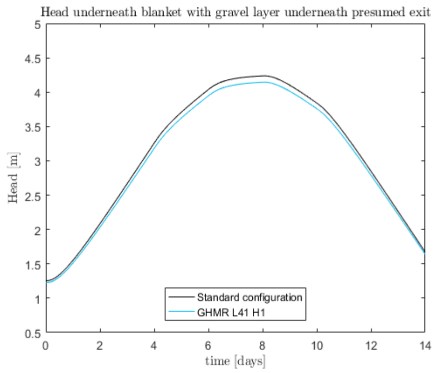
4.4.2 Aquifer compositions

All four aquifer composition are simulated. However, it is the vertical layer at the top of the aquifer that is focused on and the horizontal layer in the center or the aquifer. The effect of the resistances or conductors are observed by changing the soil type of the implemented layer. The dimensions remain constant, since the previous section concludes the dimensions amplify the effect but do not change it. However, the location of the layer can be of importance as well but is not researched in this section. The result are depicted in Figures 4.13a - 4.13d. Table 4.3 gives an overview of the changes expressed in the percentage of hydraulic head relative to the standard configuration. A negative difference is positive to the safety of the levee. Therefore, cells with negative differences are coloured green. Otherwise, A positive difference is negative to the safety of the levee, so these boxes are coloured red.

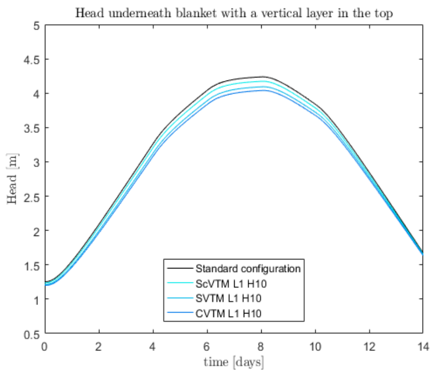
Figures 4.13a - 4.13d show similar results to the results observed in Section 4.2.2, except for the aquifer composition with a gravel layer at the centre. This gravel layer has a negative effect on the hydraulic head underneath the cover, whereas the aquifer with a gravel layer seems to be more resistant to the erosion progress. This dissimilarity occurs due to the deflection of the water flow by the gravel layer causing (a) new pipe equilibrium and thereby increasing the crital head, see Figure 4.1. On the other hand, Figure 4.1 shows a sudden increase in the pipe length at a lower gradient which implies that there are higher local gradients near the exit point. This observation is in accordance with the results of hydraulic head unterneath the cover layer. The degree of change differs mainly due to the implementation of other size layers and the highly non-linear behaviour of the boundary condition along the erosion channel (J. Sellmeijer, 2006), which determines the critical head.



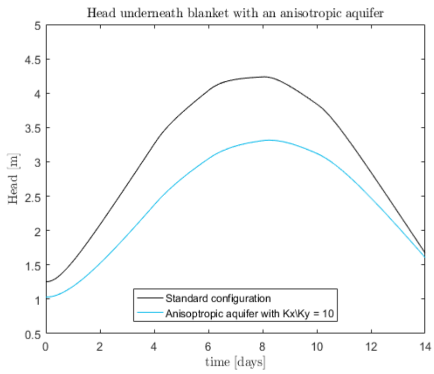
(a) a horizontal layer in the centre



(b) a horizontal layer in beneath the presumed exit



(c) a vertical layer in the top



(d) an anisotropic aquifer

Figure 4.13: The influence on the hydraulic head at presumed exit of the all elementary aquifer compositions

Table 4.3: The differences in hydraulic head at presumed exit relative to the standard configuration for all elementary aquifer compositions in terms of percentage

	properties of the implemented layer					
name	material	hydraulic conductivity [m/day]	length [m]	height [m]	anisotropy $(\frac{K_h}{K_v})$ [-]	difference [%]
A horizontal layer in the centre						
ScHC L47 H1	sand clayey	0.5	47	1	-	-0.9
SHC L47 H1	silt	0.1	47	1	-	-1.0
CHC L47 H1	clay	0.002	47	1	-	-1.0
GHC L47 H1	gravel	250	47	1	-	15.7
A horizontal layer beneath the presumed exit						
GHMR L41 H1	gravel	250	41	1	-	-2.2
A vertical layer in the top						
ScVTM L1 H10	sand clayey	0.5	1	10	-	-1.5
SVTM L1 H10	silt	0.1	1	10	-	-3.4
CVTM L1 H10	clay	0.002	1	10	-	-4.7
An anisotropic aquifer						
Anisotropic aquifer	-	-	-	-	10	-21.8

The aquifer compositions with a vertical layer at the top of the aquifer are simulated again, whereby the vertical layer is increased twice, untill it had reached the bottom of the aquifer. In that manner, the effect of the enlargement of dimensions is visualised. Although, it was already proven that larger dimensions amplify the effect (Section 4.3), the effect on the hydraulic head underneath the cover layer is re-examined. It is obviously observed that a layer with a larger height has more impact, see Figure 4.14 and Table 4.4. Besides, it can be observed that the effect of the layer is significant, if the height of the vertical layer is almost equal to the depth of the aquifer. In that case, the groundwater is forced through a narrow gap which induces significant entry resistances. As a consequence, less water flows towards the presumed exit. As a result, significant lower local pressures occur near the exit. If a vertical layer from top to bottom, with or without a narrow gap, is composed in the aquifer, the influence of the vertical layer mainly depends on the soil properties and thickness of the vertical layer.

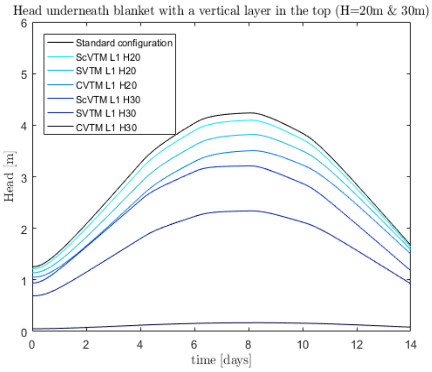


Figure 4.14: The hydraulic head at presumed exit point for a vertical layer with height of 20 m and 30 m at the top of the aquifer

Table 4.4: The differences in hydraulic head at presumed exit relative to the standard configuration for the aquifer with a vertical layer in the top of the aquifer in terms of percentage

name	material	difference
A vertical layer in the top (length = 1 m, height = 20 m)		
ScVTM L1 H20	sand clayey	-3.3
SVTM L1 H20	silt	-9.8
CVTM L1 H20	clay	-17.2
A vertical layer from top to bottom		
ScVTM L1 H30	sand clayey	-24.2
SVTM L1 H30	silt	-44.8
CVTM L1 H30	clay	-96.0

4.5 Conclusion

From all the gathered information of previous sections, conclusions can be drawn on the effects of anisotropy and hydraulic conductors and resistances for either uplift and heave, and pipe development. The primary conclusions are summarized here below. It should be noted that both an increase in critical head and a decrease in hydraulic head underneath the cover layer are positive with respect to the standard configuration.

- Both horizontal and vertical orientated impermeable layers (resistances) at the **centre** of the aquifer hinder and block the water flow. Due to the blockage, the flow pattern changes and less water converges at the exit or head of the pipe thus creating lower local gradient.

  - **Pipe development**
    The critical head increases in all aquifer composition. The degree of increase depends on the dimensions of the layer. In this case, the critical head is increased to 11%

  - **Uplift and heave**
    The hydraulic head underneath the cover layer is reduced by only 1%. The reason why the reduction is so small is because the implemented layer is very thin. The reduction is strongly dependent on the dimensions of the layer. A similar degree of increase in critical head is observed for very thin layer, see Table 4.2.

- A vertical impermeable layer or vertical screen in the **top** of the aquifer not only changes the streamlines by hindering and blocking the water flow, but also prevents **horizontal** erosion of the pipe.
  - **Pipe development**
    At a certain pipe length, the pipe starts to erode diagonally or vertically. D-Geo Flow can not calculate vertical erosion. As a consequence, the results become unreliable at a certain pipe length an the critical head will not exceed. However, it is very likely that the pipe erodes horizontally over the first meters. Therefore, it is concluded that a vertical screen thwarts the first stage of the piping process.
  - **Uplift and heave**
    The water flow experiences more resistance through the blockage and the hindrance of vertical layer. As a result a lower hydraulic head occurs at the toe of the levee. The degree of resistance is dependent on the dimensions and the permeability of layer. The lower the hydraulic conductivity of subsoil, the more difficult it becomes for water to flow through the subsoil. As a consequence lower hydraulic heads occur in less permeable elements. The hydraulic head can be reduced by 1.5% till 96.6% depending on the dimensions and the permeability of the layer.
- Horizontal permeable layers (conductors) influence the uplift, heave and pipe development both positive and negative depending on the position of the layer.
  - **Pipe development**
    If a layer is partly or entirely situated beyond the critical pipe length, water is partly conducted horizontally towards the inland boundary instead of towards the exit only. Consequently, less water converges at the exit resulting in lower local gradient. By contrast, a gravel layer situated before the critical pipe length conducts more water into the aquifer so that more water flows towards the exit point or head of the pipe thus resulting in higher local gradient.
  - **Uplift and heave**
    When a permeable layer ends before the toe of the levee, more water is conducted towards the critical point thus resulting in higher hydraulic head. On the other hand, a layer located underneath the toe of the levee forces some water horizontally towards the inland boundary. Because of this, the pressure near the toe of levee is reduced.
- By the addition of direction dependency (anisotropy), the flow area of the aquifer is used less effective under the condition that the horizontal hydraulic conductivity is larger than the vertical hydraulic conductivity ($K_h > K_v$). In that case, less water flows from the deeper layer of the aquifer towards the exit or head of the pipe so resulting in lower local gradient. In this thesis, it is observed that if $\frac{K_h}{K_v} = 10$, the critical head is increased by 40% and the hydraulic head underneath the cover layer is reduced by 20%.
- The dimensions of the layer are positively correlated to its impact. In other words, a larger layer amplifies the impact. However, it seems that the impact of gravel element beneath the exit point is limited.

5 | Realistic model

5.1 Introduction

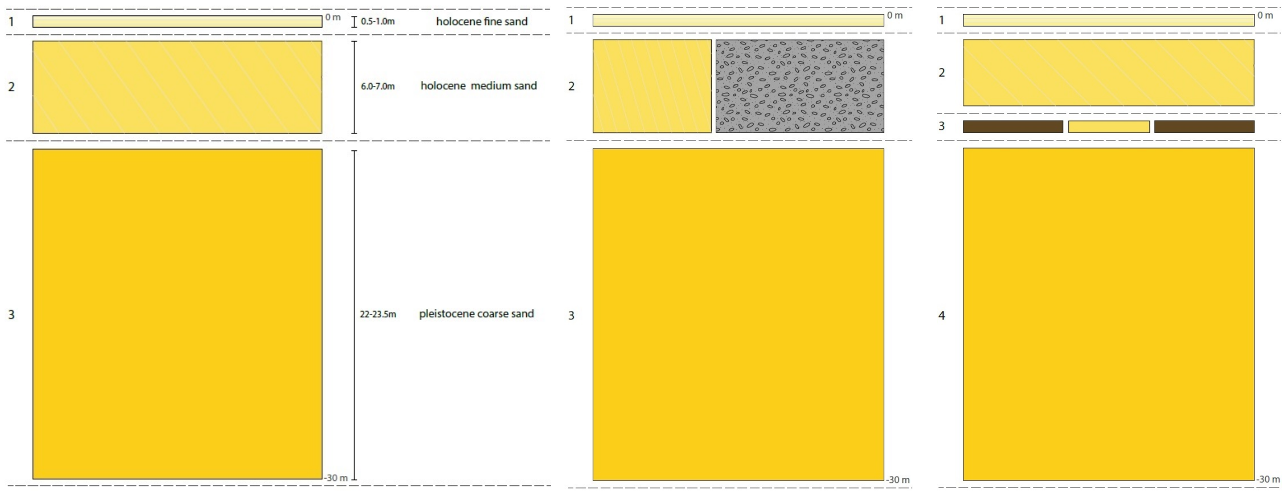
In the previous chapter, the effects of anisotropy heterogeneity were analysed by adding artificial elements with variable soil properties or by implementing anisotropy to homogeneous aquifer. In this analysis, an explicit distinction was made between the phases uplift and heave, and backward erosion piping or pipe development. Uplift and heave are mainly dependent on hydraulic head underneath the cover layer, whereas the development of a pipe is dependent on the critical gradient. Multiple valuable results were observed. The added elements, with only one exception, positively change both the critical head and hydraulic head underneath the cover layer with respect to the standard configuration. In other words, the elements decrease the pore pressures underneath cover layer mainly due to their influence on the flow pattern. As a result, the strength of the levee increases. Nevertheless, a gravel layer in the middle of the aquifer has a negative influence on the hydraulic head underneath but not on the critical gradient. This dissimilarity occurs because the gravel element is not located in front of the point of investigation, but beneath the the point of investigation. A gravel layer which is located in front of the point of investigation induces more water to flow towards the presumed exit so resulting in higher pressures, while a gravel layer located beneath the point of investigation also attracts water so that water is also flowing into the gravel layer towards the inland boundary instead of flowing only towards the exit or head of the pipe. As a result, lower pressures occur at the exit or head of the pipe. Accordingly, the degree of influence of gravel layer is highly dependent on its position. Besides that, it was observed that the influence of a layer is also dependent on its orientation and size. Obviously, a larger layer amplifies the effect. In addition, it is proven that anisotropic behaviour significantly increases the safety of the levee, if the vertical permeability is lower than the horizontal permeability.

This chapter continues on the effects of anisotropy and heterogeneity observed in previous chapter. The first aim of this chapter is to observe the effect of stratification. In this thesis, a stratified aquifer is defined as a heterogeneous aquifer (see Section 1.5). Furthermore, anisotropic behaviour and, a natural conductor and resistance are implemented to create more heterogeneous or heterogeneous and anisotropic aquifer compositions. These aquifers are composed on the basis of the literature, open source subsoil data of the Netherlands and conversations with specialists and therefore referred to as realistic aquifers. The general realistic aquifer consists of three different homogeneous and isotropic sand layers. The stratified realistic aquifers are compared with a representative homogeneous and uniform aquifer which has one calculation value for the 'bulk' permeability. The representative calculation value of the 'bulk' permeability can be used in Sellmeijer's design rules. In that way, the model outcomes provide insights into whether the current used design and assessment rules overestimates or underestimates the effects of anisotropy and heterogeneity such as stratification, a natural resistance and a natural conductor in an aquifer. In this chapter, the effects of anisotropy and heterogeneity are analysed for both pipe development (Section 5.4) and, uplift and heave (Section 5.3). The conclusions regarding the realistic aquifers are drawn in the last section.

5.2 Realistic aquifers compositions

The elementary model has been modified to a more realistic model by implementing stratified aquifer compositions. The realistic aquifer compositions are not extracted from specific Dutch sites. However, they represent typical Dutch subsoils because the realistic aquifers are established on the basis of discussions with Dutch specialists, the literature study and available open source subsoil data of the Netherlands. Three stratified aquifer compositions have been established. These compositions consists of at least three different sandy layers. In addition to stratification, each aquifer composition has been implemented with at least anisotropy or one additional element with different hydraulic properties: a gravel layer (conductor) or a discontinuous clay layer (resistance). Figure 5.1 shows the three realistic aquifers. Figure 5.1a includes also the dimensions and the soil type of the layer. The dimensions and soil types of the other realistic aquifer compositions are given in Appendix C.

As stated above, these realistic aquifer compositions can be found in the subsoil at Dutch areas. A gravel layer underneath a small sandy top layer can be found in the Maasvallei (van der Hulst, 2017). A subsoil composed of holocene and pleistocene sands with a discontinuous clay layer is characteristic for the Dutch river area.



(a) (an)isotropic pure sand layer(s) (b) a gravel layer (c) discontinuous clay

Figure 5.1: The three realistic stratified aquifer compositions

Besides, the implementation of stratified aquifers, some soil types are modified. The new soil types with their corresponding properties are described in Section 5.2.1. Furthermore, several boundary conditions are adjusted as a result of new insights obtained by, among other thing, the sensitivity analysis (Section 3.6). These boundary conditions are discussed in Section 5.2.2. Finally, the realistic aquifer compositions are plotted against the representative uniform and homogeneous aquifer with a single value for its bulk permeability. In the design rules one calculation value of the bulk permeability is used only. By comparing the 'bulk' permeability with the stratified permeability profile the effects of anisotropy and heterogeneity becomes tangible. The methods to calculate the 'bulk' permeability is described in Section 5.2.3.

5.2.1 Soil properties

In order to compose typical Dutch aquifer profiles, several sand types are selected on basis of the Dutch guidelines to schematise piping sensitive subsoils (Förster, de Bruijn, Kruse, Hijma, & Vonhögen-Peeters, 2017). The following sand types with the corresponding **horizontal** hydraulic conductivity are extracted:

Table 5.1: Sand types with the corresponding horizontal hydraulic conductivity (Förster et al., 2017)

soil type	mean horizontal hydraulic conductivity K [m/day]
holocene sands or the shallow subsurface layer	
fine sand	4
medium sand	15
coarse sand	40
pleistocene sands or the deeper subsurface layer	
moderate coarse sand	30
very coarse sand	40

It should be noticed that these values of the horizontal hydraulic conductivity are applied as averages, whereby a 50% coefficient of variation is used as the starting value in the legitimate safety assessment of piping.

In this thesis, three sand types are used and assigned to specific layers:

- Layer 1: holecene fine sand
- Layer 2: holecene medium sand
- Layer 3 (or 4): pleistocene moderate coarse sand

The properties of the other layers such as the blanket, levee, gravel or clay are equivalent to the previously assigned properties, see Table 3.1. In addition, the grain diameter of the upper layer is also of importance in case of piping, since the upper layer is the layer where erosion appears. The grain diameter of the upper layer is $1.5{\cdot}10^{-4}$ m, which is according to the literature. This value was also used in the elementary study. The auxiliary properties of all soils such as the bedding angle are similar, see Table 3.2.

5.2.2 Boundary conditions

On the basis of the sensitivity analysis, some the geometry and boundary choices are adjusted. Firstly, it was observed that the inland boundary significantly influences the results, especially the hydraulic head underneath the cover layer. Secondly, the effect of a constant head boundary condition approaches the effect of no flow boundary condition as the inland length increases. Lastly, it was noticed that models assigned with no flow inland boundary condition are time-consuming. As a consequence of three observations, an inland head boundary condition is assigned at 200 meter inland from the presumed exit.

Moreover, the hydraulic head underneath the cover layer is highly influenced by hydraulic conductivity and thickness of the blanket. A more permeable blanket induces more leakage of water resulting in lower hydraulic gradient, while a thicker blanket hinders the outflow of water so resulting in higher hydraulic gradient. The degree of influence of the cover layer becomes substantial if the permeability is in the order of 0.1 m/day and the cover layer is quite thin (in the order of 0.5 m). However, this is not very likely in practice. In this case, a poorly permeable and relatively thin blanket is assumed as a representative cover layer with a thickness of 1 m and a permeability of 0.002 m/day.

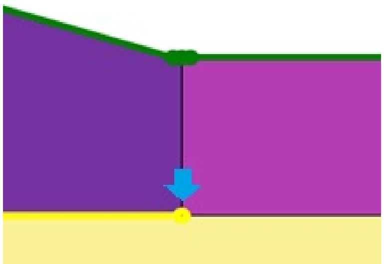


Figure 5.2: Heave boundary

Besides these adjustments, an internal boundary condition is introduced. A heave condition is assigned at the beginning of the pipe underneath the cover layer, see Figure 5.2. In this way, the model simulates flow through a presumed uplift channel. At beginning of the pipe underneath cover layer a pressure can be assigned. Through the assignment of this pressure, the resistance of the cover layer and fluidized sand in the uplift channel can be included. If a pressure is assigned then the water starts to flow when the occurred hydraulic pressure exceeds the assigned pressure. Through this internal boundary condition, an exit is created in the shape of a crack so that the construction of small ditch is superfluous and therefore omitted. As a consequence, the effect of the exit configuration can also be neglected.

Important note: the settings of the other boundary conditions are the same as in the basic model. However, the river boundary is in some case slightly changed in order to investigate some critical processes more accurate without introducing more time step. These changes may slightly change the outcome.

5.2.3 Bulk permeability

The bulk permeability of an aquifer is an essential parameter by comparing a homogeneous and uniform aquifer with a heterogeneous aquifer. As mentioned before, in the Sellmeijer design rule, the failure probability on piping can only be calculated by means of one calculation value of the bulk permeability. In the Netherlands, a computer-program (`RisKeer`) is used to calculate the failure probability on piping. `RisKeer` generates for each aquifer composition one calculation value of the bulk permeability by supposing parallel flow (Förster et al., 2017). As a consequence, the bulk permeability only concerns horizontal flow and horizontal permeabilities. This calculation value of bulk permeability is subsequently used Sellmeijer's design rules to determine the safety of levee against piping. In addition, the software can not consider variations within a horizontal layer. To by-pass that problem, at least two different aquifer scenarios should be created. This method to calculate the bulk permeability is compared with an alternative method to determine the bulk permeability in order to analyse whether anisotropy and heterogeneity can be considered in the Sellmeijer design rules

The alternative method to determine the bulk permeability assumes that the groundwater is flowing curved or radially into the (deeper) subsoils and towards the exit. This assumption is made since the groundwater flow problem in the case of an exit or pipe is comparable to the grounwater flow problem in the case of a well or a drain. In that case, the assumption of only horizontal flow properly approaches the aquifer under the condition that the aquifer is relative thin, homogeneous and unconfined (Ernst, 1963). Otherwise, the groundwater flow towards the exit is curved or radial. As a consequence, the vertical permeability is als of importance. The bulk permeability for curved or radial flow (K_r) is often be computed from the geometric or logarithmic mean of K_h and K_v (Ritzema, 1994), see Equation 5.1 and Equation 5.2. Moreover, anisotropy can be concerned by the alternative computation method due to the consideration of the vertical hydraulic permeability (K_v). In addition, the alternative method deals with variations within a horizontal layer. The alternative method consists of the following three steps, see Figure 5.3 for a visualisation of these steps.

1. **Determine the dimensions and the permeability of each layer in the heterogeneous aquifer.** if it is possible, determine both a horizontal and a vertical permeability. Otherwise, assume isotropic behaviour of the soil so that vertical permeability is equal to the horizontal permeability ($K_h = K_v$).

2. **Calculate a representative vertical and horizontal conductivity of the heterogeneous horizontal layer.**
   The representative vertical and horizontal permeability are calculated with Equation 2.12 and Equation 2.13. Be aware: the vertical permeability must be calculated with the formula for parallel flow and the horizontal permeability must be calculated with formula for perpendicular flow. Besides, the depth must be replaced by the length of each layer. In this way, a uniform horizontal layer is created so that the aquifer only consists of horizontal stratification.

3. **Calculate the bulk permeability of the aquifer**.
   Firstly, one must calculate a value for the vertical and horizontal permeability of the aquifer by using the formula for parallel and perpendicular flow. These values can be used in D-Geo flow model, but not in the Sellmeijer design rules. Therefore, a representative permeability (K_r) is calculated by assuming that groundwater is flowing in radial direction towards the exit. The representative value concerns both anistropy and heterogeinity. The rrpresentative value can be approximated by the following formula (Ritzema, 1994):

$$K_r = \sqrt{K_v K_h} \tag{5.1}$$

or

$$ln\ K_r = \frac{ln\ K_v + ln\ K_h}{2} \tag{5.2}$$

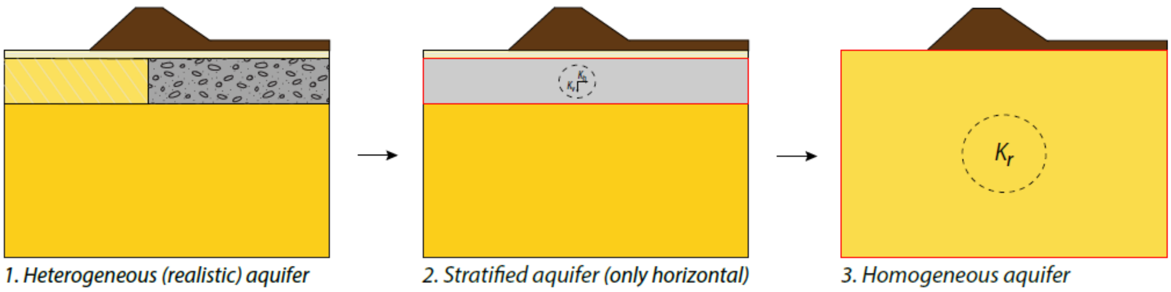


Figure 5.3: An illustration of the determination of the bulk permeability

These steps are carried out for the three realistic aquifer compositions. The results are given in Table 5.2. A displacement of the layer do not change the bulk permeability as long as the dimensions and permeability remain the same. If the dimensions or permeability of the layer changes, the bulk permeability will change as well. Logically, the degree of change of the bulk permeability depends on size of the adjustments. In addition, the radial bulk permeability is always lower than the horizontal bulk permeability, since it also concern groundwater flow perpendicular to soil layer.

Table 5.2: The dimensions and permeabilities of the realistic aquifer compositions

	properties of the layer				representative 'bulk' properties		
schematisation	soil type per layer	h[m]	l[m]	K[m/day]	K_h[m/day]	K_v[m/day]	K_r[m/day]
(a) pure sand	1 holocene fine sand	1	300	4	26.1	21.2	23.5
	2 holocene medium sand	6	300	15			
	3 pleistocene coarse sand	23	300	30			
(b) gravel layer	1 holocene fine sand	1	300	4	33.9	28.6	31.1
	2 holocene medium sand & gravel layer	6	70 230	15 250			
	3 pleistocene coarse sand	23	300	30			
(c) clay layer	1 holocene fine sand	1	300	4	25.1	18.9	21.8
	2 holocene medium sand	6	300	15			
	3 holocene medium sand & discontinuous clay layers	1	100 200	15 0.002			
	4 pleistocene coarse sand	22	300	30			

5.3 The results of the critical head

This section outlines the results of the development of pipe. In order to analyse whether heterogeneous and/or anisotropic aquifer compositions are more resistance to pipe development, the change in critical head is observed. The critical head applied in the realistic aquifer compositions is compared to a representative uniform aquifer which has one calculation value for the bulk permeability. The bulk permeability of an aquifer is used in Sellmeijer's design rules to assess and design levees. By comparing a realistic aquifer with the representative homogeneous aquifer, it can be determined for which aquifer compositions the current assessment with Sellmeijer design rules underestimates or overestimates the likelihood of piping. In this comparison, the homogeneous and uniform aquifer can have two different representative values of bulk permeability. A bulk permeability that only assumes horizontal flow and is therefore referred to as the horizontal 'bulk' permeability(K_h). This horizontal 'bulk' permeability is currently applied in the detailed assessment in `RisKeer`. The other value of bulk permeability is determined by an alternative method that takes variation within a horizontal layer and vertical flow into account. In the alternative method to determine the value of bulk permeability, it is assumed that the groundwater flowing towards the exit is curved or radial. As a result, the second value of the bulk permeability is known as the radial bulk permeability(K_r). The overestimation or underestimation of the assessment is quantified by calculating the degree of change in critical head with respect to the critical head simulated with the representative uniform and homogeneous aquifer. The changes are presented in terms of percentage. An increase in critical head is positive, since it means that the realistic aquifer compositions more is resistant to piping. As a result, the Sellmeijer design rules underestimates resistance of aquifer. If an increase is observed the results are coloured green, otherwise the results are coloured red. The following sections outlines the results of each realistic aquifer composition separately.

5.3.1 Pure sand

As described in Section 5.2, the pure sand stratified aquifer composition consists of three sandy layers:1. holocene fine sand 2. holocene medium sand 3. pleistocene moderate coarse sand In addition to stratification, anisotropic behaviour is added to one and/or two sandy layers, while the other layer(s) remain(s) isotropic. Within the anisotropic layers, the vertical permeability is decreased by a factor of 4. This reduction is reasonable or even quite conservative, since the literature study revealed that the vertical permeability can be a factor of 100 smaller, especially in former tidal flats due to horizontal sedimentation of small and thin clay layers. In addition, it is justified to decrease the vertical permeability instead of increasing the horizontal permeability, since the horizontal permeability is often only measured. Furthermore, the value of permeabilities of the sandy layers are extracted from Förster et al. (2017). This report only outlines horizontal permeabilities. The following four pure sand aquifer are simulated:

- isotropic sand layers
- anisotropic fine sand layer (top layer)
- anisotropic medium sand layer (middle layer)
- anisotropic holocene sand layers

The results of the simulations are given in Figure 5.4 and Table 5.3. It is clear that the critical head is increased for all realistic stratified aquifers with respect to the homogeneous aquifer for both the simulations with K_r and with K_h. The changes are caused by the stratification and anisotropy. The two upper layers are less permeable as the representative bulk permeability. In that case, the two upper layers allows less water flowing into the deeper layer and allows less water flowing from the deeper layer upwards to the exit. As a result less water is flowing horizontally towards the exit of head of the pipe. Consequently, lower pressure occurs near the exit or the head of the pipe so resulting in a higher critical head. These results suppose that the upper layers are of more importance then the deeper layers. This hypothesis is confirmed by the results of the simulations in which anisotropic behaviour is added to one or two holocene sand layers. Although both holocene layer has the same degree of anisotropy, a higher critical head occurs if anisotropy was added to the seven times thinner top layer. In short, it can be concluded that not only the height, permeability and degree of anisotropy of a layer is of importance, but also the positions of a layer.

The elementary study has already revealed that anisotropy has significant impact on a groundwater problem. This effect is also observed in this study. If the vertical permeability is lower than the horizontal permeability, the critical head increases significantly. This increase is caused by the fact that less water can flow downwards to the deeper layer and less water can flow from the deeper layer upward towards to the exit or head of the pipe. In other words, the aquifer is used less effective. Overall, it can be concluded that both the effect of stratification and anisotropy might be significantly underestimated in the design rules. This statement is certainly true when the upper sand layers are less permeable than the underlying sand layers and when the vertical permeability is lower than the horizontal permeability.

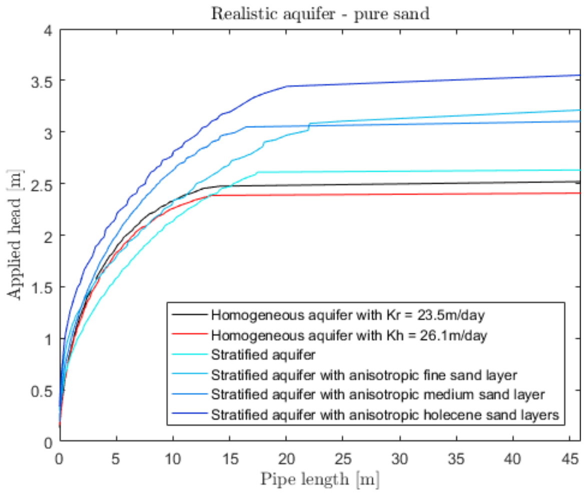


Figure 5.4: The critical head for stratified and anisotropic aquifer compositions

Next to the increase in critical head due to the simulation of realistic aquifers compositions, it can also be observed that there is a difference in critical head, when a homogeneous aquifer with 'bulk' permeability of K_h or K_r is stimulated. This difference is self-evident because the horizontal bulk permeability is larger than the radial bulk permeability so resulting in a lower critical head. The change in critical head with respect to a homogeneous and uniform aquifer with bulk permeability (K_h) is obviously larger than the change in critical head with respect to a homogeneous aquifer with radial 'bulk' permeability (K_r). It should be noted that the K_r is once calculated for the isotropic pure sand aquifer and not adjusted when anisotropic behaviour has been added, while the K_r decreases if the vertical permeability of layers decrease. As a consequence, the critical head rises resulting in a smaller increase with respect to the K_r. These results are therefore slightly distorted. However, a smaller difference between the homogeneous and heterogeneous simulations implies that the alternative method would be more accurate than the currently applied method in `RisKeer`. To justify the latter statement additional simulations are performed in which the K_r value is decreased as well, see Appendix D. In Appendix D, it can be observed that the alternative method to determine the bulk permeability is 27 % more accurate than the the currently applied method in `RisKeer`.

Table 5.3: Influence of anisotropy and stratification on the critical head in terms of percentage [%]

	direction dependence p.l. $[\frac{K_x}{K_y}]$			difference [%]	
pure sand compositions	layer 1	layer 2	layer 3	w.r.t. K_r (23.5 $\frac{m}{day}$)	w.r.t. K_h (26.1 $\frac{m}{day}$)
isotropic layers	1	1	1	5.5	9.4
anisotropic fine sand layer	4	1	1	25.5	30.2
anisotropic medium sand layer	1	4	1	23.2	27.8
anisotropic holocene sand layers	4	4	1	39.1	44.3

5.3.2 Gravel layer

The gravel layer compositions serve as a representative compositions for the Maasvallei in the Netherlands. The subsoil in the Maasvallei is typically composed of a small sand top layer with underlying gravel package with a considerable permeability (van der Hulst, 2017). In addition, the elementary study revealed that a gravel layer can have both a positive or negative effect depending on the position of the gravel package. The effect being positive or negative is dependent on whether the gravel package is fully at place at the point of investigation. To research the effects of a gravel package in a stratified subsoil, the following three aquifer compositions are simulated, whereby layer 2 consists of:

- a holecene medium sand package followed by a gravel package;
- a gravel package followed by a holecene medium sand package;
- a complete gravel package.

The transition from sand to gravel or vice versa occurs in front of the critical pipe length so that both the positive and the negative effect will appear. Besides, the results of the simulations are compared with an additional homogeneous aquifer with a larger radial bulk permeability (K_r). Otherwise, the results of a complete gravel package are distorted, since this composition has an considerable larger radial 'bulk' permeability due to the absence of the transition to sand. The K_r value of 45.9 m/day corresponds to the realistic compositions with a complete gravel layer and the K_r value of 31.1 m/day corresponds to sand - gravel or gravel - sand layer. The outcomes of this analysis are depicted in Figure 5.5 and Table 5.4.

A few valuable outcomes can be observed. Firstly, both the positive and negative effect can be noticed. As stated in the elementary study, the positive effect occur due to fact that water prefers the path of less resistance. As consequence, a fraction of the water is flowing into the gravel towards the inland boundary instead of flowing towards the exit. Hence, the gravel layer nearly contributes to the erosion process. However, the degree of increase is above all expectations and therefore clarified later on in more detail. The negative response occurs due to the fact that the gravel is located on the river side of the levee so that it conducts more water into the aquifer. The flow pattern is not disturbed by any conductor or resistance so that the water can easily flows towards the exit or head of the head. The second valuable outcome is that the homogeneous aquifer with a K_r value of 45.9 m/day almost approaches the realistic aquifer with a complete gravel layer, whereas the horizontal bulk permeability (K_h) underestimated the outcome (see Figure 5.6). In other words, the K_r value seems to be more accurate. The mild effect of a complete gravel layer in the aquifer occurs due to the fact that the negative and positive process will almost nullify each other.

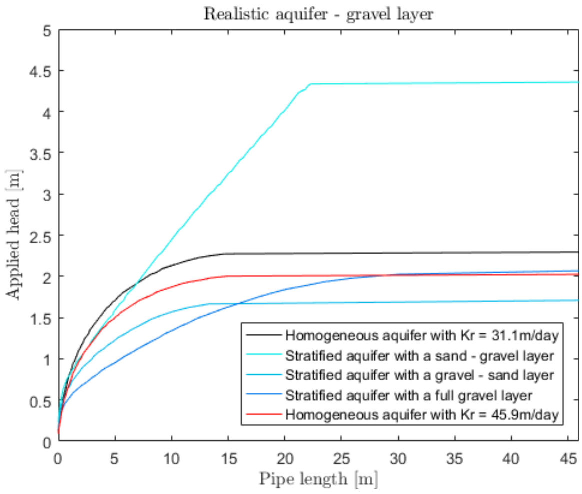


Figure 5.5: The critical head for the realistic aquifer with a (partly) gravel layer

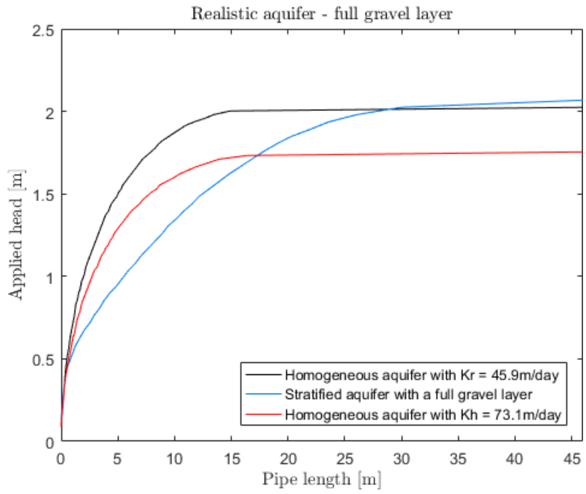


Figure 5.6: The critical head for the realistic aquifer with a full gravel layer

Table 5.4: Influence of a (partly) gravel layer on the critical head in terms of percentage[%]

	difference [%] w.r.t. bulk permeability [m/day]		
gravel layer compositions	w.r.t $K_r = 31.1$	w.r.t f.g $K_r = 45.9$	w.r.t f.g. $K_h = 73.1$
sand - gravel layer	90.7	116.5	150.2
gravel - sand layer	-26.7	-16.6	-3.9
full gravel layer	-10.9	1.1	16.1

In order to clarify the unforeseen positive increase, two points of investigation are selected, as indicated by black dots in Figure 5.10. One point at the top of the fine sand layer in which the pipe will arise and one point in the middle of the gravel layer which will conduct water towards the inland boundary. The points are located directly below each other. The erosion progress is intermittent at those points. In Figure 5.9 three stages can be perceived. In stage 1, the pipe progresses to the points of investigation. The velocities increase until the pipe reaches the point of investigation (orange dashed line). After the pipe passes the points of investigation (stage 2), the velocities decrease both in the gravel layer and the fine sand layer. This stage holds until the pipe reaches the river (red dashed line). An open connection is established (stage 3). If the open connection is established, the model becomes unreliable and therefore stage 3 is neglected.

Two interesting processes can be observed. First, significantly higher velocities arise in the gravel layer in stage 1, which implies that more water is flowing into the gravel layer and less water is flowing through the fine sand layer. Secondly, a higher decrease in velocity appears in the fine sand layer in stage 2. This implies that the gravel layer contributes to a lesser extent to the erosion process. It seems that a significant amount of water is flowing towards the inland boundary instead of flowing towards the exit or head of the pipe. Both processes result in lower local pressure near the exit or head of the pipe and thereby clarify the increase in critical head. An illustration of the flow pattern in the aquifer with a sand package followed by a gravel package is given in Figure 5.8. Since the river head level was slightly changed for this composition, the applied head is given in Figure 5.7. Above all, it can be expected that the erosion will occur at the transition between sand and gravel instead of underneath the levee due to the significant larger velocities in the gravel layer. This phenomena could be an interesting topic for follow-up research.

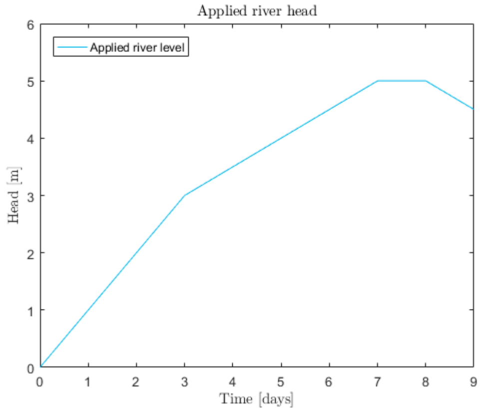


Figure 5.7: The applied (river) head assigned to the realistic gravel layer composition

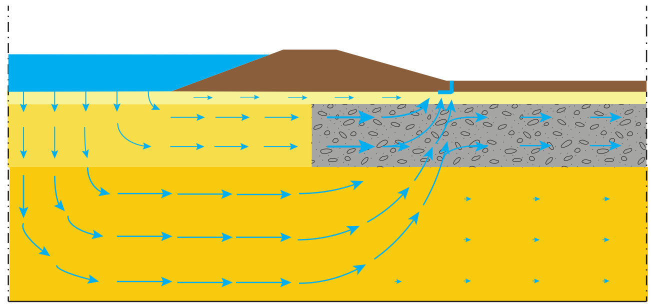


Figure 5.8: The flow pattern in an aquifer with a sand-gravel package

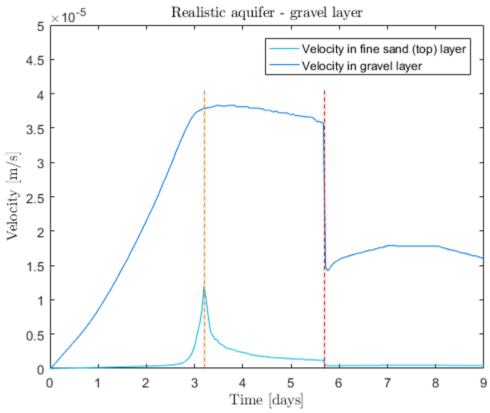


Figure 5.9: The velocities over time at the points of investigation in the realistic gravel layer composition

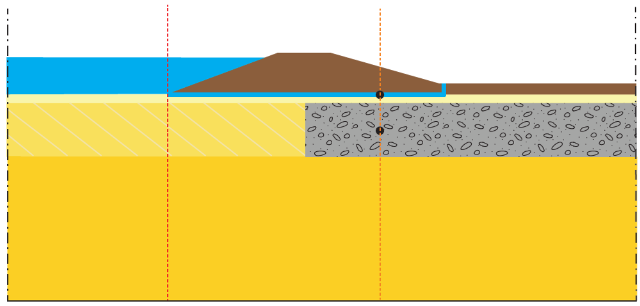


Figure 5.10: The locations of the points of investigation in the realistic gravel layer composition

5.3.3 Discontinuous clay layer

The discontinuous clay layer compositions serve as representative compositions for the downstream river area in the Netherlands. A thin discontinuous clay layer is often found between the holocene and pleistocene sands. In All these realistic compositions, the discontinuous clay layer has a gap underneath the river to allow water to flow into the deeper pleistocene sand layer. Furthermore, a gap is created underneath the levee body or exit point so that the deeper layer will contribute to the erosion process. After which the effects of the position and size of the break is analysed. The position and size of the break are changed once. The following three realistic compositions with a discontinuous clay layer are simulated:

- a break of 25 m underneath the levee body
- a break of 14 m underneath the levee body
- a break of 25 m underneath the exit point.

The results of these simulations are depicted in Figure 5.11 and Table 5.5. It is visible that all compositions have a positive influence on the critical head. In other words, the critical head is increased in all simulations. The realistic aquifers are only compared with the homogeneous aquifer with a radial bulk permeability. The previous two sections had already proven that the radial bulk permeability is

5% till 15% more accurate. The difference in critical head can be clarified by means of Figure 5.12 and Figure 5.13.

Figure 5.12 and Figure 5.13 show the flow pattern in two aquifer compositions. The flow pattern with respect to a homogeneous situation differs in both compositions. In the beginning of the erosion process, similar local pressures occur near the exit point or head of the pipe due to the convergence of flow. This statement is confirmed in Section 5.4.3. However, a difference occurs if the pipe progresses toward the river. The groundwater flow in the aquifer with a break beneath the levee can still converge near the head of the pipe so that it nearly encounters hindrance of the clay layer. However, if the exit is located underneath the exit point, the water flow encounters hindrance of the clay layer. Due to the clay layer, water takes a longer path inducing more resistance. Besides, the flow is less concentrated near the exit or head of the pipe(Figure 5.13). Both processes induce lower local pressure so that the critical head increases. The size of the gap also affects the water flow, since smaller gap induce more resistance. Nonetheless, the effect of the gap will be of significance, if its size is a few meters or narrower.

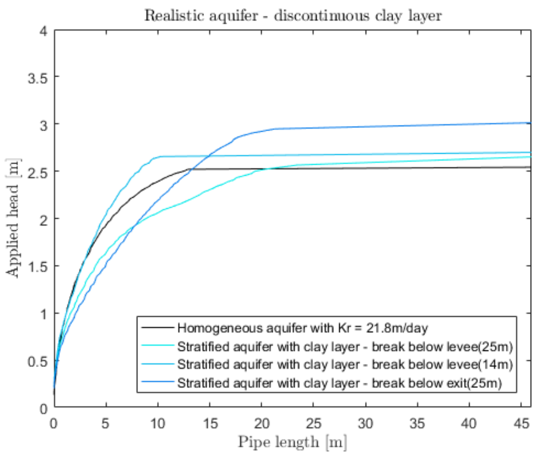


Figure 5.11: The critical head for aquifer with a discontinuous clay layer

Table 5.5: Influence of discontinuous clay layer on the critical head in terms of percentage[%]

		difference [%]
clay layer compositions	break [m]	w.r.t K_r (21.8 m/day)
break below levee body	25	1.8
break below levee body	14	5.4
break below exit point	25	17.0

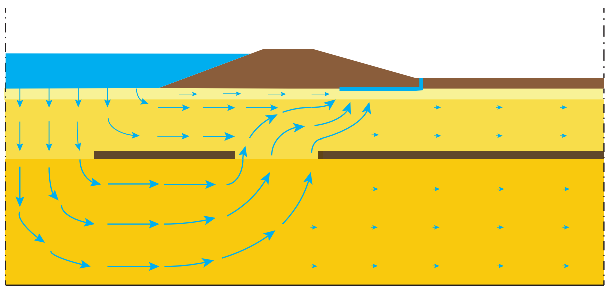


Figure 5.12: The flow pattern in an aquifer with a discontinuous clay layer with a break beneath levee

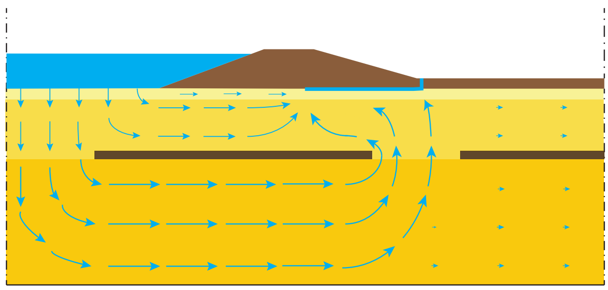


Figure 5.13: The flow pattern in an aquifer with a discontinuous clay layer with a break beneath exit

5.4 The results of the hydraulic head underneath cover layer

This section outlines the result of the hydraulic head underneath the cover layer. As mentioned before, the hydraulic head underneath the cover layer is the main driver for the occurrence of the mechanisms uplift and heave. In order the analyse the effect of heterogeneity, the hydraulic head occurring in the realistic aquifer is compared to the hydraulic head occurring in the homogeneous aquifer with a radial bulk permeability (K_r). The horizontal 'bulk' permeability is omitted in this section, since Section 5.3 has revealed that the radial 'bulk' permeability is for almost all compositions 5% till 15 % more accurate. In addition, the elementary study has revealed that changes in hydraulic head are highly correlated to the changes in critical head. Since the processes causing the changes are similar. Nevertheless, the critical head represents a resistance and the hydraulic head a load so that the correlation between the critical head and hydraulic head is negative. An increase in resistance and a decrease in load are therefore both positive for safety of a levee and coloured green. However, if the results are not negatively correlated, then it has to do with the position of the anisotropic or deviating layer with respect to the point of investigation. The results of each realistic aquifer are briefly described separately in the following sections.

5.4.1 Pure sand

The main aim of the pure sand compositions is to analyse the effect stratification and anisotropic behaviour of layer. Figure 5.14 and Table 5.6 show the results. It is visible that stratification and anisotropic behaviour both decrease the hydraulic head underneath the cover layer. The decrease is mainly caused by the upper layers, which are less permeable, so that less water is flowing into the deeper layer. Furthermore, Anisotropic behaviour of the upper layer will amplify this process. For a detailed description of this process reference is to Section 5.3.1. It can be noticed that degree of decrease in hydraulic head is much smaller than the increase in critical head, especially when anisotropic behaviour has been added. This deviation is mainly caused by the boundary condition along the erosion channel, which does not apply in the simulations of the hydraulic underneath the cover layer.

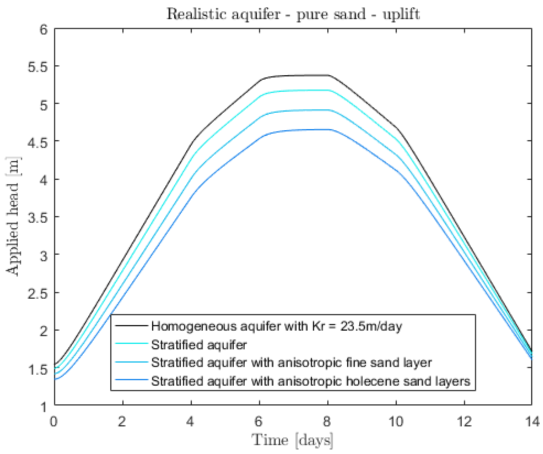


Figure 5.14: The hydraulic head at presumed exit for stratified and anisotropic aquifer compositions

Table 5.6: Influence of anisotropy and stratification on the hydraulic head in terms of percentage[%]

	direction dependence p.l. $[\frac{K_x}{K_y}]$			difference [%]
pure sand compositions	layer 1	layer 2	layer 3	w.r.t. K_r (23.5 m/day)
isotropic layers	1	1	1	-3.7
anisotropic fine sand layer	4	1	1	-8.5
anisotropic holocene sand layers	4	4	1	-13.4

5.4.2 Gravel layer

In order to analyse the effects of the gravel layer in a stratified aquifer on the hydraulic head underneath the cover layer, three different aquifer compositions are simulated. These three aquifer compositions are equally to the three aquifer compositions wherein the critical head has been analysed. The results of this analysis are depicted in Figure 5.15 and Table 5.7. The results of the hydraulic head underneath the cover layer are comparable to the result of the critical head. However, the hydraulic head also slightly decreases when a complete layer is implemented. In this simulation, it seems the positive effect of gravel package is slightly more vigorous than the negative effect. For a detailed description of the existence of both the negative and positive effect reference is made to Section 5.3.2.

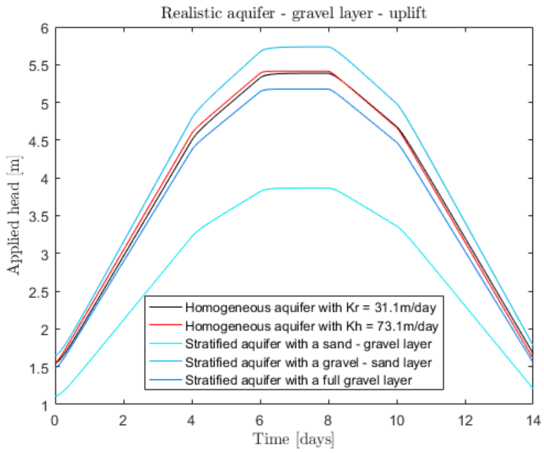


Figure 5.15: The hydraulic head at presumed exit for the realistic aquifer with a (partly) gravel layer

Table 5.7: Influence of a (partly) gravel layer on the hydraulic head in terms of percentage[%]

	difference [%]
gravel layer compositions	w.r.t K_r (31.1 m/day)
sand - gravel layer	-28.2
gravel - sand layer	6.5
full gravel layer	-3.9

5.4.3 Discontinuous clay layer

As mentioned in Section 5.3.3, the discontinuous clay layer is located between the holocene and pleistocene sand. In addition, the clay layer has two breaks. The first break is fixed and located below the river so that water can flow into the deeper layer underneath the aquifer. The second break is located underneath body of the levee where the critical length of the pipe is supposed or underneath the exit point. In both situation, the deeper layer can contribute to the groundwater flow in the upper layers. Besides, the break size is reduced once to examine the effect of the configuration of the break. The results of the discontinuous clay layer are depicted in Figure 5.16 and Table 5.7. It can be observed that the hydraulic head decreases in all aquifer compositions. Furthermore, two slight differences between the three composition can be observed. First, a smaller break obviously results in a lower hydraulic head due to the fact that a smaller break induces more resistance. However, the effect of size of the break will be of significance when it is in the order of a few meter or smaller. Secondly, the break below the exit has slightly more impact, since the seepage path is slightly longer. As a result, this aquifer composition is also more resistant to uplift and heave. For detailed description of the processes induced by a discontinuous clay layer reference is made to Section 5.3.3. This section also contains illustrations of the flow patterns.

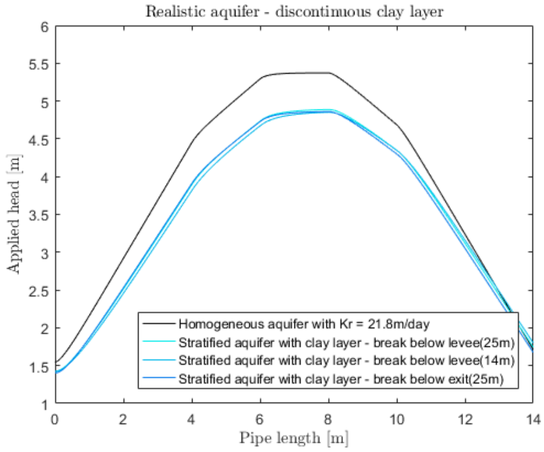


Figure 5.16: The hydraulic head at presumed exit for aquifer with a discontinuous clay layer

Table 5.8: Influence of a discontinuous clay layer on the hydraulic head of discontinuous in terms of percentage [%]

		difference [%]
clay layer compositions	break [m]	w.r.t K_r (21.8 m/day)
break below levee body	25	-8.9
break below levee body	14	-9.6
break below exit point	25	-9.5

5.5 Conclusion

Based on the results, several relevant conclusions can be drawn on the effects of anisotropy and heterogeneity for uplift and heave, and pipe development. The most valuable conclusions are summarized in this section. The effects of anisotropy and heterogeneity such as stratification, a hydraulic resistance and a hydraulic conductor are described separately. When reading the conclusions, it is important that one should be aware of the fact that both an increase in critical head and decrease in hydraulic head underneath the cover layer contribute to the piping resistance. An increase in critical head and a decrease in hydraulic head underneath the cover layer are also referred to as positive changes. All changes are relative to the simulation with a representative homogeneous aquifer with one calculation value of the bulk permeability. The bulk permeability is used in Sellmeijer's design rule to assess the safety of a levee. Because of this, the outcomes give insights into the extent to which the Sellmeijer design rules underestimates or overestimates resistance against piping. In addition, the analysis provides result of an additional method which will improve the assessment and design with Sellmeijer design rules, especially the determination of the bulk permeability.

5.5.1 Stratification

Stratification has both influence on the critical head and the hydraulic head underneath the cover layer, since stratification alters the distribution of the (ground)water. If the upper sand layers are less permeable than the deeper sand layer, it allows less water flowing into the deeper layer and it allows less water flowing from the deeper layer upwards to the exit. As a consequence the critical head increases and the hydraulic head decreases. On the contrary, if the upper layers are more permeable than the deeper sand layer, then the groundwater problem is expected to be dominated by the upper layers which allow more water flowing into the aquifer. In this case, it is expected that the critical head decreases and the hydraulic head increases. However, practical evidence shows that the holocene upper sand layers are commonly less permeable than the pleistocene deeper sand layer. As a consequence, the Sellmeijer design rules underestimates the resistance of stratified aquifers in most cases. The degree of underestimation is mainly dependent on the permeability of the upper layers. In this situation it should be noted that the higher the position of the layer the bigger the effect.

A method to take stratification into account in the Sellmeijer design rule is to enhance the determination of bulk permeability by presuming curved or radial groundwater flow to toward the exit of head of the pipe instead of presuming only horizontal flow. In that way, the bulk permeability also includes perpendicular (vertical) flow instead of parallel (horizontal) flow only. This bulk permeability (K_r) will be 5% till 10% more accurate as the currently applied method in `RisKeer`. Despite the improvement, the alternative method still underestimates the resistance of the aquifer in most cases. The underestimation can be caused by the fact that the method does not take the position of the anisotropic and/or less permeable layers into account.

5.5.2 Anisotropic behaviour

Anisotropic behaviour is principally characteristic for alluvial sediment. Due to anisotropic behaviour, the direction of the groundwater is altered and is no longer direction independent. The hydraulic conductivity tends to be lower in the vertical direction due to the layered deposition of sediments. Because the vertical permeability is lower than the horizontal permeability, the water flows to a lesser extent vertically through the aquifer. Consequently, less water can flow from the deeper layers towards the exit or head of the pipe. In other words, the effective flow area is reduced resulting in both an increase in critical head and a decrease hydraulic head underneath the cover. It can be concluded that the effect of stratification is comparable to the effect of anisotropy. In addition, anisotropic behaviour often arises in stratified sediments so that the effect of anisotropy and stratification enhance each other. For the degree of influence of anisotropy it also holds that the higher the position of anisotropic layer the bigger the influence even if the layer is significant thinner.

The Sellmeijer's design rules are constructed under homogeneous and uniform conditions, so they might be inaccurate for heterogeneous conditions. In addition, the currently used method to determine of the bulk permeability K, which is required in Sellmeijer's formula, do not take vertical permeability into account. Accordingly, anisotropy can not considered at all. As a consequence, the Sellmeijer design rules significantly underestimates the resistance of the aquifer. Besides that, it can be concluded that by using the alternative method to determine the bulk permeability, Sellmeijer design rules will approach the realistic composition up to 27% more accurate. However, the assessment is still conservative, since the alternative method does not take the position of the anisotropic layers into account.

5.5.3 A hydraulic conductor

It can be concluded that a hydraulic conductor such as gravel package can contribute both positively and negatively to the piping resistance. The effect of being positive or negative depends on the location of the gravel package. The point of investigation for pipe development is the location from where the erosion of the pipe becomes progressive. For the mechanisms uplift and heave, the point of investigation is the location of the presumed exit. A positive effect occurs if the gravel package is at least located below and beyond the point of investigation, whereas the negative effect occurs if the gravel package is fully in place of the point of investigation. The positive effect appears due to the deflection of groundwater into the gravel layer and towards the inland boundary. The negative effect appears because that the gravel package only conducts more water into the aquifer. However, if a complete gravel layer is present the positive and negative effect will almost nullify each other.

The software `RisKeer` can not consider variations within a horizontal layer, when assessing the levee with Sellmeijer design rules. The alternative method to determine the 'bulk' permeability can take variations within a horizontal layer into account. However, the outcomes of the alternative method are eclectic. When a sand package is followed by a gravel package the alternative method is much more accurate, but it still significantly underestimates the resistance. If the composition consists of a complete gravel layer, the alternative method almost approaches the result of realistic model. However, if the gravel package is at place of the point of investigation the alternative method overestimates the piping resistance by approximately 25%. In that case, the determination of the bulk permeability regarding only horizontal flow is more suitable. In brief, the effect of the gravel layer is very divergent so that compositions with a gravel package should be assessed with customized software or by specialists.

5.5.4 A hydraulic resistance

In previous analysis, the hydraulic resistance is a discontinuous clay layer located between the holocene and pleistocene sands. The discontinuous clay layer has two breaks. The first break was fixed and located underneath the river. The second break was located underneath the levee body or exit point. As a consequence, it could be determined under which properties of the second break like its position and size the deeper layer contributes to erosion process. Based on the outcomes, it can concluded that the size of (second) break will be of importance if it is relative small in the order or a few meters or even smaller. In that case, the break induces a significant amount of intrusion resistance and thereby reduce the pressure near the exit or head of the pipe. In addition, a break located beyond the point of investigation ensures that the groundwater flow in deeper layer has to take a longer path to reach exit or head of the pipe. As a result, less pressure occurs at point of investigation.

In case of a hydraulic resistance, it can be concluded that the current method to determine the bulk permeability is quite conservative, whereas the alternative method is more accurate or even nearly approaches the outcome of the realistic stratified aquifer. The latter is the case, if the break is located underneath the levee body and it has a size of 20 m or bigger. In all other cases, the alternative method underestimates the piping resistance so that it is still a bit conservative.

5.5.5 In general

On the whole, it can be concluded that anisotropy and almost all natures of heterogeneity contribute to piping resistance. Due to anisotropy and heterogeneities, the distribution and flow pattern of the groundwater alters. As a consequence, the vertical permeability will becomes more influential or will even be the key property. However, the vertical permeability is not included in currently applied method to determine the bulk permeability. By considering the vertical permeability, the Sellmeijer design rule becomes more accurate. The proposed alternative method to determine the bulk permeability includes vertical flow. As a result, the method is in almost all case more accurate and often still quite conservative. The latter is the case, because no significance is given to position of the layer, while it is revealed that the higher the layer is located the bigger the impact. In order to improve this method, more insight must be gained in the importance of the position of a layer. In addition, all above conclusions do **not** apply if a conductor such as a gravel layer is fully at place of the point of investigation. In that case, the aquifer seems to be less resistant to piping.

Table 5.9 shows an overview with ranges of the degree of influence of anisotropy and each type of heterogeneity with respect to the current assessment with Sellmeijer's design rules for both the critical head and the hydraulic head underneath the cover layer. This table is constructed on basis of the result of both the elementary study and realistic study. This table should be used to get an indication of the degree of influence of anisotropy and heterogeneity, since not all possible compositions are modulated. Besides, both studies revealed that the degree of influence is highly dependent on hydraulic properties, position, orientation and size of the deviate layer(s). Thus, the influence can be larger or smaller in practice. Furthermore, the results of a hydraulic conductor are depicted in orange boxes, since the influence can be both negative or positive.

Table 5.9: The ranges of the degree of influence of anisotropy and heterogeneity on both the critical head and hydraulic head at presumed exit in terms of percentage[%]

	influence on the critical head (ΔH_c)	influence on the hydraulic head (ϕ_o)
anisotropic behaviour	+25% – +45%	-5% – -15%
stratification	+5% – +10%	-3% – -4%
a hydraulic conductor	-3% – +150%	+7% – -30%
a hydraulic resistance	+1 – +22%	-1 – -10%

6 | Discussion

6.1 Introduction

This chapter discusses the results of this research and the circumstances in which the results are valuable and valid. The main aim of this thesis is to improve the piping assessment with regard to a better determination of the bulk permeability by analysing and modelling the effect of heterogeneity and anisotropy. In order to answer this objective, the most important aspects of the literature and the results are summarized and discussed. Firstly, the influences of anisotropy and heterogeneity on both uplift and heave and pipe development are summarized and discussed (Section 6.2). In addition, the influences are divided into positive (favourable) or negative (unfavourable). Secondly, it discusses to what extent the determined permeability, modelled permeability and bulk permeability differ and how these permeabilities can contribute to an improvement of the assessment (Section 6.3). Finally, the limitations of this research are discussed, focussing on the difference between a model and the real world, see Section 6.4.

6.2 The influence of anisotropy and heterogeneity

The effect of anisotropy and heterogeneity such as stratification, hydraulic resistance and hydraulic conductors have extensively been studied for both uplift and heave, and pipe development. Furthermore, the influence of additional components have been studied, in the sensitivity analysis. For instance, the influence of the properties of the blanket and the influence of a no flow boundary conditions which can represent a geological resistance. All these components contribute either positively or negatively to the piping resistance of a levee. The influences are described by means of the pipe development. However, if a component has a significantly different (degree of) influence to uplift or heave then it is explicitly stated. The components are described from effective to ineffective. In conclusion, an overview of all components is given. The components are divided into two different categories: a positive effect or a negative effect. Besides, the position in the table of a component determines the degree of its influence. In general, the higher its position, the bigger its impact. The influences of the hydraulic conductor and resistance are strongly dependent on the dimensions of the components. Their influence is therefore considered to be less effective.

6.2.1 Anisotropy

Anisotropic characteristics greatly influence the critical head and hydraulic head. In addition anisotropy is principally characteristic for alluvial sediment. As revealed by the literature study, the vertical hydraulic conductivity is often much lower than the horizontal conductivity ($K_v > K_h$). The difference can range from a factor of 2 till a factor of 100. In this thesis, a factor of 10 was assumed in the elementary study and a factor of 4 was assumed in the realistic study. Both factors are assumed on basis of the literature, soil maps and conversations with specialist. However, little information about the degree of anisotropy in (sub)soil is known. The value of 4 and 10 are therefore rough assumptions. These values seem to be conservative, however, also lower values exist. Besides, the degree of anisotropy of each layer can differ, while in the realistic study the same degree of anisotropy was added to each layer. As a consequence of those assumptions, the effect of anisotropy might be overestimated or underestimated. Nevertheless, anisotropy ($K_v > K_h$) increases the piping resistance of an aquifer and it is very likely that horizontal permeability in top layers is higher than vertical permeability due to the sedimentation of soil particles in former river or tidal flats. Moreover, the effect can be enhanced if the aquifer consists of more layers.

6.2.2 Stratification

Stratification is a formation of layered sediment deposits, whereby geological features are modified. Stratification is primary characteristic of the subsoil. Stratification positively influences the critical head. In this thesis, the effect of stratification in sandy aquifer by adding less permeable holocene (top) layer and the more permeable pleistocene (deep) layers is researched. In that situation, the effect of stratification is advantageous. However, the opposite situation is not analysed. The opposite situation is deliberately not analysed because the manual to schematise subsoil gives significant higher permeability values of the deeper pleistocene layers. In addition, the stratified aquifer composition with a very permeable middle layer also gives advantageous results. As a consequence, it is proved that stratification has an advantageous influence. In this thesis, the aquifer consist of only 3 or 4 different layers, while in reality much smaller and much more layers can be observed. These results do not provide insight into the effect of very stratified subsoil. It is expected that more layers have more positive effect due to the additional transitions. As a result, more resistances occurs. Finally, it should be noted that anisotropy and stratification often coexist. In reality, it is therefore difficult to assess the effect of stratification and anisotropy separately.

6.2.3 A hydraulic conductor

A hydraulic conductor such as a gravel package influences the critical head both positively and negatively. A gravel layer with a permeability of 250 m/day was implemented because it was assumed that the value of 250 m/day is an average permeability of a gravel layer. Nevertheless, the permeability of gravel layer is greatly variable. The permeability will vary between the 25 and 2500 m/day. As a consequence, the effect of the gravel layer can be significantly greater or smaller. In addition, very coarse sand elements might also serve as a conductor. The degree of influence of a conductor is therefore difficult to assess. Furthermore, relatively large and thick gravel elements were added. The effect of smaller gravel elements will obviously be significant smaller. The results of the hydraulic conductors should therefore serve as indicators.

6.2.4 A hydraulic resistance

A hydraulic resistance such as clay, peat or silt package will hinder or withhold the water flow and thereby change the flow pattern. Three hydraulic resistances will be discussed; a vertical resistance at top of the aquifer, discontinuous horizontal layer between the holecene and pleistocene sand, and a blocking geological element after and closely to the presumed exit.

6.2.4.1 A vertical layer at top of the aquifer

A vertical layer at top of the aquifer has a positive influence on the critical head. This vertical layer changes the streamlines by hindering and blocking the water flow. The degree of influence is dependent on the length, the permeability and the horizontal position of the layer. For instance, a long impermeable layer close to the presumed exit will highly decrease the pressures near the exit. The results of a vertical layer can differ from real cases because only very tall vertical layers were added. It is therefore possible that the effects of thicker vertical layers are greater. In addition, the vertical layer will also prevent **horizontal** erosion of the pipe so that the pipe will erode diagonally or vertically. However, vertical or diagonal erosion cannot be considered in D-Geo Flow. As a consequence, the results of only the first few meters of the pipe development are reliable.

6.2.4.2 A discontinuous horizontal resistance

A discontinuous horizontal resistance can positively influence the critical head in a large or small matter. The influence is greatly depending on the size of the break and the dimensions and permeability of the resistance. So if the size of break is in the order of a few meters or smaller, the influence of the discontinuous will be very huge and only the layer above the horizontal layer will have effect. Nevertheless, the effect of the size of the break is researched by only one reduction. In order to study the effect of the size of the break, the size should be reduced step by step. In addition, the permeability and the thickness of the resistance remain constant in each realistic simulation unless the elementary study shows that both parameters decrease or increase the effect of a resistance. The effect of a resistance is therefore dependent on the dimensions and permeability. It is therefore hard to generally quantify the effect of a resistance.

6.2.4.3 A blocking geological element

A blocking geological element after and closely to the exit seems to have only highly negative influence on the hydraulic head. The blocking geological element is modelled as a no flow boundary close to the exit. Due to this blocking element, the water will be reflected so that higher pressure occurs at the presumed exit. Nevertheless, the effect on the critical head is limited due to fact that the simulation of pipe erosion seems to be dominated by the exit. In this thesis, the blocking element seems to have only influence on the hydraulic head, as is depicted in Table 6.1 in cyan. The blocking geological element is simulated as impermeable element. This might be a very rough assumption since slightly permeable elements will also reflect water and thereby increase the potential hydraulic head. The effect of a blocking element can be less in practice.

6.2.5 Blanket

The permeability of a blanket highly influence the likelihood of uplift. However, it seems that the blanket permeability barely influences the critical head. When the critical head is simulated, the model is dominated by the exit so that the blanket has almost no effect. In addition, the effect of resistance caused by fluidized sand grains in the uplift channel is hardly investigated. Nevertheless, the effect on the hydraulic head is of significance. A more permeable blanket decreases the hydraulic head and consequently the likelihood of uplift, since it allows more water to leak through the blanket thus resulting in lower pressure underneath the blanket. This process was also observed by van der Hulst (2017). Furthermore, a thicker blanket will increase the hydraulic head, since it is more difficult for water to flow through the blanket. On the contrary, a thicker blanket causes more resistance to uplift due to its greater weight. The influence of the thickness of the blanket is not elaborated in detail and therefore excluded in the table. The influence of the permeability of the blanket is depicted in cyan, since it seems that it only influences the likelihood of uplift and heave.

Table 6.1: An overview of the influences of all components

positive influence	negative influence
+ anisotropy	- a blocking geological element after and closely to the exit
+ a very permeable blanket	- a hydraulic conductor fully before the p.o.i.*
+ a hydraulic conductor below and beyond the p.o.i.*	
+ stratification	
+ a vertical hydraulic resistance in top of the aquifer	
+ a discontinuous horizontal hydraulic resistance	
+ a complete hydraulic conductor	

* p.o.i. means point of investigation

6.3 The permeability in a piping problem

The permeability of the aquifer is essential to determine the amount of groundwater flow in the aquifer and it therefore also determines the piping resistance. In order to improve the safety assessment of piping by means of including heterogeneity and anisotropy, the permeability of the aquifer should be measured, determined and calculated in a different way. Unfortunately, the permeabilities assigned in a numerical model (cm or m) or the calculated bulk permeability (entire aquifer) are commonly not quite similar to the actual permeability of the aquifer due to the necessity of simplifications and the method chosen to determine the actual permeability. See Figure 6.1 for a visual representation of the relation between the real world, the numerical model and the design rules. The visualisation of the real world contains a lithographic cross profile of only the Holocene soil layers at IJzendoorn drawn by Taal (2015). The cross profile shows heterogeneity on the small scale of cm. However, heterogeneity also occurs at the scale of mm in reality. A numerical model can contain heterogeneity on the scale of m. The design rules do not allow heterogeneity, but require an uniform and homogeneous aquifer. The bulk permeability can be determined in two different ways. First, a schematisation of the subsoil is made on the basis of soil measurements like probes or soil mapping like REGIS. After this, the average weighted (bulk) permeability can be calculated. These two steps are given with the cyan arrows. The other way is to derive the bulk permeability directly from permeability measurements, see the darkcyan arrow. In this thesis was focused on the cyan arrows, especially the arrow between numerical model and design rules. This section discusses to what extent the determined/measured, modulated and calculated bulk permeability can be improved to contribute to a more accurate assessment of the aquifer.

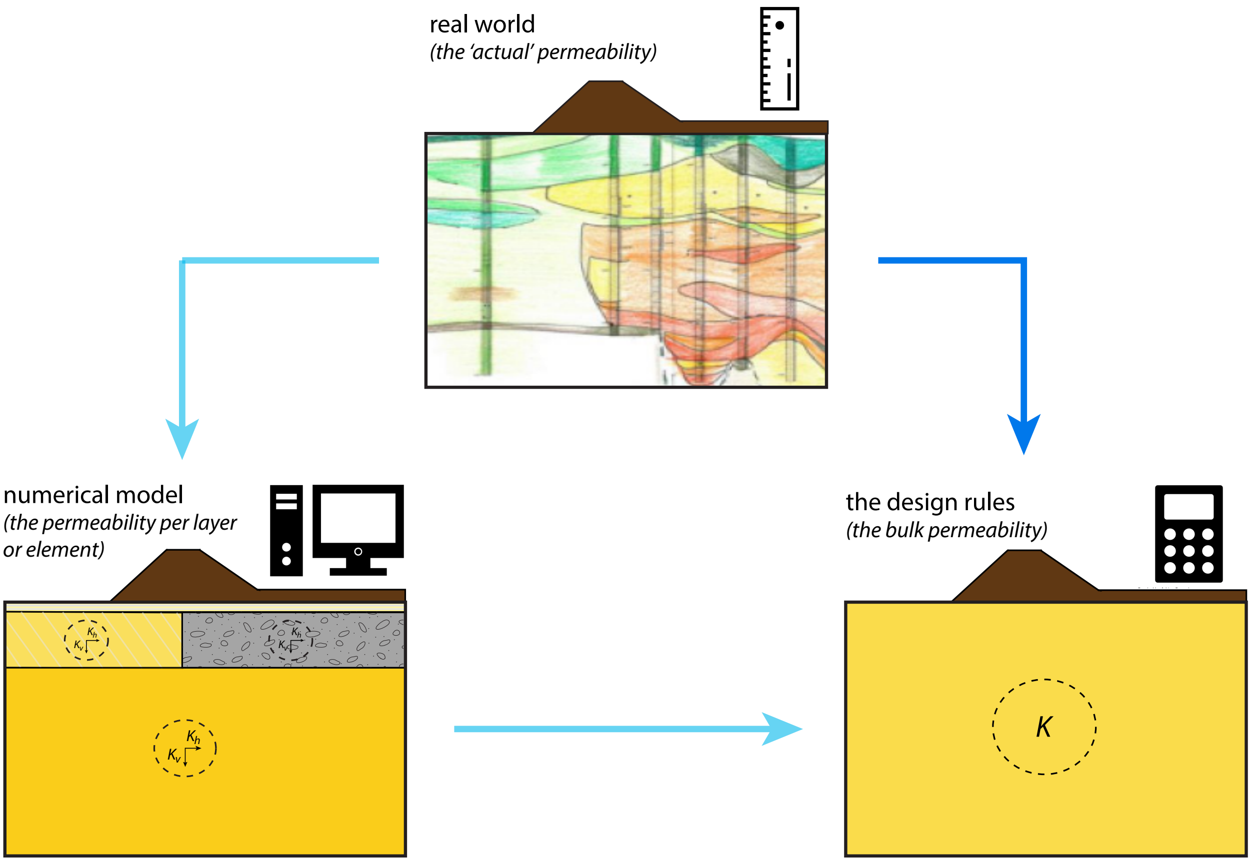


Figure 6.1: A visual representation of the relation between bulk permeability, the permeabilities in a multi-layered approach and the actual permeability

6.3.1 Determination of the permeability

The 'actual' permeability can be determined by different methods or techniques. The determination methods range from quick and simple methods to expensive and time-consuming methods. In addition, the quick and simple methods often measure the permeability of a small soil volume (cm^3) or determine the permeability on the basis of the rough correlation between other soil properties, whereas the expensive and time-consuming methods measure the permeability of a larger soil volume (m^3). As a consequence, these methods provide significant different results, see Section 2.4.2.5. The selection of the determination method is therefore depending on soil volume involved and the degree of simplification. The (numerical) schematisation often consist a (multi-)layered composition so that a (anisotropic) permeability per layer of element on the scale of meters or maybe decimeters needs to be determined, while the design rules requires one (isotropic) calculation value of the bulk permeability. The following two sections discuss the numerical approach and analytical approach in which the bulk permeability is required.

6.3.2 Numerical approach (multi-layered approach)

A (numerical) schematisation is a simplification of the reality. Due to the simplified schematisation of the reality, very small soil layer or elements (in the order of cm or mm) are neglected. A schematisation often consists of a stratified or (multi-)layered aquifer which is composed on the basis of for instance, probes or WTI-SOS scenario's provided by D-Soil model. After this, hydraulic properties such as the permeability should be assigned to each element or layer. Because hydraulic measurement methods often do not provide insights into the hydraulic properties of each layer or element, the properties should be determined on the basis of the correlations method, soil maps or legal guidelines. It can be discussed whether these values are representative for a specific research location. Although these values are generally quite reasonable for common soil types, they often considerably deviate from the actual permeability due to heterogeneity on a smaller scale. The heterogeneity on a smaller scale can be embodied due to the assignment of anisotropic characteristics. However, anisotropic characteristics of soil layer can often only be obtained on the basis of soil maps such as lithographic maps. Nevertheless, the addition of anisotropic characteristics to each layer and element will lead to a better approach to the location-specific groundwater problem. The site-specific groundwater problem can be assessed more accurately if a (continuous) permeability profile is measured, so that the (numerical) schematisation is not only based on probes, soil maps or guidelines, but also on the permeability of the layer. These measurement techniques are relatively expensive and time-consuming so that it should be analysed if the accuracy of the measurements weight up against the additional measurement costs.

6.3.3 Analytical approach (one layer approach)

The bulk permeability is one calculation value of the permeability that represents the entire aquifer so that it can be used in Sellmeijer's formula to asses or design a levee. The bulk permeability can be determined directly from large scale in-situ measurement or by calculating the weighted average permeability of a multi-layered (heterogeneous) aquifer. The (multi-)layered aquifer is a schematisation of the reality. The multi-layered or stratified aquifer is often the input in numerical models, see Figure 6.1. In the safety assessment performed in `RisKeer`, the bulk permeability is calculated by taking the weigthed average of the horizontal permeabilities. Hereby, it is assumed that the groundwater flow is only horizontal (Förster et al., 2017). In this thesis, this assumption is debated because in deep, heterogeneous and anisotropic aquifers water flows rather curved or radially towards the exit or head of the pipe instead of only horizontally. As consequence, an alternative method is supposed to determine the bulk permeability. In this method, the bulk permeability is computed from the geometric or logarithmic mean of the vertical and horizontal permeability. This alternative method, with one exception, approaches the equivalent heterogeneous and anisotropic aquifer more accurately than the currently applied method. However, when a gravel layer is completely at front of the critical point then the alternative method is less accurate. Besides, the alternative method is still conservative for some aquifer compositions, especially the anisotropic composition or the composition with a gravel element. The deviations in results occur due to two imperfections. First, no additional weight is given to the permeability of anisotropic ($K_h > K_v$) top layers. In the case of anisotropic top layers, a significant amount of the groundwater will only flow horizontally through the top layer towards the exit instead of vertically into the deep layer and towards the exit. In that case, the horizontal permeability of the top layer will become leading. Furthermore, the influence of a gravel element is not fully correctly considered. Since the effect of the gravel layer is dependent on its horizontal location, while the the horizontal location of the gravel layer is not explicitly included. The alternative method should be improved if more information is gained in the influence of the positions of a gravel or a anisotropy layer. In conclusion, the assumption of curved or radial flow towards the exit or head of the pipe instead of only horizontal flow is better for deep, heterogeneous and/anisotropic aquifers. Nevertheless, the alternative method to determine the bulk permeability of deep, heterogeneous and/anisotropic aquifers can still be improved.

6.4 The model study versus the real world

A model is used to represent, usually in a simplified way, the complex and detailed reality. In order to simplify the reality, a model embodies some essential and interesting aspects of that reality, but not all of it. As a result, a model is by definition incomplete. The central question is therefore: To what extent do the results of the model apply on the complex reality?

In this thesis, the numerical model D-Geo Flow is used, which also contains some questionable aspects. Firstly, the numerical model assumes a two-dimensional situation. By assuming 2D, the spatial variability in the longitudinal direction of the levee is neglected. Nevertheless, this assumption is according to the current assessment, so that results are convenient and appropriate. However, one should keep in mind that spatial variation in the longitudinal direction will also influence the piping resistance. Secondly, the numerical model runs for hours or days or even becomes unstable if the geometry of the aquifer is too complex. As a consequence, very small elements or layers should be neglected. This seems to be of little influence, since small scale effects are often dominated by the large scale effects (Section 2.4.2.5). Furthermore, the software can only calculate pipe erosion along a defined horizontal path. However, heterogeneities such as natural barriers in the top of the aquifer will influence the pipe path. As a consequence, the pipe path will not be perfectly smooth and horizontal in reality. Finally and most importantly, D-Geo Flow is developed and validated on the basis of simple cases. So it is not completely clear whether the model provides accurate results in case of complex and stratified conditions. Nevertheless, these results appropriately predict or approach the effect of heterogeneities and can act as the primary focus of further experiments, but they should used with care. In addition, a model study can always show whether more ground investigation weighs up against the implementation of reinforcement measures.

7 | Conclusions & Recommendations

7.1 Introduction

In this thesis, a model study is performed to analyse the effects of anisotropy and heterogeneity in the piping sensitive layer. These effects have been investigated in order to improve the current safety assessment, especially with Sellmeijer's formula, on the failure mechanism piping. This chapter presents the final conclusions and recommendations based on the model study performed in this thesis. In Section 7.2 the primary conclusions are described by means of the research questions. More detailed and specific conclusions can be found in the corresponding chapters. Section 7.3 outlines recommendations for further research based on this study.

7.2 Conclusions

This section outlines the final conclusions by means of the research questions. The research questions were defined to achieve the following research objective:

The research objective is to improve the assessment of the bulk permeability of the piping sensitive aquifer, used in Sellmeijer's formula, by modelling and analysing the effects of anisotropy and heterogeneity in the subsoil

7.2.1 What kind of spatial variabilities are present in the subsoil?

Due to geomorphological processes such as sedimentation, the subsoil is certainly not perfectly homogeneous. As a consequence, different soil types with different hydraulic properties exist in the subsoil. In addition, the hydraulic properties of the soil (layer) often differs with direction (**anisotropy**). For instance, the vertical hydraulic conductivity of soil layer differs from the horizontal mainly due to vertical difference in texture, structure and porosity due to layered deposition and biological activities. Besides, the effects of anisotropy on a large scale (m) can be the effects of **heterogeneity** on a small scale (cm or mm). As a consequence, an aquifer is commonly anisotropic and heterogeneous. A heterogeneous aquifer can consist of one or a combination the following types of heterogeneity, which describes a variation in physical properties between given points or elements:

- Stratification
- Hydraulic resistances
- Hydraulic conductors

7.2.2 What are the effects of anisotropy and heterogeneity, in particular variations of hydraulic conductivity, in the aquifer on the critical hydraulic head or gradient?

Table 7.1: Influence of anisotropy and heterogeneity on the critical gradient (ΔH_c) in terms of percentage

spatial variability	the representative bulk permeability K_h [m/day]	difference in (ΔH_c) [%]
stratification (isotropic)	26.1	↑ **9.4**
anisotropy	26.1	↑ (27.8 –) **44.3**
hydraulic conductor (a continuous layer)	73.1	↑ **16.0**
hydraulic conductor (element)	73.1	↓ **3.9** – ↑ **150**
hydraulic resistance (a discontinuous layer)	25.1	↑ (1 –) **22**

Table 7.1 shows that both anisotropy and heterogeneity have effect on the critical head or gradient. However, the magnitude of the effects is contingent on the degree of anisotropy and the number of anisotropic layers or the properties and location of the heterogeneity. The increase (↑) or the decrease (↓) in critical head are given in terms of percentage for anisotropic and/or heterogeneous aquifer with respect to the outcome of Sellmeijer's formula in which a representative bulk permeability (K_h) is applied. An increase (↑) in critical head implies that the current assessment with Sellmeijer's formula underestimates the piping resistance of the aquifer, while a decrease (↓) implies that Sellmeijer's formula overestimates the piping resistance of the aquifer. However, it is difficult to generally assess the effects of anisotropic and heterogeneous aquifers since the effects of anisotropy and heterogeneities are highly variable. This variability occurs because the effects of anisotropy and heterogeneity are dependent on multiple characteristics such as physical properties, dimensions, locations and the numbers of layers or elements.

The processes causing the effects of anisotropy and heterogeneity are briefly described below:

↑ **Stratification**
Stratification increases the critical head, especially when the upper (sand) layers are less permeable than the deeper (sand) layers. In that case, the upper layers allow less water to flow towards the deeper layer. In addition, the upper layers also allow less water to flow from the deeper layer towards the exit or head of the pipe. Furthermore, resistance occurs at the transitions between the different layers. The effect of stratification will be enhanced when the upper (sand) layers are less permeable or when (some) layers are anisotropic.

↑ **Anisotropy**
Anisotropy or directional dependency ($K_h > K_v$) also increases the critical head and therefore increases the piping resistance. As a consequence of anisotropy, water flows to a lesser extent vertically through the aquifer so that less water can flow from the deeper layer of the aquifer towards the exit or head of the pipe. As a result, lower pressures occur near the exit. The increase in the critical head will be larger if the degree of anisotropy of elements or layers is larger or if more soil layers are anisotropic.

↕ **Hydraulic conductors**
As indicated with the ↕ sign, the effect of hydraulic conductors can be negative or positive depending on its location. A decrease occurred when a hydraulic conductor is completely located in the front of the critical pipe length. In all other cases, an increase occurs. The increase in critical head appears due to the deflection of groundwater into the gravel layer. An amount of groundwater is therefore flowing towards the inland boundary instead of towards the exit or head of the pipe. The decrease in critical head appears when a gravel layer only allows more water to flow into aquifer and it does not bifurcate the water flow so that water flows nearly undisturbed towards the exit.

↑ **Hydraulic resistances**
Hydraulic resistances such as clay, peat or silt layers or elements increase the critical head due to the blockage and the hindrance of the water flow. As a consequence, the water flow experiences more resistance and/or the seepage length increases. In addition, a hydraulic resistance at the top of the aquifer does not only increases the critical head, but it also prevents horizontal erosion.

7.2.3 What are the effects of anisotropy and heterogeneity, in particular variations of hydraulic conductivity, in the aquifer on the hydraulic head underneath the cover layer at the presumed exit point?

Table 7.2: Influence of anisotropy and heterogeneity on the hydraulic head underneath the cover layer (ϕ_o) in terms of percentage

spatial variability	the representative bulk permeability K_h [m/day]	difference in (ϕ_o) [%]
stratification (isotropic)	26.1	↓ **3.7**
anisotropy	26.1	↓ (8.5 –) **13.4**
hydraulic conductor (a continuous layer)	73.1	↓ **3.9**
hydraulic conductor (element)	73.1	↑ **6.5** – ↓ **28.2**
hydraulic resistance (a discontinuous layer)	25.1	↓ (8.9 –) **9.6**

Table 7.2 shows that the effects of anisotropy and heterogeneity on the hydraulic head underneath the cover layer (ϕ_o) at the presumed exit point. These effects are opposite to the effects of spatial variabilities on the critical head. However, in this situation a decrease (↓) is favourable (positive) and a increase (↓) is unfavourable (negative) since the hydraulic head is a load parameter while the critical head is a resistance parameter. A decrease in hydraulic head implies that an anisotropic and/or homogeneous aquifer induce a lower load or hydraulic head, while an increase implies that anisotropic and/or heterogeneous aquifer induce a higher load or hydraulic head. The decrease (↓) or the increase (↑) in hydraulic head are given in terms of percentage for anisotropic and/or heterogeneous aquifer with respect to the outcome of the simulation with the standard geometry and a bulk permeability (K_h). Besides, the degree of the effect on the hydraulic head and the critical head differs by roughly a factor of 2. This difference occurs because both parameter are calculated on the basis of other functions. The processes causing the effect of anisotropy and heterogeneity are similar to the processes described in previous section. As a consequence, it is also difficult to asses the effect of anisotropy and heterogeneity on the hydraulic head at the presumed exit because it depends on multiple characteristics as well.

7.2.4 How can heterogeneity and anisotropy in the subsoil be included in the safety assessment of piping, especially in the bulk permeability used in Sellmeijer's formula?

A number of conclusions are drawn below with regard to the improvement of the current safety assessment of piping, in particular the determination the bulk permeability required in Sellmeijer´s formula by including anisotropy and heterogeneity.

- First of all, the assessment of the bulk permeability of a heterogeneous and/or anisotropic aquifer should consider both vertical and horizontal flow in order to take the variation of hydraulic conductivity in the aquifer into to account, especially anisotropy.
- The proposed alternative method to determine the bulk permeability, with only one exception, approaches anisotropic and/ore heterogeneous up to 27 % more accurate, compare Table 7.1 with Table 7.3. However, in some cases it still underestimates the piping resistance. In addition, when a gravel layer is completely located at the front of the critical pipe length, the alternative method approaches the heterogeneous aquifer less. It overestimates the piping resistance of the aquifer. Within the alternative method, the bulk permeability (K_r) is computed from geometric mean of the horizontal and vertical permeability by presuming curved or radial flow towards the exit or head of the pipe.

Table 7.3: Influence of anisotropy and heterogeneity on the critical gradient (ΔH_c) when the K_r-value was implemented

	the representative bulk permeabilities			
spatial variability	K_h [m/day]	K_v [m/day]	K_r [m/day]	difference in (ΔH_c) [%]
stratification (isotropic)	26.1	21.2	23.5	↑ **5**
anisotropy	26.1	5.3 – 13.8	11.8 – 19.0	↑ (12 –) **18.7**
hydraulic conductor (a continuous layer)	73.1	28.8	45.9	↑ **1.1**
hydraulic conductor (element)	33.9	28.6	31.1	↓ **30** – ↑ **90**
hydraulic resistance (a discontinuous layer)	25.1	18.9	21.8	↑ (1 –) **17**

Table 7.3 shows the differences in critical head of the anisotropic and/or heterogeneous aquifer with respect to the standard levee proposed by Sellmeijer by using the K_r-value. Furthermore, the horizontal, vertical and radial bulk permeability computed are given.

- This thesis revealed that the top layers contribute more to the erosion process. For that reason, the (alternative) method to determine the bulk permeability can be improved, if more weight is given to the top layers, in particular to anisotropic top layers. More specifically, if very anisotropic or much less permeable top layers are present, the deeper layer(s) barely contribute to the erosion process so that the permeability of the deeper layer is not of importance.

- The determination of the actual permeability should also be changed or enhanced in order to assess the piping resistance of an aquifer, in particular deep aquifer. Large scale methods should be used instead of small scale methods. Because small scale methods represent only local parts of the aquifer and they therefore provide often too low values of the actual permeability. The large scale method should ideally perceive not only a single value of the bulk permeability, but also provide insight into stratification, hydraulic resistances, hydraulic conductors and especially anisotropic behaviour.

7.3 Recommendation

With the conclusions in mind, some recommendations can be made. The recommendations are twofold. The first set of recommendations focusses on the improvement of the current assessment. The second set of recommendations describes some essential and influential aspects that should be investigated in future research.

7.3.1 The safety assessment of piping

The Dutch assessment mainly consists of 3 steps: a simple assessment, the detailed assessment containing Sellmeijer's formula and if necessary a customized assessment. An assessment can obviously only be accomplished if sufficient data is collected, especially the permeability of the aquifer. On the basis of this thesis, several recommendations can be drawn for the detailed assessment and customized assessment. In addition, recommendations on the collection or determination of the actual permeability are drawn.

7.3.1.1 The determination of the permeability

In order to improve the assessment of piping, additional or different data are required. As a result, other methods to derive these data might be needed. Several recommendations regarding the determination of the permeability are drawn below:

- In the case of deep aquifers, the method to determine the permeability must also be suitable for measuring a large soil volume. Otherwise, false K-values will be derived due to the scale effect, especially too low K-values. The HPT/MPT method is currently the most advanced large scale method that derives a continuous permeability profile so that stratification, a hydraulic resistance and a hydraulic conductor can be observed as well.

- Large scale methods are time-consuming, relatively expensive and provide hardly any insight into anisotropic behaviour. To get insight into anisotropic behaviour of (a) soil layer(s), especially the top layer, laboratory methods or correlation methods can be used. However, the laboratory tests represent only local part of the aquifer and the correlation test might be quite inaccurate so that the outcomes of both methods should be used with care.

- If the horizontal permeability of top layers is much higher than the vertical permeability then the deeper layer will barely contribute to the erosion process. In that case, the permeability of the deeper layer barely influences the critical head so that deviations in the permeability of the deeper layer hardly influences the critical head. The permeability of the deeper layer can than be determined by quick and simple correlation methods.

7.3.1.2 The detailed assessment

The detailed assessment contains the design rules of Sellmeijer. Within these rules, one calculation value of bulk permeability which embodies the entire aquifer is required. Several recommendations regarding the detailed assessment, especially the method to calculate the bulk permeability, are drawn below:

- Modify the method to calculate the bulk permeability, in particular for anisotropic and/or heterogeneous aquifers, by taking the geometric or logarithmic mean of the horizontal and vertical permeability instead of taking only the mean of the horizontal permeability. In that way, curved flow towards the exit or head of the pipe can be considered.

- In the case of relatively thin and homogeneous aquifers, the seepage flow through the aquifer is mainly horizontal (Ernst, 1963) so that it is sufficient to take the (weighted) mean of the horizontal permeabilities.

- In order to assess an aquifer with a potential conducting element at front of the critical point. It is recommended to introduce an advanced aquifer scenario and determine the likelihood of that aquifer composition. Afterwards, a relatively simple customized assessment can be performed to estimate the influence of potential conducting element on the safety of a levee.

7.3.1.3 The customized assessment

The customised assessment should be performed in a numerical model that can determine the (critical) head in anisotropic and/or heterogeneous aquifer.

- The numerical model D-Geo Flow is suitable for the assessment of piping, since it can perform 2D calculations with complex and stratified foundations. In addition, anisotropic elements can be implemented by assigning the horizontal and vertical permeability separately. Furthermore, D-Geo Flow is the only numerical model that includes a special piping module.

7.3.2 Aspects for further research

The recommendations for further research include four primary aspects: anisotropy, the position of the layer, the blanket and the D-Geo Flow model. More research of these aspects will greatly improve the knowledge on the failure mechanism piping.

7.3.2.1 Anisotropy

This thesis revealed that anisotropy is a primary cause of the underestimation of the piping resistance of a aquifer. In addition, the literature study revealed that the vertical hydraulic conductivity can be a factor of 100 or even more lower than the horizontal conductivity and thereby anisotropic behaviour is characteristic for alluvium sediments. It is therefore recommended to investigate the degree of anisotropy in alluvium sediments and if possible to assign a degree of anisotropy to each soil type with the same hydraulic characteristics within their lithographic class. In that way, anisotropy is insightful and can be included in piping problems.

7.3.2.2 Position of the layer

An other important aspect in a stratified aquifer is the position of a particular layer. This model study proved that the top layers will have more impact than the underlying layers, even when the top layers are much smaller. This phenomena is caused by the simple fact that water must first flow through the top layers, before it can infiltrate into the underlying layers. As a consequence, the deeper layer contributes less to the erosion process. It is therefore recommended to investigate to what extent upper layers contribute to the erosion process. If more insights are gained into the importance of the positions of the layer, then the method to determine the permeability can be improved. An example of a potential improvement would be to assign a weighting factor to each layer.

7.3.2.3 Blanket

The sensitivity analysis has shown that the blanket mainly influences the hydraulic head potential underneath the cover layer instead of influencing the critical head. In addition, the model study has also shown that the resistance in the uplift channel mainly thwarts the initiation of pipe erosion, but it will nearly influence the critical head. The latter does not correspond to the literature, because according to the literature the hydraulic head used in Sellmeijer rules may be reduced by 0.3 times the thickness of the blanket to correct for the resistance of fluidized sand in the uplift channel. It is therefore recommended to perform more simulations in D-Geo Flow model, whereby a pressure is assigned to the heave boundary and to check whether this pressure affects the pipe development. Besides, it could be validated whether the D-Geo Flow model predicts the critical head correctly if a pressure is assigned to the heave boundary in a stratified aquifer. Lastly, the effect of the permeability and thickness of the blanket on the likelihood of uplift should be investigated in more detail.

7.3.2.4 D-Geo Flow model

This study was entirely performed in the numerical D-Geo Flow model.The D-Geo Flow model is a recently developed numerical model which is validated on the basis of simple compositions. In that way, it is not fully clear whether the results of complex and stratified compositions are accurate. It is therefore recommended to compare the results with more realistic cases or to verify them with laboratory experiments. However, it will be quite difficult to perform experiments with heterogeneous aquifer. Furthermore, the software has some limitations such as the impossibility to calculate vertically or diagonally erosion. Besides, it is quite time-consuming or even impossible to construct very complex compositions. For that reason, it is recommended to continuously improve and validate the D-Geo Flow model. Nevertheless, the software is currently suitable for common engineering and can provide more insight and knowledge on piping in heterogeneous foundations.

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Appendices

A | Original design formula by Sellmeijer

This appendix contains the original mathematical rules for piping assessment formulated by Sellmeijer in 1989. The rules were developed, in the context of TAW research, based on observation of piping (TAW, 1999).

$$\Delta H_c = \alpha\ c\ \frac{\gamma_p}{\gamma_w}\ tan(\vartheta)(0.68 - 0.10\ ln(c))L \tag{A.1}$$

$$\alpha = \left(\frac{D}{L}\right)^{\left(\frac{0.28}{\left(\frac{D}{L}\right)^{2.8}-1}\right)} \tag{A.2}$$

$$c = \eta\ d_{70}\ \sqrt[3]{\frac{1}{\kappa L}} \tag{A.3}$$

Where:

ΔH_c	critical hydraulic head $[m]$
γ_p	(apparent) saturated unit weight of soil particles $[\frac{kN}{m^3}]$
γ_w	unit weight of water $[\frac{kN}{m^3}]$
ϑ	bedding angle [°]
η	White's constant [-]
κ	intrinsic permeability of the aquifer $[m^2]$
d_{70}	grain diameter for which 70 percent of particles are smaller $[m]$
D	aquifer depth $[m]$
L	the horizontal seepage length under the levee $[m]$

In (TAW, 1994) is a practical concept is developed for the design and the assessment of a levee. The concept was based on above formula, especially with regard to the manner in which uncertainties in the parameter choices should be discounted.

B | Typical range of soil properties

This appendix summarises some typical ranges of the properties of the soil required for the seepage modelling in D-Geo Flow. Some of these ranges are also mentioned in the literature review, especially section 2.4.2. The ranges are extracted from different sources that are depicted below Table B.1.

Table B.1: Overview of typical ranges of soil properties

soil type		hydraulic conductivity $[\frac{m}{day}]$ [a]	grain size $[m]$ [b]	porosity[-] [c]	compressibility $[\frac{m^2}{N}]$ [e]
gravel	coarse	25 - $2.5{\cdot}10^4$	$2.0{\cdot}10^{-2}$ - $6.3{\cdot}10^{-2}$	0.24 - 0.36	$1{\cdot}10^{-8}$- $5.2{\cdot}10^{-9}$
	medium		$6.3{\cdot}10^{-3}$ - $2.0{\cdot}10^{-2}$	0.24 - 0.44	
	fine		$2.0{\cdot}10^{-3}$ - $6.3{\cdot}10^{-3}$	0.25 - 0.38	
sand	coarse	10 - 50	$6.3{\cdot}10^{-4}$ - $2.0{\cdot}10^{-3}$	0.31 - 0.46	$1{\cdot}10^{-7}$- $1{\cdot}10^{-8}$
	medium	1 - 5	$2.0{\cdot}10^{-4}$ - $6.3{\cdot}10^{-4}$	0.29 - 0.49	
	fine	1 - 3	$6.3{\cdot}10^{-5}$ - $2.0{\cdot}10^{-4}$	0.25 - 0.53	
silt		10^{-4} - 1	$2.0{\cdot}10^{-6}$ - $6.3{\cdot}10^{-5}$	0.34 - 0.61	$1{\cdot}10^{-6}$- $1{\cdot}10^{-7}$
clay		$< 2.5{\cdot}10^{-3}$	$\leq 2.0{\cdot}10^{-6}$	0.34 - 0.57	$1{\cdot}10^{-6}$- $1{\cdot}10^{-7}$
peat [d]		$3.5{\cdot}10^{-3}$ - 30	- (organic)	0.80 - 0.95	-

sources: [a] (Fitts, 2013) [b] (ISO, 2003) [c] (Yu et al., 2015) [d] (Boelter, 1968) [e] (Domenico & Mifflin, 1965)

C | Realistic aquifer compositions

In this appendix, the schematisations of the three realistic aquifer compositions are depicted. A schematisation includes the dimensions and soil type of each layer.

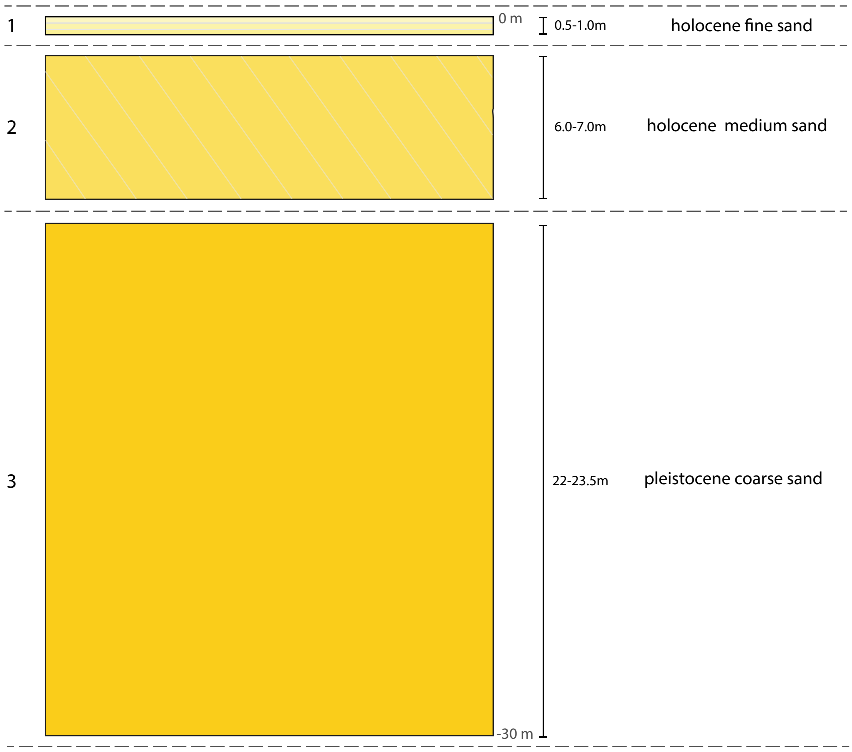

Figure C.1: The schematisation of the pure sand stratified aquifer composition

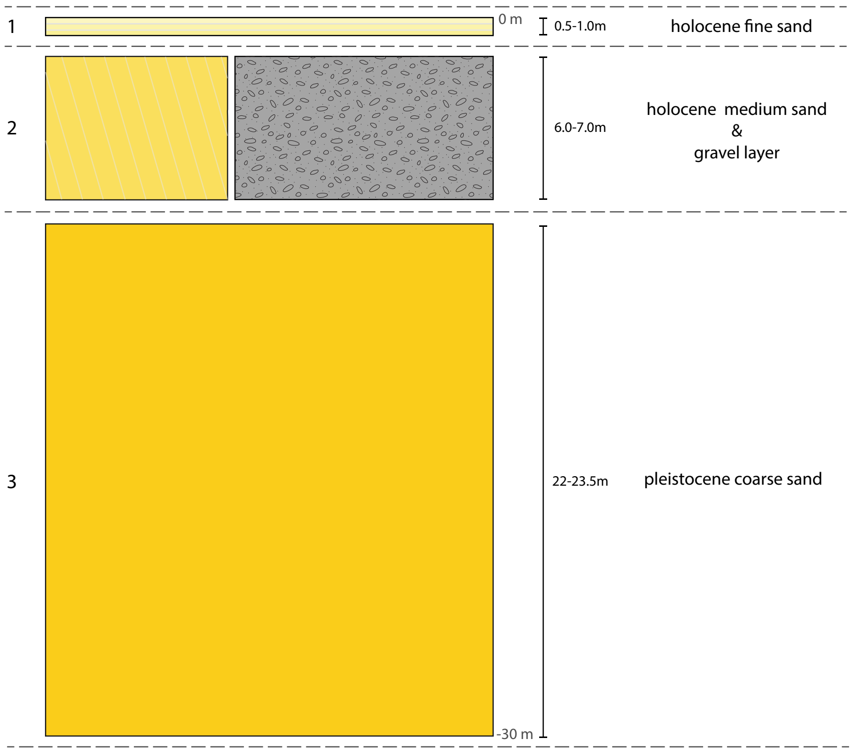

Figure C.2: The schematisation of the Maasvalley aquifer composition

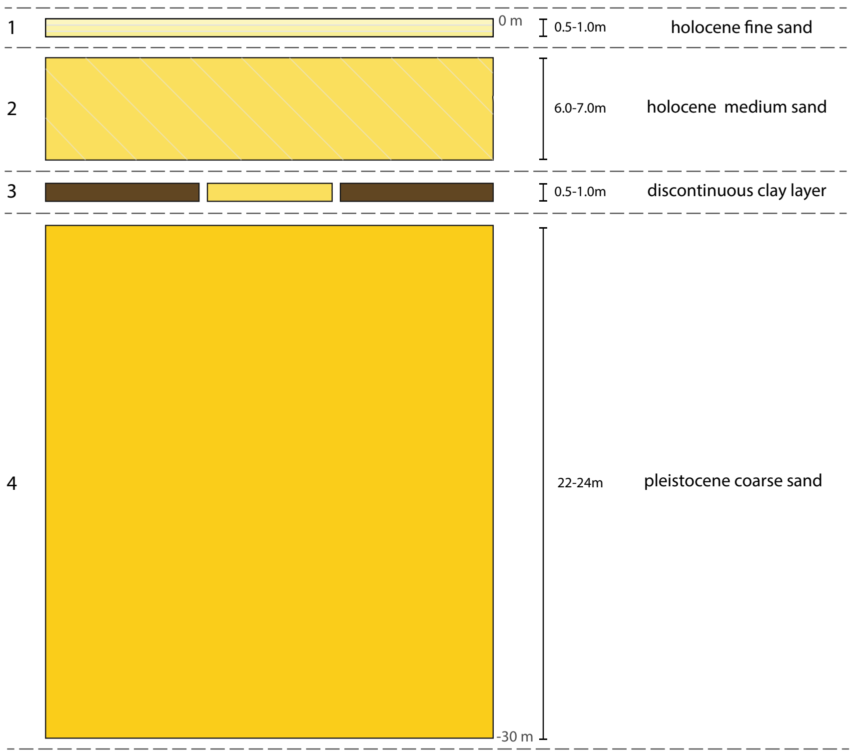

Figure C.3: The schematisation of the Dutch downstream river area aquifer composition

D | Additional pure sand simulations

In order to justify the statement that alternative method is more accurate when the anisotropic vertical permeability is taken into account, additional simulations are performed. In this case, the 4 times smaller vertical permeability of sand layer(s) are taken into account, when the bulk permeability was calculated. As a result, the K_v-value and K_r-value change. The bulk permeability K_r was determined by the alternative method, whereas the bulk permeability K_h was determined according to the currently applied Dupuit assumption. The calculated K_h, K_v and K_r-value are depicted in Table D.1. It is obvious that the K_h value will not change, since only the vertical permeability was changed to add anisotropy. The simulations of homogeneous aquifer with a bulk permeability are again compared with the simulation of the stratified and anisotropic aquifer. The result of an aquifer with anisotropic medium sand layer is given in Figure D.1. The results of all anisotropic compositions are depicted in Table D.2. In addition, Table D.2 clearly shows that both methods underestimate the piping resistance of the heterogeneous and anisotropic aquifer. However, the alternative method is in some cases up to 27% more accurate and still conservative.

Table D.1: The bulk permeability of the anisotropic aquifer compositions

	direction dependence p.l. $[\frac{K_x}{K_y}]$			bulk permeabilities [m/day]		
pure sand compositions	layer 1	layer 2	layer 3	K_h	K_v	K_v
anisotropic fine sand layer	4	1	1	26.1	13.8	19.0
anisotropic medium sand layer	1	4	1	26.1	8.9	15.3
anisotropic holocene sand layers	4	4	1	26.1	5.3	11.8

Table D.2: Influence of anisotropic compositions on the critical head in term of percentage [%]

	direction dependence p.l. $[\frac{K_x}{K_y}]$			difference [%]	
pure sand compositions	layer 1	layer 2	layer 3	K_r [m/day]	K_h [m/day]
anisotropic fine sand layer	4	1	1	18.7	30.2
anisotropic medium sand layer	1	4	1	12.0	27.8
anisotropic holocene sand layers	4	4	1	17.4	44.3

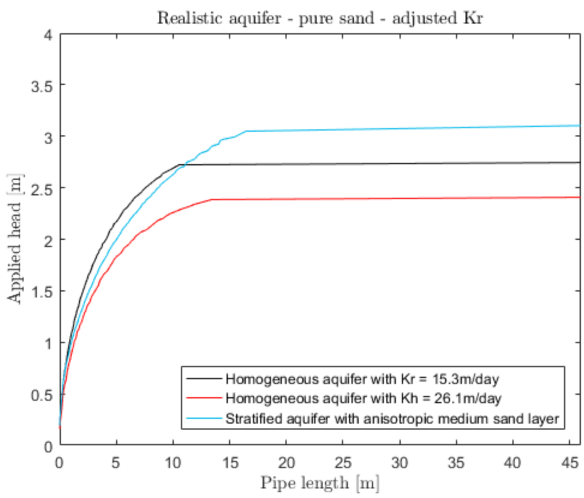

Figure D.1: The critical head of an aquifer with a anisotropic medium sand layer

E | 0,3d-rule

As stated in Section 2.2.3.5, the hydraulic gradient used in the rules of Sellmeijer may be reduce by a factor of 0.3 times the thickness of the cover layer. In that way, the resistance caused by fluidized sand grain in the uplift channel can be taken into account. In order to verify whether the $0.3d_b$ reduction is also observed in the numerical results of a realistic aquifer composition, a pressure of $1.3d_b$ is assigned to the heave boundary. The factor has been increased to $1.3d_b$ so that not only the resistance of fluidized sand uplift channel is taken into account, but also the pressure of water present in the uplift channel is taken into account.

$$1.3\rho_l g d_b = 1.3 \cdot 10^4 [Pa]$$

Where:

ρ_l density of water $[1 \cdot 10^3 \frac{kg}{m}]$
g gravitation acceleration $[10 \frac{m}{s^2}]$
d_b thickness of the blanket $[1m]$

The result of the simulations with and without resistance in the uplift channel are given in Figure E.1. It can clearly be observed that the erosion of the pipe initiates at a higher applied head. In addition, it can be observed that the critical head is increased. However, the critical head is increased with 0.09d instead of 0.3d. This implies that the resistance in the uplift channel mainly thwarts the initiation of pipe erosion and to a lesser extent influences the development of the pipe. This result can mean two things. D-Geo flow takes the resistance in the uplift incorrectly into account or the 0.3d-rule is not appropriate for stratified aquifers. Anyway, more research on the 0.3d-rule and thereby the influence of the blanket is highly recommended.

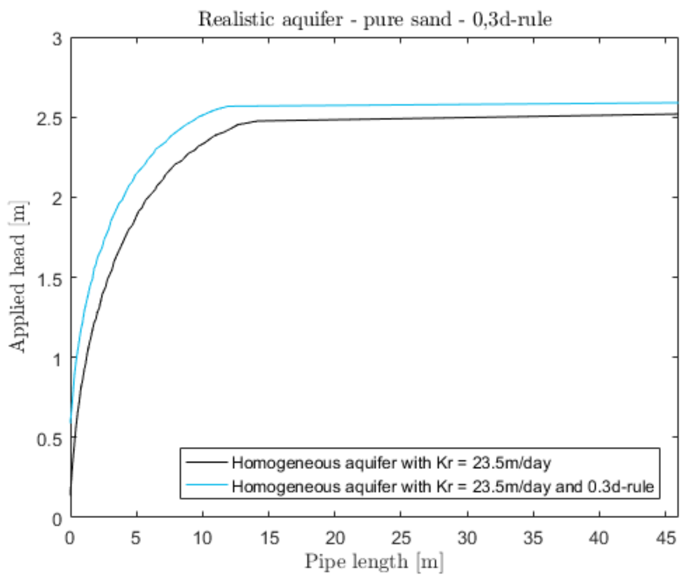

Figure E.1: The result of heterogeneous aquifer with an without the 0.3d resistance of the cover layer

F | K-ΔH$_c$ relation in Sellmeijer's formula

This appendix outlines the relation between the bulk permeability K and the critical head ΔH_c used in Sellmeijer's design rules. Based on this analysis, it can be concluded for which ranges of K the ΔH_c significantly change. In order to plot the relation between K and ΔH_c in Sellmeijer design rules, the value of K is stepwise increased from 0.01 m/day till 200 m/day, while the rest of the parameters remain fixed. The K-ΔH_c relation is plotted for three different seepage lengths (L); 20 m, 45 m and 90 m. The K-ΔH_c relation is given in Figure F.1 and Figure F.2. Table F.1 shows the values of all other parameters:

Table F.1: Parameters used in Sellmeijer design rule for the determination of the K-ΔH_c relation

symbol	value	unit	symbol	value	unit
γ'_p	$1.65{\cdot}10^1$	kN/m^3	d_{70}	$1.50{\cdot}10^{-4}$	m
γ_w	$1.00{\cdot}10^1$	kN/m^3	d_{70m}	$2.08{\cdot}10^{-4}$	m
g	9.81	m/s^2	D	30.0	m
η	0.25	-	ρ_w	$1.00{\cdot}10^3$	kg/m^3
θ	37.00	°	μ_w	$1.30{\cdot}10^{-3}$	kg/(m·s)
K	5.000	m/day			

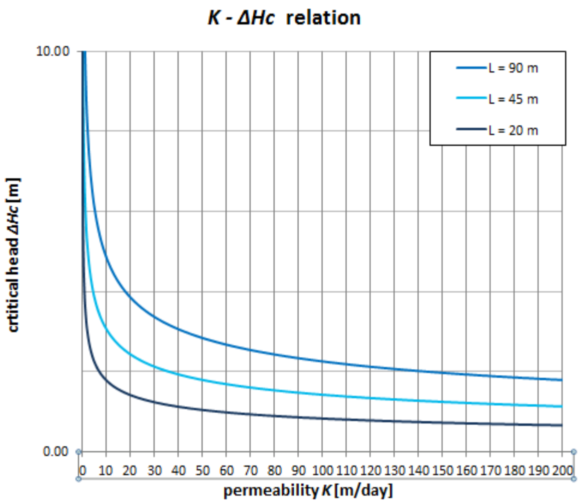

Figure F.1: The K-ΔH_c relation in Sellmeijer design rules

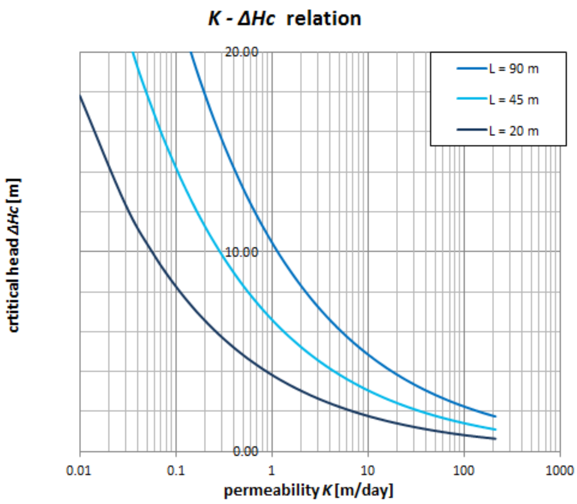

Figure F.2: The K-ΔH_c relation in Sellmeijer design rules with a logarithmic K-axis

Both figures show that the ΔH_c greatly increase if the K - value is smaller than 10 m/day. In addition, Figure F.2 shows that a decrease in permeability (K) by a factor of 10 is almost counterbalanced by an increase in critical head (H_c) by a factor of 2 and visa versa. It should be noted that K-ΔH_c relation is **not** linear, but it is a third order relation. Furthermore, it seems that the ΔH_c will nearly change for large values of K (> 200 m/day) and it will reach a equilibrium so that small deviations in higher K - values are not of significance. On the other hand, small deviations in the lower K-values (< 0.1 m/day) result in large changes in ΔH_c (> 10 m). However, piping is not likely for large value of H_c (> 10 m). As a consequence, small values of K are not of importance. Based on these result, it can be concluded that the K-values range between 0.1 m/day and 200 m/day are mainly important in Sellmeijer's formula. In addition, deviations in the lower K-values will highly influence the critical head ΔH_c.

G | Glossary

Aquifer	A permeable geologic formation or stratum in the saturated zone, so it allows water to pass through.
Aquitard	A poorly permeable or impermeable geologic formation or stratum that lies adjacent to an aquifer, so it allows a very small amount of water to pass through
Bulk permeability	The mean Darcy permeability or the permeability that represent the entire aquifer
Critical gradient	The maximum hydraulic gradient for which a pipe erosion process can reach an equilibrium, otherwise, the erosion process becomes progressive and the pipe develops until an open connection is created between the inner and outer side of the levee.
Failure	The inability to prevent unintentional inundation for flood susceptible areas, this commonly implies the initiation or development of a breach.
Heave	The situation in which the pressure induce by the vertical groundwater flow exceeds vertical grain pressure in a sand layer; also fluidization or the formation of quicksand.
Internal erosion	A process whereby soil particles erodes within or beneath an embankment due to seepage flow
Levee	Raised, predominantly earthen structure with primary objective to provide protection against flood events along coasts, rivers and artificial waterways that are not reshaped by natural action such as current or wave and wind action.
Piping	The progression of shallow pipes in the sandy subsoil underneath the body of a levee due to seepage flow.
Seepage flow	Ground water flow through the aquifer or foundation commonly induced by external high water levels.
Seepage length	The length of the possible path of the seepage flow, from it entry point to the exit point.
Uplift	Bursting of the inner cover layer or blanket due to excessive water pressure in the aquifer underneath the cover layer. General: failure due to the exceedance of the vertical equilibrium underneath the cover layer due to the influence of water overpressure.