The Pipingontspanner

Feasibility of a piping measure based on relief wells

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Challenge the future

The Pipingontspanner

Feasibility of a piping measure based on relief wells

by

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to obtain the degree of Master of Science
at the Delft University of Technology,
to be defended publicly on Wednesday September 26, 2018 at 15:00 PM.

Student number:	4382730	
Project duration:	January, 2018 –September, 2018	
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Preface

This thesis is a part of my master in Hydraulic Engineering at the Faculty of Civil Engineering. The development and assessment of a new piping mitigation measure presented me with a chance to work creatively towards a new potential solution against piping failure under the guidance of Movares and Delft University of Technology.

I would like to thank Movares for this exciting topic and their partnership in developing this measure to its full potential. Special thanks go to Richard Rijkers and Onno Langhorst, as my daily supervisors, for supporting me throughout the thesis with their expertise. The environment and support at Movares allowed me to approach this topic thoroughly.

Furthermore, I would like to thank Mark Voorendt for his involvement and help in shaping the research project, as my daily university supervisor. Also, I would like to extend my gratitude to Prof. Cristina Jommi and Timo Schweckendiek for their expert knowledge and valuable input. At last, I thank committee chairman Prof. Matthijs Kok for his targeted insight and feedback.

Thom Olsthoorn
Utrecht, September 2018

Summary

The Dutch flood defences are subjected to the new Dutch safety assessment program (WBI 2017) that incorporates the latest insights and safety regulations. As a consequence of the updated piping assessment model, studies are expected to identify more piping prone areas in the Netherlands, which makes it worthwhile to investigate alternative piping mitigation solutions that could be less expensive. A new potential solution is the Pipingontspanner, which consists of relief wells in combination with a water catchment area. The Pipingontspanner relieves water pressure from the aquifer and uses the water as counter pressure against the Uplift mechanism, thereby reducing the likelihood of piping.

The study investigates the technical and economic feasibility of the Pipingontspanner as a piping mitigation measure. A problem analysis was used to identify the components that form challenges in realising the concept and pinpoint the research to the functioning, effectiveness and applicability of the Pipingontspanner concept.

A hydraulic model and a design approach demonstrate the functioning of the concept. The hydraulic model describes the flow underneath the dyke, through the well and towards the basin above it. A critical point for the flow calculation appeared to be the time-dependent interaction between the river and the basin water level, which leaves only numerical calculation methods to describe the problem. The numerical program Modflow was chosen to predict the Pipingontspanner groundwater flow by simulating a flood wave scenario for a green dyke with piping problems and implementing relief wells and a basin. A parametric design approach was followed to create a Pipingontspanner model that could obtain the optimal configuration for a measure that can only be calculated with a groundwater flow model. For the verification of design configurations, relevant failure mechanisms have been included in this model. For the selection of the optimal configuration, a cost-benefit analysis has been used as an evaluation criterion. The Pipingontspanner model creates, calculates, verifies and evaluates the different design configurations.

The effectiveness of the Pipingontspanner was illustrated with a sensitivity and a cost analysis. For a variety of subsoil and hydraulic conditions, the sensitivity analysis showed that the safety factor for Uplift increases substantially for higher permeability of the aquifers and slowly growing hydraulic loads. The influence of the cover layer permeability and storage coefficient is negligible on the performance of the Pipingontspanner. On the other hand, the costs analysis showed that well maintenance, well monitoring and basin dyke construction costs are the main cost drivers of the design. The total cost grows exponentially if the basin width behind the dyke is limited as the number of wells increases for smaller basins.

The applicability of the Pipingontspanner was demonstrated through a case study of a green dyke in Tiel with piping problems. In addition, the design and costs of the Pipingontspanner in Tiel were compared against a traditional piping berm measure to illustrate the economic feasibility of the concept. The results showed that the Pipingontspanner is not only able to mitigate the piping problem, but it does so with a smaller footprint and lower total cost than the piping berm. However, the case study also showed that limitations in the current groundwater flow model setup required an adaptation of the case geometry to prevent the drying up of the top layer cells, which would terminate the simulations.

The Pipingontspanner concept has proven to be technically feasible and economically competitive compared to the piping berm under the conditions of:

1. a permeable aquifer with a minimum transmissivity of 25 ($T_{aquifer} \geq 25$ m^2/d);
2. a minimum hinterland space of 10 m behind the dyke ($w_{sb} \geq 10$ m).

List of symbols and abbreviations

List of symbols

Sign	Description	Unit
A	cross-sectional area	m^2
A_{sb}	seepage basin surface area ($A_{sb} = L_w \cdot W_{sb} \cdot n$)	m^2
c'_d	cohesion	kN/m^2
C	hydraulic resistance of layer ($C = d/K_v$)	d
d	1. layer thickness 2. blanket thickness (Chapter 1 and Appendix D)	m
d_{50}	median grain diameter	m
d_{70}	grain diameter in which 70% of the grain mass has a smaller diameter	m
d_{70m}	d_{70} reference value	m
D	aquifer layer thickness	m
F_{rf}	duration rising front flood wave	d
$F_{1,2,3}$	resistance/scale/geometry factor in revised Sellmeijer formula	-
FOS	factor of safety	-
g	gravitational constant (9.81)	m/s^2
h	head	m
$h_{floodwave}$	flood wave height	m
h_p	phreatic head	m
h_r	river water level	m
h_{sb}	seepage basin water level	m
h_{sd}	height seepage dyke	m
H	total head	m
i	gradient	-
$i_{c,h}$	critical gradient for the heave mechanism	-
K_h	horizontal hydraulic conductivity	m/d
K_v	vertical hydraulic conductivity	m/d
l	length	m
L	seepage length	m
L_i	influence length	m
L_w	well spacing	m
m	model factor	-
n	number of wells	-
P_f	failure probability	-
q	specific discharge ($q = Q/A$)	m^2/d
Q	discharge	m^3/d
r	1. horizontal radial distance 2. net discount rate (Chapter 5)	m %
S_c	storage coefficient for confined layer	-
S_{un}	storage coefficient for unconfined layer	-
S_s	specific storage of layer	-
S_y	specific yield of layer	-
t	time	d

Sign	Description	Unit
T	1. transmissivity ($T = K_h \cdot d$) 2. total life cycle (Chapter 5)	m^2/d years
t_i	number of interest periods	-
u	flow velocity	m/d
V	volume	m^3
w	width	m
x	horizontal distance (perpendicular to dyke axis direction)	m
x_{well}	horizontal distance between the dyke body centre and the well	m
y	horizontal distance (in dyke axis direction)	m
z	vertical distance	m
Z	value limit state function	depends
β	reliability index	-
$\Delta\phi_{c,u}$	potential difference limit for Uplift	m
ΔH	head difference	m
γ_{dry}	dry volumetric soil weight	kN/m^3
γ_{sat}	saturated volumetric soil weight	kN/m^3
γ_w	volumetric weight of water	kN/m^3
ϕ	piezometric head or potential	m
ϕ'_d	internal friction angle	°

Subscript	Description
c	refers to critical value
d	refers to design value
exit	refers to the value at the exit point for piping
h	refers to Heave mechanism
max	refers to maximum value
p	refers to Piping mechanism
pb	refers to Piping berm
s	refers to generic direction s
sb	refers to seepage basin
sd	refers to seepage dyke
u	refers to Uplift mechanism
w	refers to wells

LIST OF ABBREVIATIONS

CPT	Cone Penetration Test
FDM	Finite Difference Method
FEM	Finite Element Method
FOS	Factor Of Safety
HWBP	flood defence reinforcement program (Hoogwaterbeschermingsprogramma)
LCC	Life Cycle Costs
NAP	reference level (Normaal Amsterdams Peil)
NPV	Net Present Value
POVpiping	discovery project for the piping mechanism (Project Overstijgende Verkenning piping)
USACE	United States Army Corps of Engineers
VNK	global safety assessment project Netherlands (Veiligheid Nederland in Kaart)
WBI	legal safety assessment tool (Wettelijk Beoordelingsinstrumentarium)

Contents

1

Introduction

1.1. Motivation

The Netherlands is protected by different types of flood defences throughout the country, which safeguard people and cities against storms, river flood waves and climate changes. The fact that 55 per cent of the land is situated within dyke rings proves the importance of the flood protection. Recent insights into failure mechanisms and changing hydraulic loads requires dykes to be assessed for new conditions to guarantee their safety. A large national project called VNK adapted the new insights in the reliability estimation method and re-estimated the safety against flooding (Van Westen, 2005), which showed that multiple dyke sections did not provide the required reliability.

The renewed Water Act of January 2017 stipulated new safety regulations, which incorporates the latest insights for flood defences. The safety assessment program WBI2017, based on safety regulation of the Water Act 2017, provides a more accurate assessment of the failure risks (Rijkswaterstaat, 2018). As a consequence, more locations will be identified as inadequate reliable with the new assessment program (Luijendijk et al., 2017). These dyke sections are then included into the flood defence programme HWBP (Hoogwaterbeschermingsprogramma) to be reinforced.

Piping is one of the failure mechanisms that lead to failure of a flood defence. In Figure 1.1, the results of the 2014 assessment program show the sections where piping reliability is inadequate, and this number is predicted to increase in the current WBI2017 assessment. The improvements and measures to mitigate the piping problem in dykes are coupled with significant investments, which makes it worthwhile to look for possible improvements in knowledge and technique.

The POVpiping (Project Overstijgende Verkenning piping) is a project of the flood defence programme HWBP that stimulates and organises the development of knowledge and technique in piping mitigation measures. At this moment in the Netherlands, drainage techniques are not considered as an established alternative for piping measures in a dyke reinforcement (Luijendijk et al., 2017). Even though, many examples of drainage techniques in dykes have been realised such as the Waterontspanner and the Grindkoffer (Niemeijer et al., 2017).

A new concept that mitigates the piping problem is the Pipingontspanner, which is based on a drainage technique. An advantage of the Pipingontspanner is the reduction in seepage discharge when compared to other drainage measures. The possibilities of this concept are unknown, and therefore this thesis investigates the capability and feasibility of the Pipingontspanner concept as a piping measure.

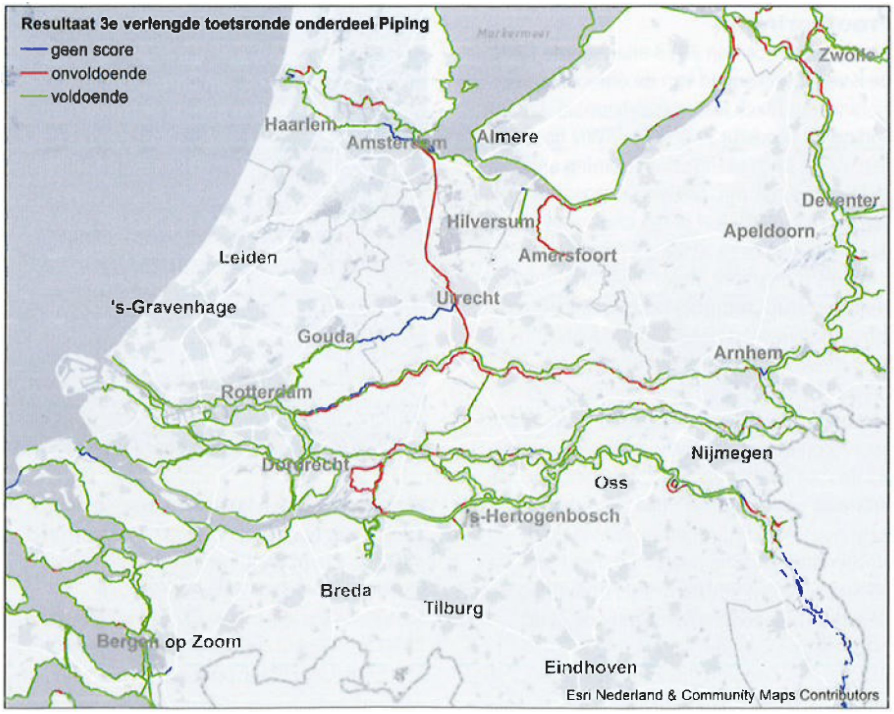
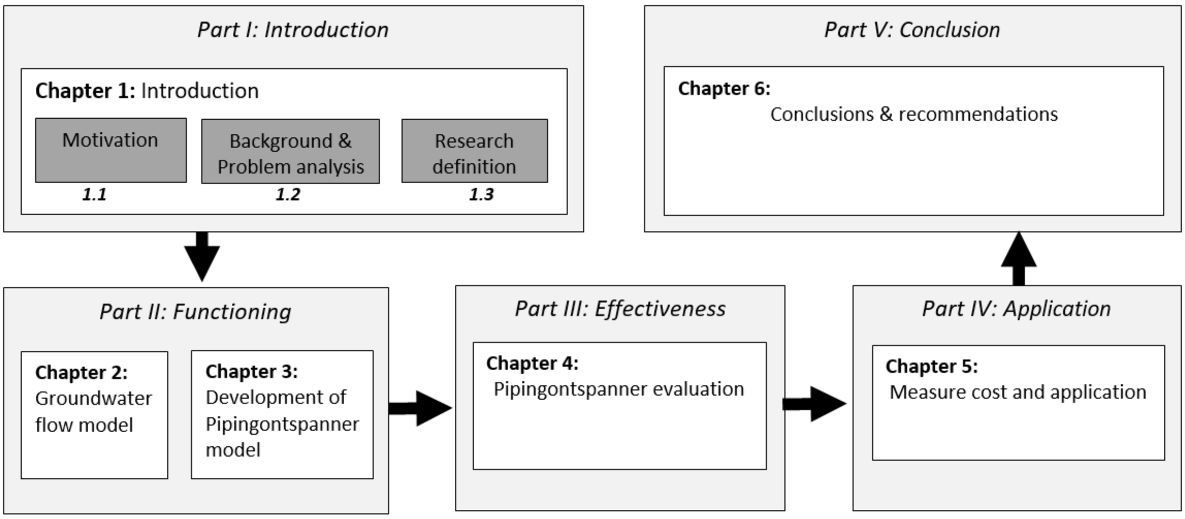


Figure 1.1: Results piping safety assessment program 3+ 2014 (Luijendijk et al., 2017)

The report can be divided into five parts, namely introduction (I), functioning (II), effectiveness (III), application (IV) and conclusion (V) of the new concept. This Chapter is the first part and describes the motivation, problem analysis and background, research definitions and reading guide.



1.2. BACKGROUND AND PROBLEM ANALYSIS

This section presents background information and a problem analysis for the Pipingontspanner concept. In the development of a piping measure, it is valuable to know, which components of the Pipingontspanner form challenges in realising the concept. The problem analysis highlights these aspects and focuses the problem statement and research questions.

In the next two sections, the background of the failure mechanism piping and drainage techniques are described followed by the problem analysis.

1.2.1. PIPING

Piping is a significant failure mechanism of flood defences in the Netherlands. In the past five years, there has been an increase in scientific and practical interests for the mechanism piping to reduce knowledge uncertainties and improve piping measures. It resulted in new insights and developments for the failure mechanism piping, which are implemented in the safety regulation of the Water Act 2017. The number of piping prone locations is expected to increase due to the adapted insights in the Water Act 2017. The Water Act 2017 changed the assessment method of dykes from a probability of exceedance to a dyke segment failure probability using a probabilistic approach. The assessment program WBI2017, accompanying the Water Act 2017, implemented the following elements from the explored insights and developments (POVpiping, 2017b):

- explicit uncertainty of (stochastic) subsoil parameters;
- subsoil scenario's per section.
- the length effect;
- new assessment criteria: growth speed of pipe;
- new design rule Sellmeijer including new (dependent) safety factors.

The dyke sections in the Netherlands need to comply with the new safety regulation of the Water Act 2017 before 2050. The transition between the old and new norms is phased, starting with the assessments of dyke sections and hydraulic structures according to the new assessment criteria (POVpiping, 2017b).

A dyke with insufficient protection against piping can be strengthened with multiple permanent measures consisting of traditional or new solutions. An example of a traditional solution is adding a hinterland berm at the toe of the dyke. This measure prevents uplift of the aquitard but requires sufficient space and large soil transports to realise the berm. Another option is a sheet pile wall or diaphragm wall in the dyke, which extends the flow path. The more recent solutions are for example drainage systems or geotextile screens. All solutions are based on lowering the head difference, lengthening the flow path or preventing the Heave mechanism.

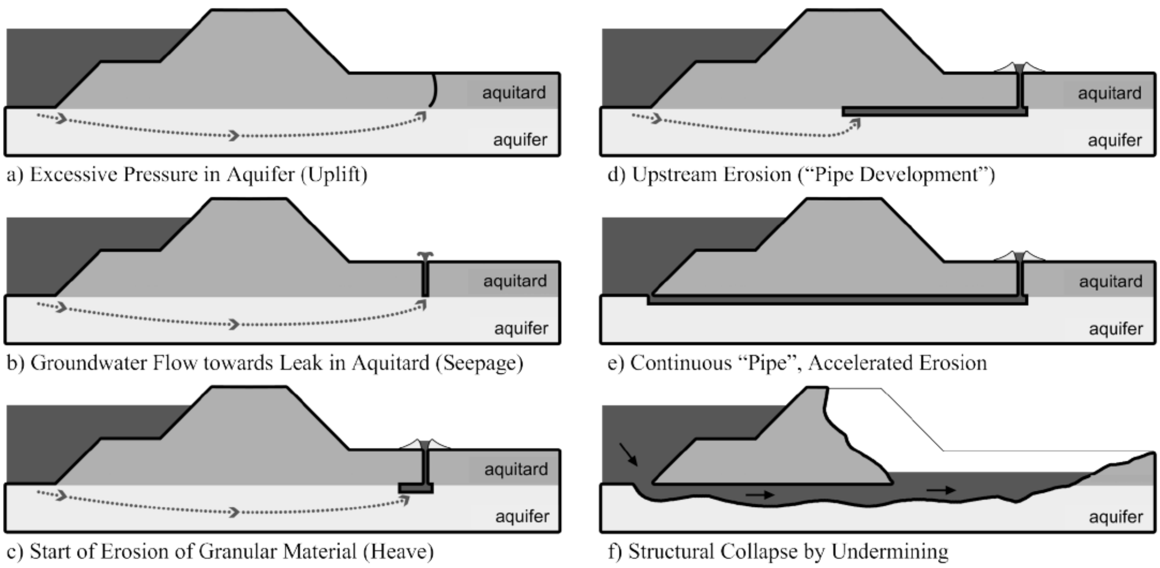


Figure 1.2: Description of the piping process (backward internal erosion) (Schweckendiek, 2014)

MECHANICS

The failure mechanism of backward internal erosion (piping) is a process of sand transport under a dyke as a consequence of under-seepage. This process results in the structural collapse of the dyke body and failure of the water retaining function. The failure mechanism is initiated by the occurrence of sub-mechanisms Uplift, Heave and Piping. In Figure 1.2, the process of piping is depicted and step a, c and e represent the sub-mechanisms Uplift, Heave and Piping, respectively (Schweckendiek, 2014).

Uplift

The upward force of the pore pressure in the aquifer is larger than the downward force of the aquitard layer above causing uplift of the aquitard. Consequently, the aquitard ruptures and a leak occurs in the aquitard (Figure 1.2 a/b). This leak enables transport of sand in certain circumstances.

Heave

The mechanism of Heave evaluates the start of particle transport. The transport occurs if the hydraulic gradient at the exit point exceeds the critical value for transport of sand particles. This step is characterised by a heap of sand (Sand boil) above the aquitard. (Figure 1.2 c).

Piping

The transport of sand particles develops a pipe in the aquifer, which is a continuous accelerating process (Figure 1.2 d). The mechanism piping occurs if a pipe of water flow exist along the entire dyke length and connects the entrance and exit on both sides of the dyke. The mechanism piping destabilises the back side of the dyke and causes a structural collapse by undermining (Figure 1.2 e/f).

The dyke needs to meet three conditions before a piping process, as described above, can occur.

1. A water level difference between the water body and the hinterland, which causes a pressure difference between the both sides of the dyke. This is a time dependent process and this condition needs to be present for the duration of the piping process.
2. The aquifer needs to consist of loosely packed particles (sand), such that it forms a permeable layer.
3. The aquitard covers the aquifer and separates the hinterland water level from the river by densely packed material (such as clay or peat) that forms an impermeable boundary.

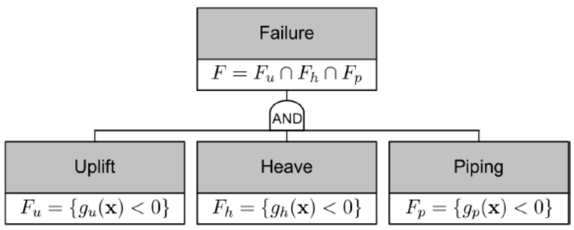


Figure 1.3: Fault tree mechanism piping (Backward internal erosion) (Schweckendiek, 2014)

The sub-mechanisms (Uplift, Heave and Piping) behave like a parallel system, as all mechanisms need to fail before backward internal erosion failure occurs. The AND-gate in a fault tree for the piping mechanism represents the parallel system, depicted in Figure 1.3. The sub-mechanisms also play an essential role in the design of piping measures, as only one of the mechanisms needs to be averted/mitigated to prevent Piping (Schweckendiek, 2014).

ASSESSMENT MODELS

The three mechanisms of piping (Uplift, Heave and Piping) are described by models to determine the occurrence of the mechanism. These models can be used in a deterministic and probabilistic approach, which are described by a limit state function Z. The mechanism occurs if the limit state function Z is lower than zero. The relevant definitions for variables in the limit states of the sub-mechanism are presented in the schematic dyke cross-section in Figure 1.4.

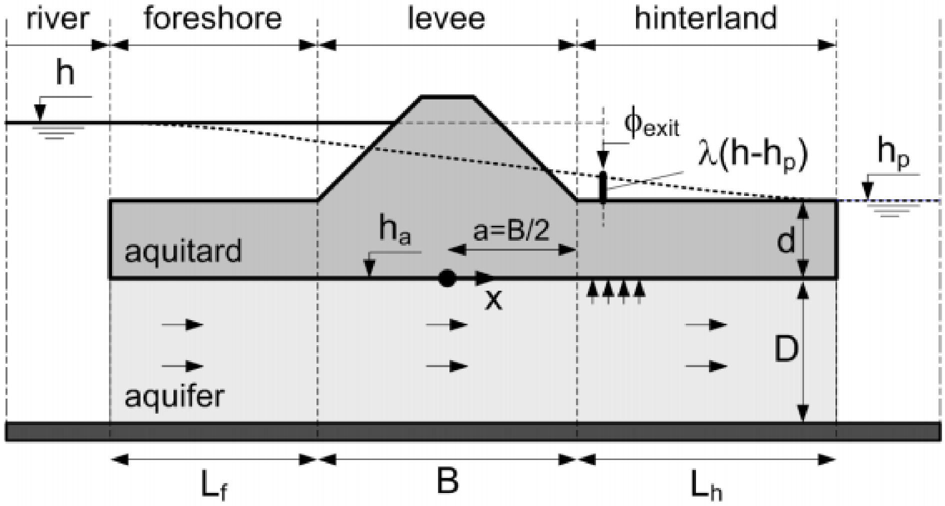


Figure 1.4: Definitions relevant for limit state (Schweckendiek, 2014)

h	river water level [m]	λ	damping (response) factor
h_p	head at land side [m]	ϕ_{exit}	piezometric head at exit point [m]
d	blanket thickness [m]	γ_{sat}	saturated volumetric soil weight [kN/m^3]
D	aquifer thickness [m]	γ_w	volumetric weight of water [kN/m^3]

The water pressure in the aquifer drops between the two boundaries of the river water level (h) and the polder water level (h_p). The hydraulic head pressure (ϕ_{exit}) in the aquifer and the gradient (i) over the dyke can be expressed by the following equations:

$$\phi_{exit} = h_p + \lambda(h - h_p) \tag{1.1}$$

$$i = (\phi_{exit} - h_p)/d = \lambda(h - h_p)/d \tag{1.2}$$

The damping factor (λ) can be estimated in a simplified situation or determined with groundwater flow models, monitoring or expert judgement.

Uplift

The limit state for uplift is based on a vertical force balance of the downward pressure (resistance) caused by the aquitard and the upward water pressure in the aquifer (solicitation). The uplifting of the aquitard occurs if the water pressure in the aquifer exceeds the counter-pressure of the aquitard layer above. The uplift results in cracking of the impermeable layer. The balance of the two forces is expressed in head differences, in which the downward pressure of the aquitard is described by the potential limit. The limit state and Factor of Safety (FOS) for the mechanism Uplift are described by:

$$Z_u = m_u \cdot \Delta\phi_{c,u} - \Delta\phi = m_u \cdot \Delta\phi_{c,u} - (\phi_{exit} - h_p) \qquad FOS_u = m_u \frac{\Delta\phi_{c,u}}{\phi_{exit} - h_p} \tag{1.3}$$

The potential limit ($\Delta\phi_{c,u}$) at the exit point is described by:

$$\Delta\phi_{c,u} = d\frac{\gamma_{sat,blanket} - \gamma_w}{\gamma_w}$$

Heave

The limit state for Heave compares the occurring gradient (i) over the blanket (solicitation) to the critical gradient ($i_{c,h}$) required for sand transport (resistance). The critical gradient depends on the median particle diameter in the sand layer, which can be determined among others methods by a distribution of the occurring diameters. The United States Army Corps Engineers (USACE) described the critical gradient for seepage and internal erosion under different conditions by experimental data. The data shows the gradient at which sand boils were detected (USACE, 1992). The flow will transport the particles to the surface once the critical gradient is exceeded.

$$Z_h = i_{c,h} - i = i_{c,h} - \frac{\phi_{exit} - h_p}{d} \qquad FOS_h = \frac{i_{c,h}}{i} \tag{1.4}$$

Piping

The failure mechanism of piping can be evaluated using multiple methods developed over time. Bligh introduced the first method to assess piping in 1912 using an empirical approach. In 1935, Lane continued on the Bligh formula by researching multiple dams in the United States. He concluded that seepage is three times more efficient in vertical planes than horizontal planes. Sellmeijer introduced a more fundamental approach in 1988 by calculating flow patterns caused by head differences between the two water systems and the resistance of particle erosion in the piping channel. The Sellmeijer model was adapted by the TAW in 1999 and specified for the dutch subsoil and dyke systems. The work of Sellmeijer was improved upon in 2011 by researching physical models.

In the current assessment approach of WBI2017, the method of Bligh was discarded due to safety concerns for the reliability of the method. The WBI2017 uses the improved Sellmeijer method (2011) in the current safety assessments as it proved to be more reliable in evaluating the piping mechanism.

The limit state for piping describes the equilibrium state in the pipe development based on the critical head difference, according to Sellmeijer's improved model. The limit state and Factor of Safety for the mechanism piping are shown below.

$$Z_p = m_p \cdot \Delta H_c - \Delta H = m_p \cdot \Delta H_c - (h - h_p - 0.3d) \qquad FOS_p = m_p \frac{\Delta H_c}{h - h_p - 0.3d} \tag{1.5}$$

The critical Head difference (ΔH_c) is described by:

$$\Delta H_c = F_1 \cdot F_2 \cdot F_3 \cdot L \qquad F_1 = \eta \cdot \frac{\gamma_s}{\gamma_w} \tan\theta \qquad F_2 = \frac{d_{70m}}{\sqrt[3]{\frac{\upsilon K L}{g}}} \left(\frac{d_{70}}{d_{70m}}\right)^{0.4} \qquad F_3 = 0.9 (d/L)^{\frac{0.28}{(d/L)^{2.8}) - 1} + 0.04}$$

where:

L	seepage length [m]	d_{70}	70%-fractile grain size distribution [m]
θ	bedding angle [°]	d_{70m}	reference value for d_{70} [m]
η	drag factor coefficient [-]	g	gravitational constant (9.81) [m/s^2]
υ	kinematic viscosity of water [m^2/s]	K	specific conductivity [m/s]
γ_s	weight solids aquifer [kN/m^3]		

1.2.2. DRAINAGE SYSTEMS IN GENERAL

Drainage systems provide a solution for dykes to prevent the occurrence of piping. Drainage techniques are only one of the possible options to prevent piping and, currently, are not often considered in the Netherlands as an alternative piping measure (Luijendijk et al., 2017).

A drainage system releases pressure in a controlled manner without allowing internal erosion to occur. The drainage system decreases the pressure locally by extracting the water to the surface either passively or actively. The general impact of a drainage system near a dyke at high river water levels is presented in Figure 1.5. There are two different drainage measures currently applied in the Netherlands: drains (horizontal and vertical) and gravel filters, which are depicted in Figure 1.6.

Drainage systems can be applied in both regional and primary flood defences, but the most gain is achieved in the highly varying water levels of primary flood defences. Drainage systems for dykes are often applied in a closed off sand layer (aquifer), which decreases the pressure head in this layer. The decrease reduces the probability of the Uplift, Heave and Piping failure sub-mechanisms. The application of a drainage system in the sand layer (aquifer) is considered to be promising with a limited permeability, but not too low (Niemeijer et al., 2017).

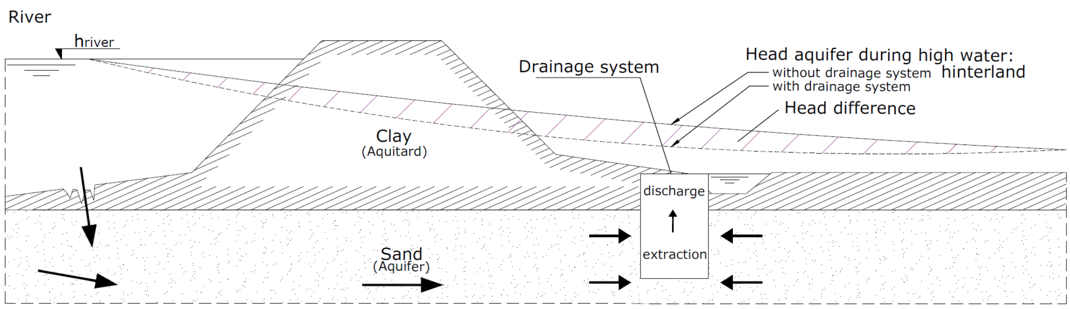


Figure 1.5: Schematic representation of a drainage systems near a dyke during high water levels (Niemeijer et al., 2017)

A drainage system is also used to influence the failure mechanism macro instability by installing drainage systems at a shallow depth near the toe of the dyke. This drainage system affects the phreatic waterline and lowers the water content inside the dyke body.

A drainage system in an aquifer does not only affect the pressure head, but also the water system of the polder behind the dyke. The drainage system releases pressure and discharges the seepage to the surface water. This process reduces the seepage discharge from the river to the polder, which has positive and negative influences for the polder behind the dyke. An advantage is the reduced seepage discharge that limits water nuisance in the polder during high water levels. A disadvantage of the reduced seepage is the effect on the groundwater quality and potential settlements.

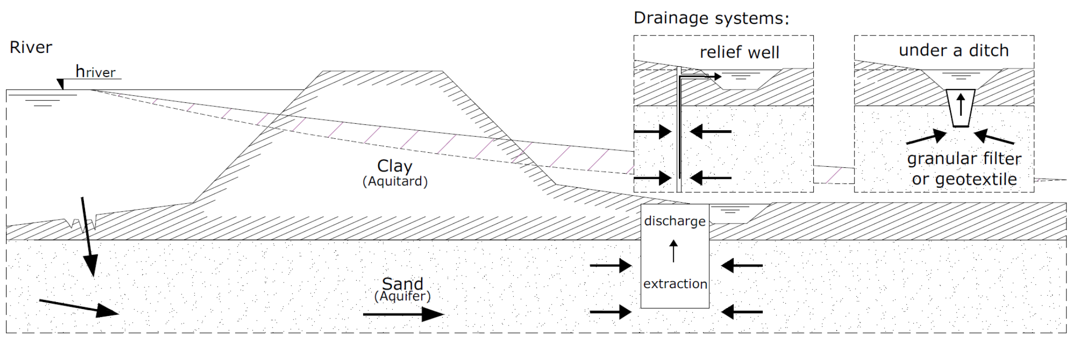


Figure 1.6: Schematic representation of a relief well and gravel filter (Niemeijer et al., 2017)

1.2.3. PIPINGONTSPANNER CONCEPT

The Pipingontspanner has been developed as a drainage technique and based on a different measure named the Waterontspanner. The function of the Waterontspanner is to increase the macro stability of a dyke by relieving the upward pressure in the sand layer beneath dyke. The pressure is released with vertical drains near the toe of the dyke. The macro instability is caused by high water pressures in the sand layer that will lift up the impermeable cover layer.

The Pipingontspanner uses the same principle but serves the purpose of preventing the piping mechanism. The Pipingontspanner concept is selected by Rijkswaterstaat to be included in the discovery program POVpiping (Projectoverstijgende Verkenning) of the flood defence programme HWBP (Hoogwaterbeschermingsprogramma) (POVpiping, 2017a).

CONCEPT DESCRIPTION

The Pipingontspanner focuses on preventing excess pressure over a larger distance than the Waterontspanner. The mechanics of the Pipingontspanner are displayed in Figure 1.7. The concept is based on a vertical drainage system (also called relief well), which is a passive system that releases the excess water pressure. The water from the well is temporarily stored in a basin above the cover layer. The water stored in the basin provides counter pressure and prevents excess pressure during high river water levels.

The layout of the Pipingontspanner is different from other relief well or drainage solutions, as the water is now temporarily stored instead of directly discharged by other means. The configuration relies on the discharge from the passive drainage system and the total water storage volume. As the water level in the storage basin rises, the head difference becomes smaller, and as a result, the seepage discharge decreases. The seepage discharge is not only dependent on the varying outside river level but also on the seepage basin water level as this level is no longer a static boundary condition.

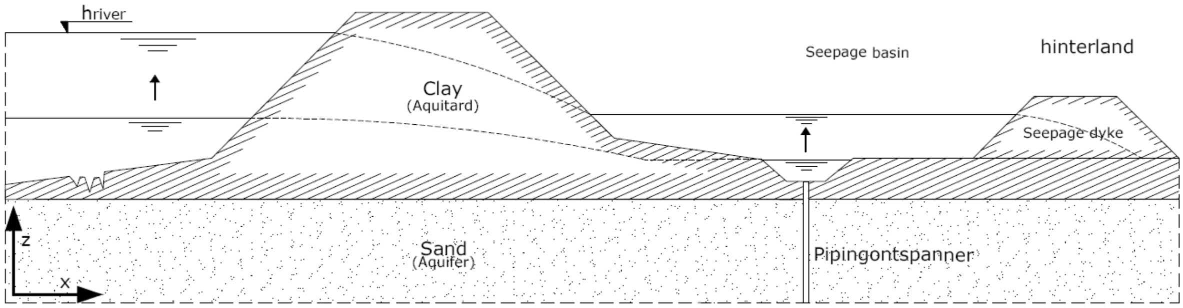


Figure 1.7: Principle cross-section of the Pipingontspanner

The influence of time dependency in the hydraulic model is larger when compared to other relief well solutions. The main design variables are related to the transient flow process, and more information is needed to describe the mechanics up to the level that is required for the design phase.

PRINCIPLE EXAMPLE

The Pipingontspanner resembles an emergency measure that is used to stop the piping mechanism. An ongoing piping process can be recognised by the appearance of sand heaps behind the dyke due to the active sand transport in the Heave process. Once sand heaps are detected behind the dyke, it is possible to mitigate the piping process using an emergency measure. A conventional emergency measure in the Netherlands is called ‘Opkisten’ and uses sandbags to create a watertight dam around the leak in the aquitard. The measure of Opkisten is described in Figure 1.8 and shows 1. the sand heap detection and building of the dam structure and 2. the result of the structure. An example of this measure is shown in Figure 1.9.

The water level inside the formed basin rises and provides counter-pressure on the aquifer. Additionally, the rise of the water level in the ring reduces the head difference to a small extent (Technische Adviescommissie Waterkeringen, 1993). This basic principle of the emergency measure Opkisten directly relates to the introduced concept Pipingontspanner, but is used on a larger scale and as a preventive measure.

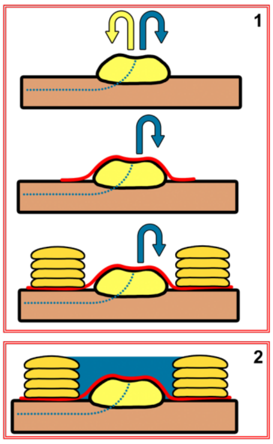


Figure 1.8: Measure Opkisten (Deltares, 2016)

Figure 1.9: Application of emergency method Opkisten (Technische Adviescommissie Waterkeringen, 1993)

APPLICATION

The implementation of the Pipingontspanner requires an area behind the dyke, which is suited as a potential retention area. The land behind the dykes is only in use when necessary in high river water level periods and otherwise available for other purposes. The open land can be combined with nature developments, farming grounds or retention areas in cooperation with ecologist and landscape architecture. The multifunctional use of the ground behind the dyke requires an agreement between the landowners, municipality and dyke administrators.

The Pipingontspanner uses a varying river water level to counteract the piping mechanism only when necessary. Therefore, the application is mainly focused on primary dykes, as secondary dykes often have a constant water level. A constant water level works counteractive with the Pipingontspanner principle because the inflow would be continuous.

Figure 1.10: Nature behind the dyke

CHALLENGES IN THE CONCEPT DEVELOPMENT

The Pipingontspanner imposes challenges on the development and realisation of the concept. This section describes the challenges that need to be resolved before the Pipingontspanner concept is considered as a piping mitigation option.

Societal acceptance

The Pipingontspanner allows water behind a dyke to prevent a dyke failure, which might be a strange concept for the landowners, municipality and dyke administrators. These stakeholders need to be convinced that the water behind the dyke is not a calamity (flooding), but an inconvenience. The concept needs to be approved and accepted as a valid measure against piping by all the stakeholders before the measure is considered as a piping mitigation option. The concept is proven if sufficient project results have shown acceptable cost, risks and benefits. Moreover, this proof is achieved by a process of case studies, scale tests and pilot projects that validate the performance of the design.

Implementation

The implementation of the Pipingontspanner is limited by the space requirements behind the dyke, which reduces the application possibilities in the Netherlands. The required width behind the dyke is determined by the river load and the soil parameters, which can be found by a sensitivity analysis. The implementation of the design should consider farming purposes and nature developments, as those functions fit in the environment created by the Pipingontspanner design.

Reliability, monitoring and maintenance

The reliability of the concept needs to be calculated according to the safety requirements of the Water Act 2017. This means that every part of the Pipingontspanner design should be specified as the total failure probability of the dyke. The maintenance and monitoring aspects of the Pipingontspanner allow the proper functioning of the design, which is vital in achieving the required reliability. The functioning of the wells is difficult to monitor once these are installed, and the consequences of a well blockage or malfunction are high. Currently, new techniques are tested to monitor the flow in a drain by for example temperature changes (POVpiping, 2017b).

Post-flood event scenarios

The Pipingontspanner allows water behind the dyke, which should be cleared in periods with regular river water levels (depending on the function of the land behind the dyke). The basin water level would theoretically lower by a negative hydraulic gradient after a high river water level. However, the basin water will never drop below the river level, and flow by gradient is ineffective at small water level differences. The basin cannot be drained effectively without the use of other measures.

The Pipingontspanner basin can temporarily store water during a river flood wave, after which discharge pumps can remove the water down to the desired level. The time required for the basin to become relatively dry depends on the discharge capacity and use of the land. A high basin water level could also form a safety risk for flooding by the failure of the seepage dyke. There are more options to drain the Pipingontspanner basin, but increase the complexity and cost of the project. The requirements of the design are leading in creating a suitable piping measure.

1.2.4. KNOWLEDGE GAPS

In the paragraphs below, an inventory of knowledge gaps is presented first, followed by a series of potential research questions. The aspects relate to geotechnical and hydraulic processes and are presented per cross-section and longitudinal section, the X-Z and Y-Z Plane respectively.

Cross-section

The X-Z plane of the Pipingontspanner is schematically presented in Figure 1.11 and differs from typical dyke cross-sections in the Netherlands. A new smaller seepage dyke is constructed to create a basin and the water level behind the dyke is raised, during high river water levels, which also increases the phreatic waterline inside the dyke body.

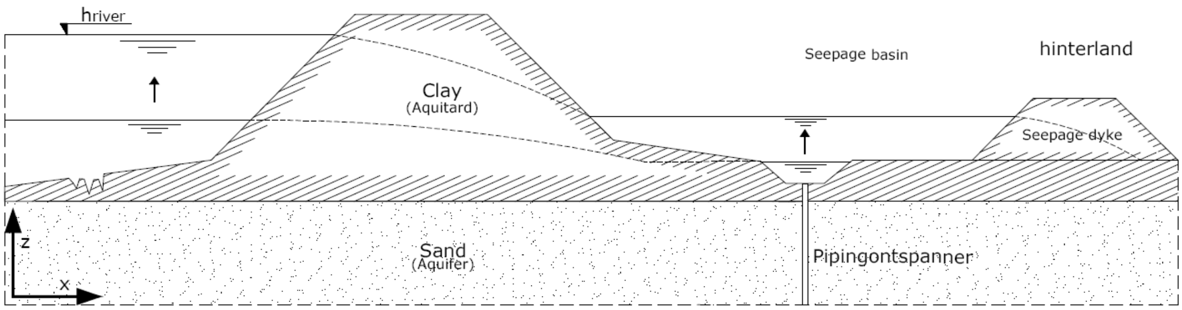


Figure 1.11: Schematic representation Pipingontspanner cross-section X-Z plane

From a geotechnical point of view:

- The raised phreatic waterline inside the dyke negatively influences the inward macro stability of the dyke as the dyke is now more saturated and gains mass. On the other hand, the water layer in the basin should provide some extra counter pressure, but the extent of which is unknown.

From a hydraulic point of view:

- The main design parameter of the Pipingontspanner is the relief well discharge and the basin storage area. The relief well discharge depends on the head difference between the inner and outer water level, which are both variable. The inner water level is defined by the hydraulic conductivity of the aquifer and the hydraulic resistance of the river bed. Presently, there are no models that describe this transient flow with enough accuracy to create a reliable design.
- A high water level in the basin could cause a secondary piping mechanism around the newly built seepage dyke. The water retaining function of the primary dyke fails if the seepage dyke collapses by a second piping mechanism. The conditions in which a secondary piping mechanisms could occur are unknown.
- The design of the Pipingontspanner is dependent on the development of the relief well discharge over time. The hydraulic conductivity variability of the subsoil has a large influence on this discharge and thus causes complex situation when accounting for the variability in soil properties. Currently, there are inaccuracies in modelling the hydraulic conductivity and resistance of flow from the river to the aquifer due to a large variation in heterogeneity.

Knowledge questions X-Z Plane:

1. How does the phreatic surface develop inside the dyke core and what effects does this have on the dyke stability?
2. Which conditions cause a secondary piping mechanism on the new seepage dyke?
3. How can the relief well discharge be modelled to the required accuracy level?
4. How does the time-dependent hydraulic load influence the seepage basin's water level?
5. What is the influence of the variability in the subsoil conductivity properties on the design of the Pipingontspanner?

Longitudinal section

In the Y-Z plane, depicted in Figure 1.12, the design focuses on the distances between the relief wells to prevent piping along the required dyke length.

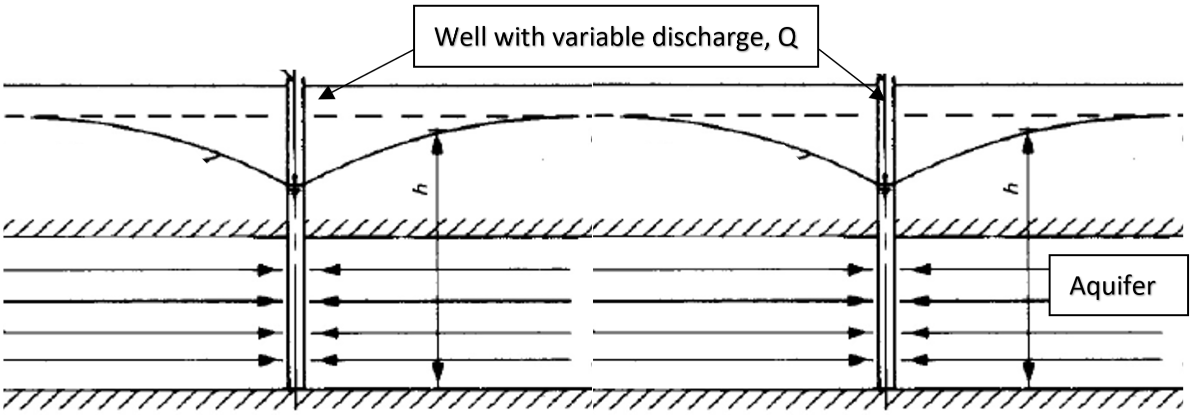


Figure 1.12: Schematic representation of a relief well along the dyke length Y-Z plane (Adapted (USACE, 1986))

From a hydraulic point of view:

- The hydraulic conductivity of the subsoil varies in the cross direction but this holds even more for the longitudinal direction. This variance cause differences in the relief well discharge. The effect and extent of discharge variability on the relief wells in the Pipingontspanner is unknown.
- The water in the basin needs to be controlled in both the cross and longitudinal directions. The new seepage dyke acts as a boundary in the cross direction, but the longitudinal direction also requires boundaries. The influences of a design with dedicated basin boundaries around a single or system of relief well(s) in combination with variable discharges is unknown.

The knowledge gaps and potential (failure) risks as explained above are described by the following knowledge questions:

1. How does the heterogeneity of the subsoil influence the seepage through and underneath the dyke along the dyke length?
2. What is the influence of a variable discharge between multiple wells in the design of the Pipingontspanner?
3. How can the design account for variability in the relief well discharge?
4. What are the influences of a design with dedicated basin boundaries around a single well or multiple relief wells?

1.3. RESEARCH DEFINITION

1.3.1. PROBLEM DEFINITION

The Pipingontspanner is only a potential flood risk reducing measure because the workings of the concept have never been tested or applied. Therefore, the extent of its effectiveness, benefits and potential risks are still unknown. Currently, the conventional piping measures are assumed to provide a more economical solution due to the lower risks and more experience in the design and construction aspects. The feasibility (applicability and cost-effectiveness) has not yet been sufficiently proven to be accepted as an official piping mitigation measure.

1.3.2. MAIN RESEARCH QUESTION

The problem analysis of the Pipingontspanner and the problem statement result in the main research question and corresponding sub-questions. The main research question is:

'Under what circumstances is the Pipingontspanner concept technically and economically feasible?'

The main purpose of this research is to find the circumstances that allow the Pipingontspanner concept to be economically feasible and thus be competitive with other piping mitigation measures. This goal is achieved by showing the workings of the concept, effectiveness and applicability of the Pipingontspanner measure. A comparison with an existing (proven) measure provides information on the aspect of economic feasibility. The results indicate the added value of the Pipingontspanner, quantified in an example, and provide an advancement in the development of piping mitigation measures.

1.3.3. SCOPE

A collection of important aspects to the Pipingontspanner is presented and coupled to the knowledge questions in Section 1.2.3. From this set of knowledge questions, only a selection is considered in this thesis, which is incorporated into the sub-research-questions. The remaining questions are relevant but, in the context of forming a basic design of the Pipingontspanner, not an objective of this study. The aspects that are considered in this study relate to the hydraulic model, design characteristics and acquired safety, which will lead to the design of the Pipingontspanner.

The piping process is defined according to the assessment criteria of the Netherlands, as detailed in Section 1.2.1. The guidelines and requirements of the Water Act 2017 are taken into consideration in the design of the Pipingontspanner. In the first instance, long and short-term effects due to construction phases are neglected such as pore pressure changes and settlement of the subsoil.

1.3.4. SUB RESEARCH QUESTIONS

To answer the main research question, the following sub-questions are specified:

1. How can the seepage discharge be modelled with the required accuracy level?
2. Which requirements can be used to assess the reliability of Pipingontspanner concept?
3. Which design approach defines the Pipingontspanner concept?
4. Under which conditions is it technically possible to implement the Pipingontspanner as a piping mitigation measure?
5. What is the impact of the Pipingontspanner on the macro stability of the dyke?
6. Under which conditions is the Pipingontspanner economically favourable compared to other piping mitigation measures?

An outline of the report is given in Section 1.4.

1.3.5. RESEARCH APPROACH

To meet the purpose of the thesis, the following research approach is proposed. In the overview presented in Table 1.1, the research questions are coupled to the different activities in the study.

Table 1.1: Research approach

Method		Research question	Content
A.	Literature review	Q1 & Q2	Research in groundwater flow and drainage techniques.
B.	Hydraulic model	Q1	Design hydraulic model to calculate the groundwater flow.
C.	Parametrisation	Q3	Design specification and coupling of the hydraulic model to design parameters.
D.	Sensitivity analyses	Q4	Analysis of the design variables sensitivity in the parametric model.
E.	Assessment macro stability	Q5	Investigation of the dykes macro stability under the conditions caused by the Pipingontspanner.
F.	Case study	Q6	An example framework to verify and compare the design with other mitigation methods.

A. The start of the thesis consists of a literature study, which provides the necessary background information to describe the Pipingontspanner concept from a technical and functional point of view. The literature study includes the following topics: the failure mechanism piping, developments in drainage systems as a measure against piping, background of the relief well systems, groundwater flow in dykes and the Pipingontspanner concept.

B. The background information is used to create a hydraulic model that evaluates the flow through the dyke and the relief well under certain boundary conditions. The transient flow process is approached with both an analytic and numerical solution that calculates the groundwater flow through the dyke. The appropriate calculation method follows from the literature study with either a finite difference method (Modflow) or finite element method (FE-flow and Plaxis).

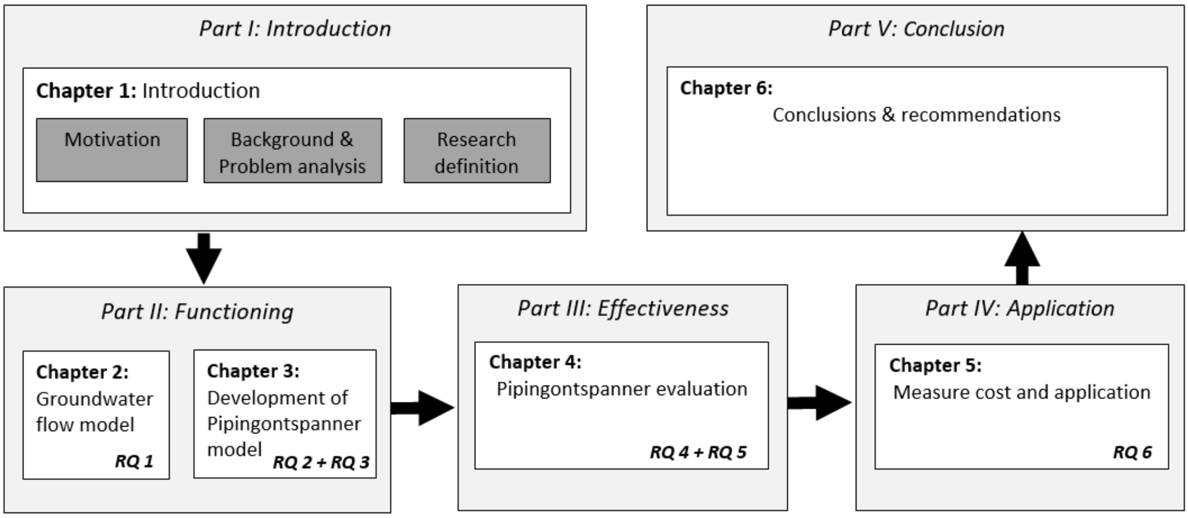
C. The Pipingontspanner design is defined according to general boundary conditions and requirements. This basic design is converted to a parametric model including all available variables and coupled to the hydraulic model via Python. This (Pipingontspanner) model can then be used to effectively calculate design changes in correspondence with the hydraulic effects.

D & E. After which, the focus is turned to the sensitivity analysis. The Pipingontspanner model is used to show the sensitivity of the design variables and find the optimal operating range.

F. The basic design and Pipingontspanner model are verified in an example framework. This example is located in Tiel at the river Waal on the north bank between the streets Waalstraat and dijkstraat, where 0.6 kilometres of green dyke is sensitive for piping. The final design is compared against other measures in terms of costs to study the cost-effectiveness of the Pipingontspanner measure.

1.4. READING GUIDE

The report can be divided into four parts, namely introduction (I), functioning (II), effectiveness (III), application (IV) and conclusion (V) of the Pipingontspanner concept. The structure of the report roughly follows the order of the sub-questions, presented in Section 1.3.4. This chapter concludes the first part and in the second part, formed by Chapter 2 and 3, the Pipingontspanner concept is transformed into a functioning design.



The second part starts with building a hydraulic model to describe the groundwater flow caused by the Pipingontspanner in Chapter 2, which is followed by the conditions for a design of the Pipingontspanner in Chapter 3. The design is coupled with the hydraulic model to form a parametric (Pipingontspanner) model.

In the third part, Chapter 4, the Pipingontspanner concept is evaluated by a sensitivity analysis of the Pipingontspanner model and a macro stability analysis.

The fourth part, Chapter 5, discusses the general cost of the measure and a case study to compare the Pipingontspanner with the cost of other piping measures. Moreover, the Pipingontspanner is applied in the realistic environment of the case study.

The last (fifth) part presents an overview of the answers to each of the sub-questions, as well as, the main research question in Chapter 6, followed by a list of recommendations for future research that finalises the study.

2

GROUNDWATER FLOW MODEL

In this Chapter, the first research sub-question is answered by analysing and developing the calculation models for the groundwater flow caused by the Pipingontspanner (POS) in the dyke.

2.1. GROUNDWATER FLOW

The river water enters the polder as groundwater flow through the porous subsoil layers and is called seepage. The seepage process is a precondition of the piping mechanism and its behaviour is of importance in predicting the occurrence of piping. In figure 2.1, a schematic representation of seepage in and around dykes is shown.

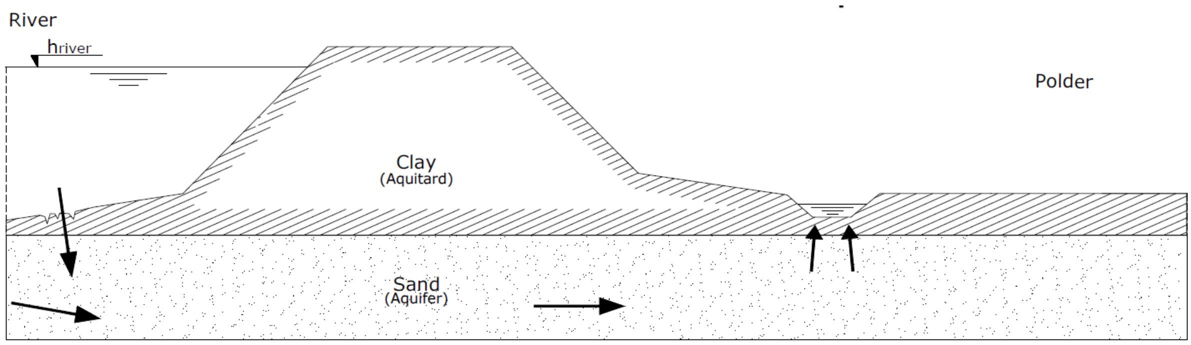


Figure 2.1: Schematisation of seepage flow under a dyke (Technische Adviescommissie Waterkeringen, 1993)

The flow under a dyke can be characterised as flow underneath an impermeable layer. The impermeable dyke body and cover layer (e.g. clay layer) are called aquitard and the water-bearing permeable layer (e.g. sand layer) underneath the dyke is called aquifer. The water level difference between the river and polder causes a gravity-driven flow from higher to lower pressures, which is also referred to as flow under a seepage gradient. The river water infiltrates both the aquifer and aquitard, but the infiltration rate of the aquitard is much lower.

The aquifer is often characterised by its transmissivity ($T = K_h \cdot d$), which is the horizontal hydraulic conductivity (K_h) multiplied by the layer thickness (d). The hydraulic behaviour of the aquitard is described by the hydraulic resistance ($C = d/K_v$), in which the layer thickness (d) is divided by the vertical hydraulic conductivity of the layer (K_v).

2.1.1. PRINCIPLE OF FLOW

The empirical relation of Darcy's law and the conservation of mass principle provide the required tools to analyse the groundwater flow. Darcy showed with his experiments that the discharge is proportional to the head difference between two manometers and (was also) inversely proportional to the distance between those manometers. The discharge can be related to the flow through a certain cross-sectional area which combined with the linear relation of Darcy's law for one dimensional (1D) flow gives:

$$Q_s = -K_s \frac{dh}{ds} A \qquad q_s = Q_s / A = -K_s \frac{dh}{ds} \tag{2.1}$$

Q_s discharge in s direction [m^3/d]
q_s specific discharge in s direction [m^2/d]
K_s hydraulic conductivity [m/d]
dh piezometric head difference [m]
ds distance along s direction [m]
A cross-sectional area [m^2]

The hydraulic conductivity (K_s), also known as conductivity, is a measure of the water's ability to flow through a medium, where high values signify larger volume transfers. When the head difference (dh) and distance (ds) remain constant, the flow velocity is proportional to the hydraulic conductivity, which is a soil property. The hydraulic conductivity can be determined by a constant head or falling head experiment and an overview of characteristic values is shown in Figure 2.2.

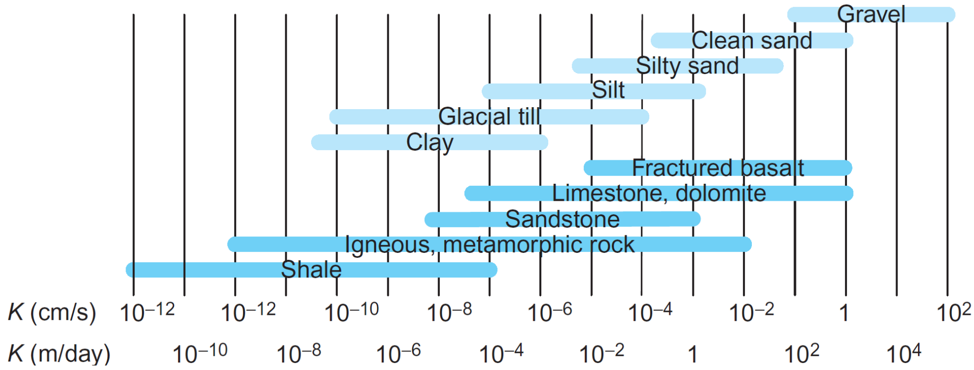


Figure 2.2: Typical values of hydraulic conductivity based on experiments (Fitts, 2002)

The conductivity of a soil layer differs for the x, y and z-direction, due to spatial inhomogeneity and a varying layer structure. To describe the flow patterns through the layers, Darcy's law is used in three dimensions with a Cartesian coordinate system (x,y,z), given by:

$$q_x = -K_x \cdot \frac{dh}{dx} \qquad q_y = -K_y \cdot \frac{dh}{dy} \qquad q_z = -K_z \cdot \frac{dh}{dz} \tag{2.2}$$

The direction and intensity of the specific discharge are given by the vector sum of the x, y and z components: $|\mathbf{q}| = \sqrt{q_x^2 + q_y^2 + q_z^2}$

The application of Darcy's law in groundwater flow is limited to a physical principle. Darcy's law becomes invalid if this principle no longer holds, which is when:

- the flow velocity in the subsoil is too high; For example in a subsoil with large pores;
- the subsoil medium is too irregular;
- the small-scale variations are being researched;
- the flow is turbulent.

2.1.2. GROUNDWATER STORAGE

The groundwater storage is influenced by the water table, the porosity of the soil and compression processes. The amount of water released or stored per unit area for a change in hydraulic head is expressed by the storage coefficient (S) and reflects how fast a groundwater system responds to river level changes (Fetter, 2001). For an unconfined aquitard, the storage coefficient (S_{uc}) equals to the specific yield (S_y), which is the volume of water per volume of aquitard that can be yielded by drainage.

$$S_{uc} = S_y \quad [-] \qquad S_c = S_s \cdot d \quad [-] \qquad S_s = \frac{K_v}{c_v} \quad [1/m] \tag{2.3}$$

For a confined aquifer, the storage coefficient (S_c) is given by the specific storage (S_s) and determined with pumping tests or estimated from the consolidation coefficient (c_v), layer thickness (d) and the vertical soil permeability (K_v) in equation 2.3 (Niemeijer et al., 2017).

2.1.3. RELIEF WELLS

An important part of the Pipingontspanner concept is a relief well, which reduces the pressure in the aquifer. The relief well, also referred to as artesian well, brings water to the surface by utilising the pressure in the aquifer instead of a mechanical pump that is used in wells. The water is transferred from the aquifer to the seepage basin by the relief, which is depicted in 2.3. A full description of the Pipingontspanner concept is provided in section 1.2.3.

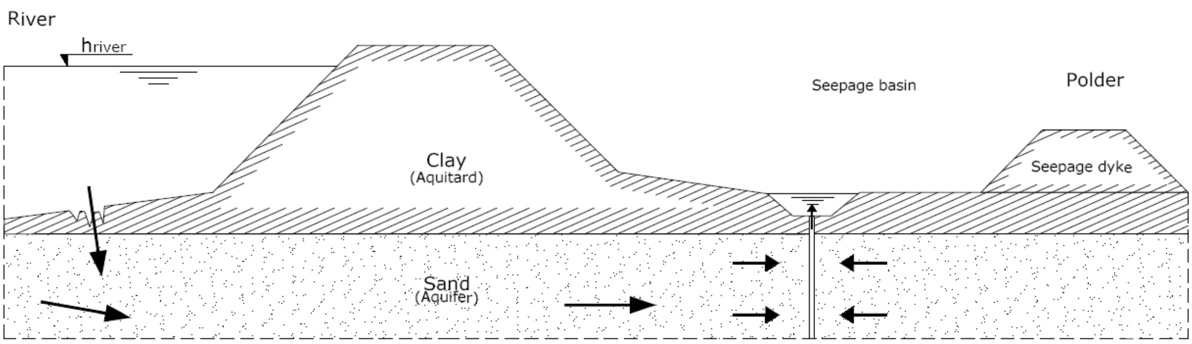


Figure 2.3: Schematisation of the Pipingontspanner with a relief well

The behaviour of a well can be described as steady state or transient state. In a steady state, the groundwater flow does not change in time and is considered in an equilibrium with the (constant) boundary conditions. Hence, the discharge of the well does not change in time as long as the boundary conditions remain constant. The period before reaching the steady state is called transient state, in which the groundwater flow varies over time to adapt to the equilibrium state.

The situation in the Pipingontspanner, depicted in Figure 2.3, is in a continuously transient state due to the rapidly varying river boundary. The Pipingontspanner is examined for a river flood wave, in which the time between variations is too short for a steady state to occur. A typical flood wave in the River Rhine in the Netherlands generally lasts several days to weeks (Rijkswaterstaat, 2017).

Analytic solutions for pumping wells (steady or transient conditions) are well documented and can describe the lowering of the head level (dh) for a certain pump discharge (Q_w) based on Darcy's law and the mass balance equation. The general equation for steady-state radial flow to a well is a differential equation, presented below, and leads to an expression for the head (h) after integration with a certain boundary condition ($h(r_0) = h_0$). This approach is based on laminar or viscous flow in order to apply the Darcy equation.

$$\frac{dh}{dr} = \frac{Q_w}{2\pi r_w \cdot T_{aquifer}} \qquad h(r_1) = \frac{Q_w}{2\pi \cdot T_{aquifer}} \ln r_1 + h_0 \tag{2.4}$$

Q_w discharge by well [m^3/s]
$T_{aquifer}$ transmissivity aquifer [m^2/d]
h piezometric head [m]
r_w well radius [m]
r_1 radial distance from the well [m]

In the case that the well is located near a straight boundary (e.g. river), an imaginary mirror well is added to comply with the acting flow boundary conditions of a river. The head can then be described for a coordinate (x,y) and the radial distance r_1 and r_2 from the coordinate to the well and image well, respectively. The well equation near a river is represented by the formula:

$$h(r_1, r_2) = \frac{Q_w}{2\pi \cdot T_{aquifer}} \cdot ln \frac{r_1}{r_2} + h_0 \tag{2.5}$$

Equation (2.5) can also describe a relief well, however, the well discharge (Q_w) is not specified as a boundary condition. Therefore, the equation with two unknowns (Q_w and h) cannot be solved. The United States Army Corps Engineers (USACE) developed a semi-empirical solution for the head caused by a relief well based on Equation (2.5).

The USACE approach uses an empirical 'Well factor' to estimate the discharge in the well for a steady state after an iterative process. The Well factor is developed by Bennett and Barron in 1954 and verified by an electric analogy test (USACE, 1992). The solution is used to design a piping mitigation measure for dams and dykes with relief wells.

The USACE approach could be applied to the Pipingontspanner concept to calculate the head and discharge of the relief well, but differences in the schematisation prevent a correct representation of the Pipingontspanner situation. These differences are:

- The design load for the Pipingontspanner is a river flood wave and does not provide a steady-state situation for the relief well.
- The extracted water from the well is stored in a basin and is not directly transported away from the dyke system. The volume of stored water influences the well's discharge by decreasing the head difference.

2.2. CALCULATION METHOD

The approach to calculate the hydraulic behaviour of the Pipingontspanner is described in this section. The requirements of the calculation model are defined according to the problem characteristics, which is followed by a comparison of different calculation methods.

2.2.1. MODEL REQUIREMENTS

A definition of the model requirements allows for the selection of a suitable calculation model. The properties of a model are defined by the:

- models purpose
- answered research questions
- transient or steady state
- 1D, 2D plain or 3D map view

The purpose of the model is to predict the pressure underneath the aquitard and the volume of water stored inside the seepage basin. These two results allow the determination of the pressure equilibrium between the aquifer and aquitard. This information helps to assess the Uplift failure mechanism and answers the question: How effective is the Pipingontspanner in reducing the piping probability. Additionally, the model answers questions related to the development of pressure in the aquifer, flow velocity in the well and volume of stored groundwater in the seepage basin.

The problem is characterised by a transient state and limits the solution strategies more than a steady state. The influences of the well are examined for the X-Z and Y-Z direction, making the choice for a 3D model trivial, especially when introducing multiple wells in the model.

The calculation model provides indicative results of the hydraulic behaviour to further develop the Pipingontspanner concept. The results can be validated and calibrated by measurements in the case the model shows promising results.

2.2.2. CALCULATION MODELS

The groundwater flow calculation model can be described using different techniques, which are graphical solutions, analytic solutions or numerical flow modelling. The preferred calculation model is based on the model requirements.

GRAPHICAL AND ANALYTIC SOLUTIONS

A graphical solution provides a simple and flexible method to estimate the steady-state head distribution and discharge. A frequently used method is Flownets, which uses a simple method of streamlines and equipotential lines based on 2D flow plane. The Pipingontspanner situation is too complex to accurately calculate the flow with a graphical solution.

Analytic solutions allow for fast computation of results and insight into the influence of parameters. However, for most of the transient problems, the flow processes are too complex for analytic solutions leaving only steady-state solutions. The semi-empirical method of USACE does describe relief wells, but differences in schematisation render the solutions incompatible, as was concluded in Section 2.1.3.

NUMERICAL FLOW MODELLING

The problem can be approached by numerical methods such as the finite difference method (FDM) and finite element method (FEM). In both methods, a series of algebraic equation based on Darcy's law and the mass balance equation are solved for a hydraulic head at discrete points. These points are based on a network of nodes within the modelled area. The modelled area is subdivided in blocks with a certain block size and a calculation node is placed inside the block. The program's scheme determines the location of the nodes inside each block, which can ,for example, be along the edges or centre. Each block is characterised by homogeneous physical properties for that specific block. This principle allows for a heterogeneous domain with different properties in each block (K_x, K_y, K_z, S_s, and S_y).

The difference between FDM and FEM is the shape of the discrete elements, which are more flexible in FEM. FDM uses only boxes for the discretisation, whereas FEM allows for triangular and trapezoidal shaped elements. The strengths and limitations of FDM and FEM are similar and mostly differ from an analytic approach (Fitts, 2002). The FDM and FEM methods use discretised domains, in contrast to the non-discretised domains of analytic models. The last allows for a higher accuracy and more flexibility in the formation of the domain but is limited by the complexity of the problem.

The numerical methods FEM and FDM meet the model requirements and allow transient, 2D and 3D calculations. Feflow is an example of a FEM-program that models groundwater. FDM is commonly used in groundwater flow modelling with the program Modflow. The Pipingontspanner problem is easily discretised in blocks, which makes Modflow a good choice. In addition, the last can be controlled using Python which allows for integration with a parametric design.

2.3. GROUNDWATER FLOW MODEL PIPINGONTSPANNER

In this section, the approach and results of the groundwater flow model for the Pipingontspanner are presented. The results are obtained by cycling through the modelling process until the right representation for the Pipingontspanner is found.

Modelling process cycle:
1. Define problem → 2. Schematisation → 3. Computation → 4. Evaluate results → Repeat (2-4)

2.3.1. PROBLEM DEFINITION

The hydraulic problem can be defined as a generic implementation of the Pipingontspanner according to the concept description in Section 1.2.3. The generic design is based on the expectations for the final design.

The Pipingontspanner is designed for a primary dyke section bordering a river that is sensitive for the Piping mechanism. This situation is depicted in Figure 2.4 and can be characterised by a permeable aquifer, a shallow cover layer and a low hydraulic resistance from the river to the aquifer.

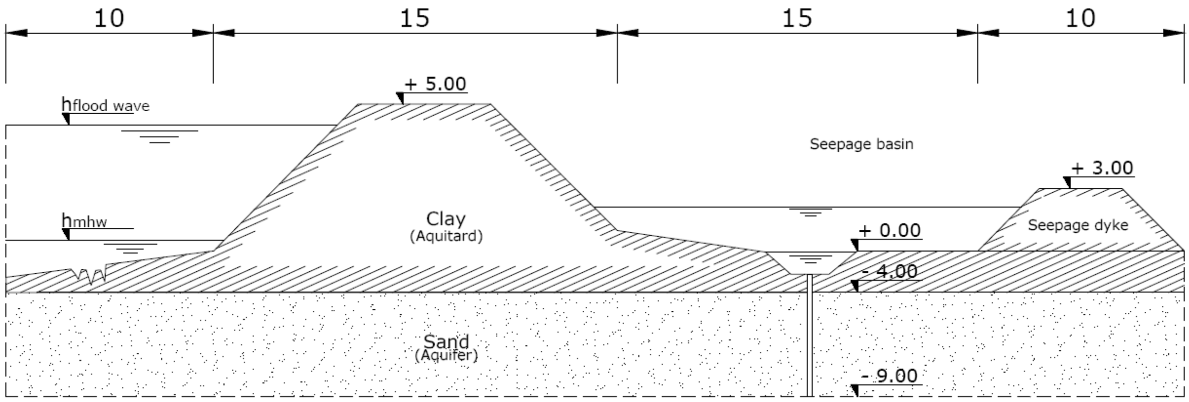


Figure 2.4: Principle sketch for flow mechanism

The characteristic values for this hydraulic problem are taken from a similar example project (Waterontspanner) at the river Lek in Langerak. The soil properties of the aquifer and aquitard (thickness, hydraulic conductivity and porosity) are adapted to fit the situation depicted above.

The boundary conditions are characterised by a river flood wave and a constant polder level behind the dyke. A river flood wave on top of the river mean high water level is found to be the governing load for the piping mechanism in primary dykes. The fast increase of the river water level on top of the mean high water level can last 10 to 30 days depending on the river system.

The hydraulic resistance from the river to the aquifer is determined by the streambed conductance and describes the delay in the pressure transfer to the aquifer. A low streambed conductance prevents rapid pressure increases below the cover layer. In piping situations, the streambed is locally affected by erosion of the bed material reducing the protective thickness in for example river bends and local interventions. The hydraulic model uses a high conductance as the governing situation.

This hydraulic problem focuses on the effects of the river flood wave on the water pressure and flow behind the dyke. The influences of recharge, evapotranspiration and other sources or sinks are not considered in this problem. The parameters for the initial design of the Pipingontspanner (seepage dyke height, seepage basin width and volume) are estimated based on the soil properties of the example project and initial model results.

2.3.2. SCHEMATISATION

The generic problem can be translated into a three dimensional (3D) schematisation for the Modflow calculation software. The schematisation can be subdivided into: model domain, geometry, layer properties, boundary conditions and well implementation.

MODEL DOMAIN

The area of interest is centred around the dyke and the seepage dyke. The domain is restricted to an initial dyke length (y-axis) of 50 m and a cross-sectional distance (x-axis) of 50 m. The cross-section includes 5 m before the dyke and 5 m behind the seepage dyke.

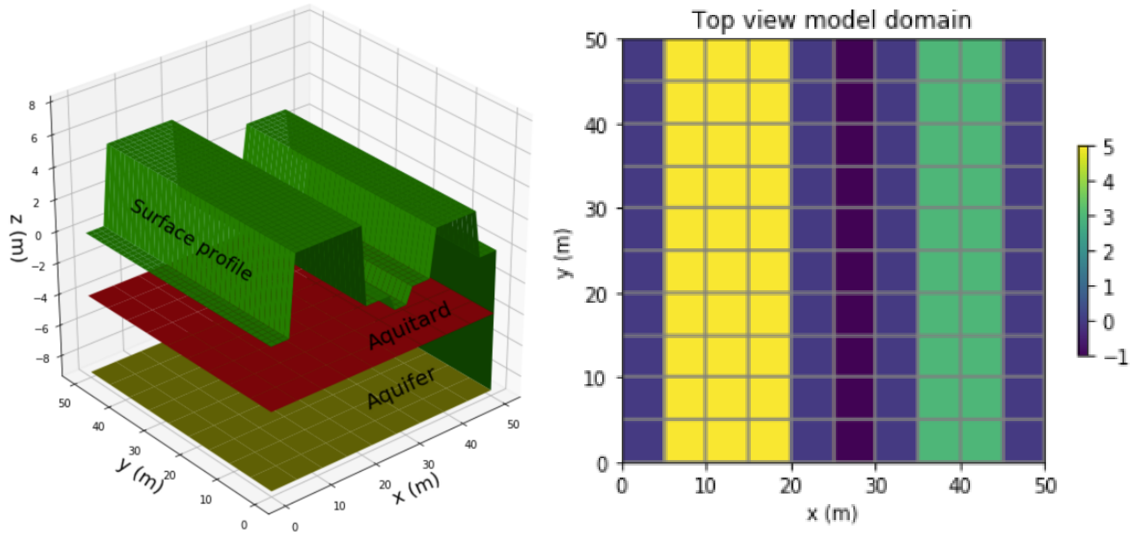


Figure 2.5: Model domain 3D and top view

GEOMETRY

The geometry is uniform in the dyke length axis of which a cross-section is depicted in Figure 2.6. The system is divided into three separate layers: top layer, aquitard and aquifer. The reference level (REF) is taken at z = 0 m. The top layer (layer 1) reaches from REF +5 to -1 m and describes the surface level. The aquitard (layer 2) starts at REF -1 m and has a thickness of 3 m. The aquifer (layer 3) starts at REF -4 m and has a thickness of 5 m.

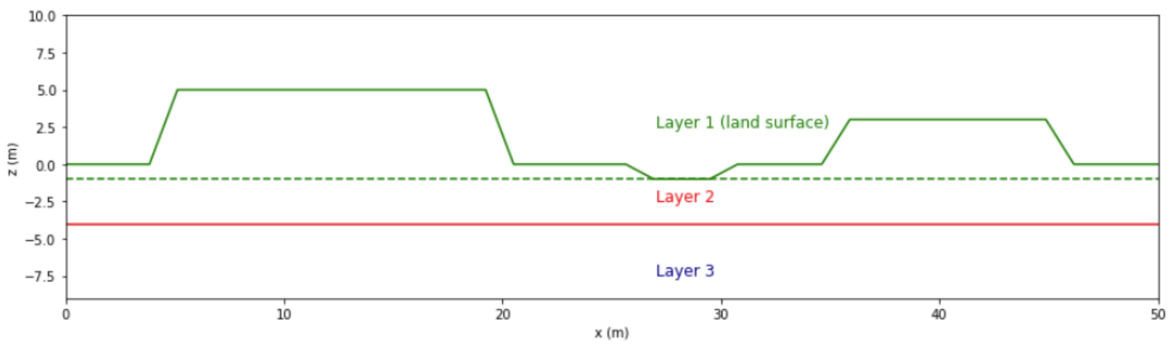


Figure 2.6: Overview geometry schematisation

The top layer comprises of the dyke, the bottom of the seepage basin and seepage dyke from left to right in Figure 2.6. The height of the dyke is taken at REF +5 m, while the seepage dyke height is taken at REF +3 m. The width of the dyke and seepage dyke is taken as 15 and 10 m, respectively. The dimensions of the seepage basin are 15 x 10 x 3 m (w_{sb} x l_{sb} x h_{sd}).

LAYER PROPERTIES

The layer properties are assigned to each layer and consist of the specific yield (S_y), the specific storage (S_s) and the hydraulic conductivity ($K_{h/v}$). The values for these properties in the generic model are taken from a example project at the river Lek near Langerrak (the Waterontspanner). The transmissivity (T) and hydraulic resistance (C) of each layer can be determined from the layer properties, which are presented in Table 2.1.

Parametrisation layer model										
Property	Layer type	Top	Bottom	Thickness	Kh	T	Kv	C	Ss	Sy
		[m NAP]	[m NAP]	[m]	[m/d]	[m^2/d]	[m/d]	[d]	[1/m]	[-]
Layer 1 (part a)	dyke body	10	-1	11	0.005	0.055	0.005	2200	-	0.05
Layer 1 (part b)	empty space	10	-1	11	86400	950400	86400	0.00013	-	0.99
Layer 2	cover layer	-1	-4	3	0.0005	0.0015	0.0005	6000	0.001	0.05
Layer 3	aquifer	-4	-9	5	10	50	3	1.67	0.0001	0.2

Table 2.1: Layer properties

Layer 1 (top layer) consists of the two dyke structures and the storage basin between them. The top layer is separated into two parts by assigning different properties to the correspondent cells. The dyke bodies are impermeable, and the storage basin (empty model space) is very permeable. In Figure 2.7, the vertical and horizontal hydraulic conductivity of the top layer is depicted.

The storage basin is modelled as soil with a porosity of 99% and a high permeability that causes no restriction to flow. This approach deviates from reality, but works as the basin is only considered as a storage for groundwater. A permeability of 1 m/s for the vertical and horizontal hydraulic conductivity of empty space was found to be sufficiently high to pose no resistance to the flow in and towards the storage basin.

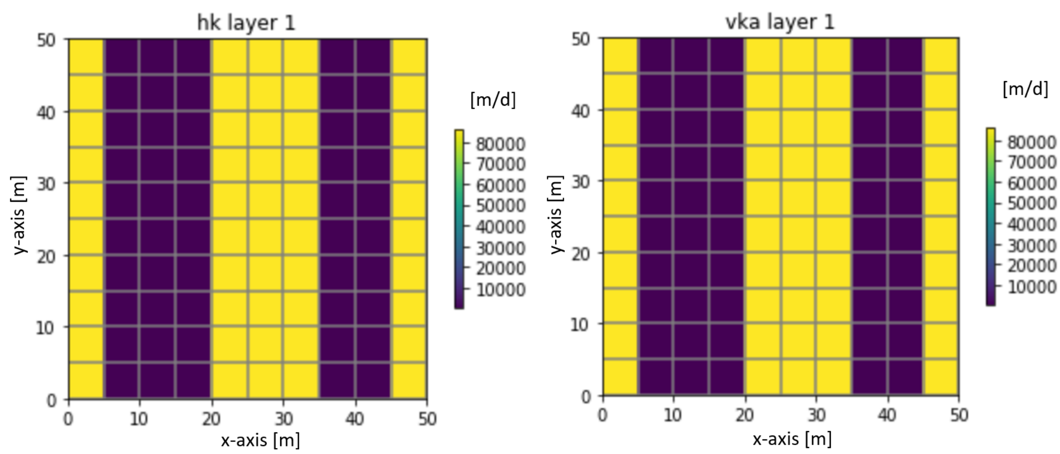


Figure 2.7: Hydraulic conductivity horizontal and vertical, left and right respectively

An alternative approach to model the basin is to use a top boundary for layer 1 that acts as the surface profile. However, intermediate results have shown that large water balance errors occur in the space above the surface profile and thus this approach produces unrealistic results.

BOUNDARY CONDITIONS

The boundary conditions in the model determine the specific location where and how water enters the model domain. The external boundary of the grid in Modflow is by default assumed to be a no-flow (impermeable) boundary.

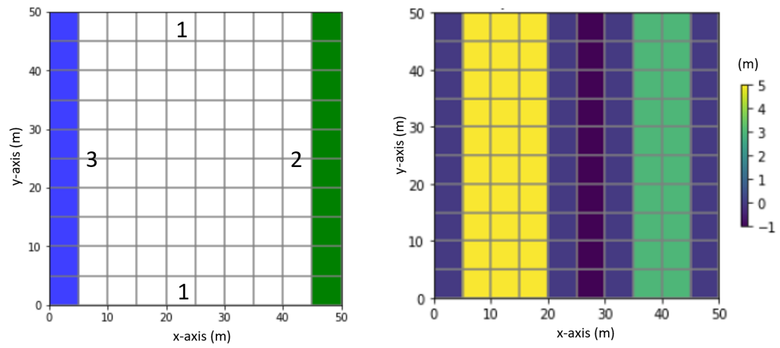


Figure 2.8: Boundary conditions on the grid (x,y)

The water in the Pipingontspanner situation enters through the river boundary and exits the system at the polder boundary. Three types of boundaries are used in the model:

1. A no-flow outer boundary at y = 0 and y = 50 m, which is set by default in Modflow.
2. A constant head for the polder water level using a head-dependent flux boundary at x = 50 m.
3. A river boundary at x = 0 m to simulate the river water intrusion in the three layers.

The polder water level is a constant head at the reference level and is located a certain distance from the river. To limit the size of the model domain, the constant head boundary is simulated by a head-dependent flux boundary at x = 50 m, which calculates the flow through the boundary with Darcy's law and the head gradient between the two points.

The actual distance of the polder boundary is determined by the influence length of the river water level. The influence length (L_i) for a river next to a polder is given by the leakage length (λ_h). At x = L_i, the polder head is no longer influenced by the river water level and remains constant (USACE, 1992). This distance is around 500 m for the properties given in Table 2.1.

$$\lambda_h = \sqrt{T_{aquifer} \cdot C_{aquitard}} \tag{2.6}$$

The river level varies according to an example river flood wave from the example project (Waterontspanner). The example flood wave is translated into a step function with a step size of 12 hours, as depicted in Figure 2.9.

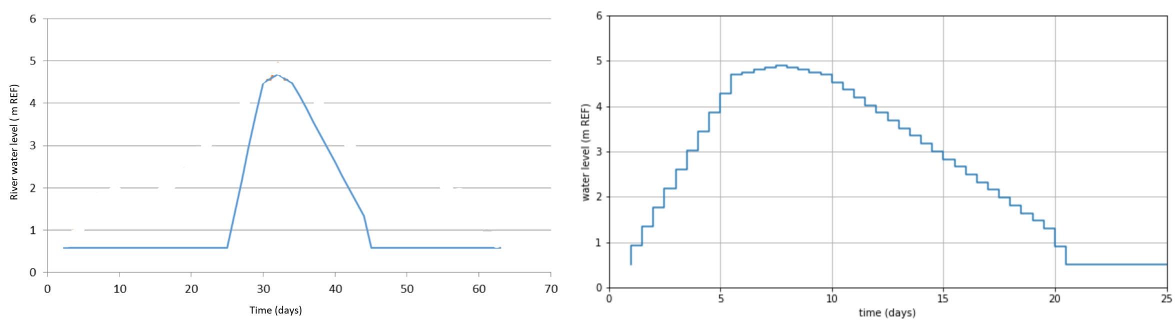


Figure 2.9: Flood wave example and schematisation, left and right respectively

The river water level uses a head-dependent flux boundary that takes the hydraulic resistance of the streambed into account. The streambed conductance is estimated with the formula: $C_r = \frac{K_h \cdot W_r \cdot L_r}{D}$ (USGS, 2005). The initial streambed hydraulic conductivity (K_h) is assumed to be equal to the vertical hydraulic conductivity of the connecting soil layer. The width of the river (W_r) is 100 m, the length (L_r) is equal to the length of the calculation cell and the thickness (D) is equal to the streambed thickness which is assumed 3 m.

Well implementation

The well forms an additional boundary condition in the model domain and is located between the dyke and seepage dyke at point (x,y) = (27.5,22.5) m. The relief well design is taken from the example project (Waterontspanner) and consists of a vertical pile of coarse gravel with a diameter of 0.5 m.

Modflow offers two packages (DRN and WELL) that simulate wells, which extract water from the aquifer by either a specified discharge or until a certain water level. However, both of these packages extract water from the system and do not store the water in the seepage basin. An alternative approach is taken that uses the properties in a certain cell to simulate the workings of a relief well. The hydraulic resistance of cell should be equivalent to the well. A smaller cell size would increase the accuracy of flow in the well.

Measurements of the head gradient (dh/dr) near the well are necessary to determine the entry resistance of the well accurately. There is little information available for accurate estimates of the resistance, which depends highly on the conductivity of the surrounding soil and design/construction of the well (Niemeijer et al., 2017). The maximum well discharge (Q_{max}) is used to calculate the equivalent vertical hydraulic conductivity of the well cell ($K_{v,well}$). The maximum filter velocity (u_{max}) in the relief well is determined with the frequently used empirical formula of Sichardt ($u_{max} = 10 \cdot \sqrt{K_h}$). Combining the filter velocity (u_{max}) with Eq (2.4) and the continuity equation provides Q_{max} and $K_{v,well}$:

$$K_{v,well} = \frac{Q_{max}}{A_{cell}} \qquad Q_{max} = 2\pi r_w \cdot D \cdot v_{max} = 2\pi r \cdot D \cdot 10\sqrt{K_h} \tag{2.7}$$

A_{cell} cross-sectional area cell well [m^2]
r_w radius well [m]
D aquifer thickness [m]
K_h horizontal hydraulic conductivity of aquifer [m/d]

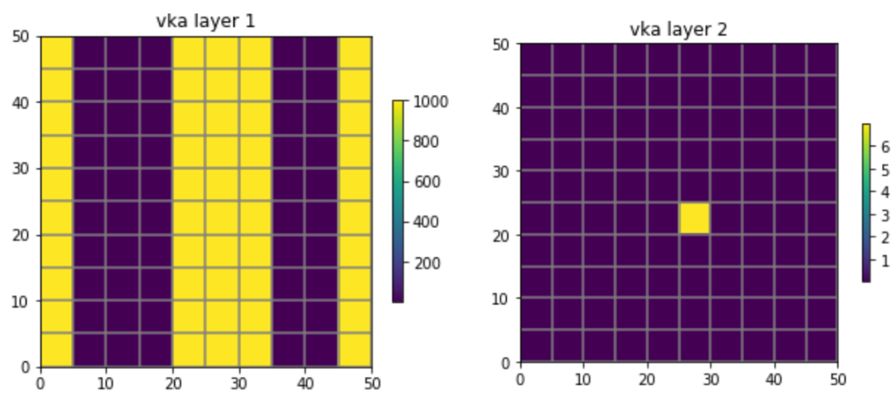


Figure 2.10: Vertical hydraulic conductivity of the well cell, layer 1 and layer 2

For a cell size of 5 x 5 m, this would lead to a well conductivity ($K_{v,well}$) of 10 m/d. The conductivity is scaled up for smaller cells by keeping the discharge equal under the changing area of the well cell.

2.3.3. COMPUTATION

The schematisation is initially discretised into 3 layers, 10 rows and 10 columns. This configuration would render a cell size with a width (Δx) of 5 m, a length (Δy) of 5 m and a height (Δz) equal to the layer thickness. The computations are performed with the block centred flow option, in which the calculation node is centred inside each cell. The resolution of the cells is initially kept low as the main goal is to analyse the global phenomena. The low resolution causes some inaccuracy as the linear interpolation between the cells can deviate from reality.

The total run time is 100 days with a variable time step. The river water level is simulated over 25 days with a time step of 2 hours, after which the time step increases to 48 hours to reduce the calculation time. The effects of the variable river water level cancel out in the 100 days and result in a steady-state condition at t =100 days.

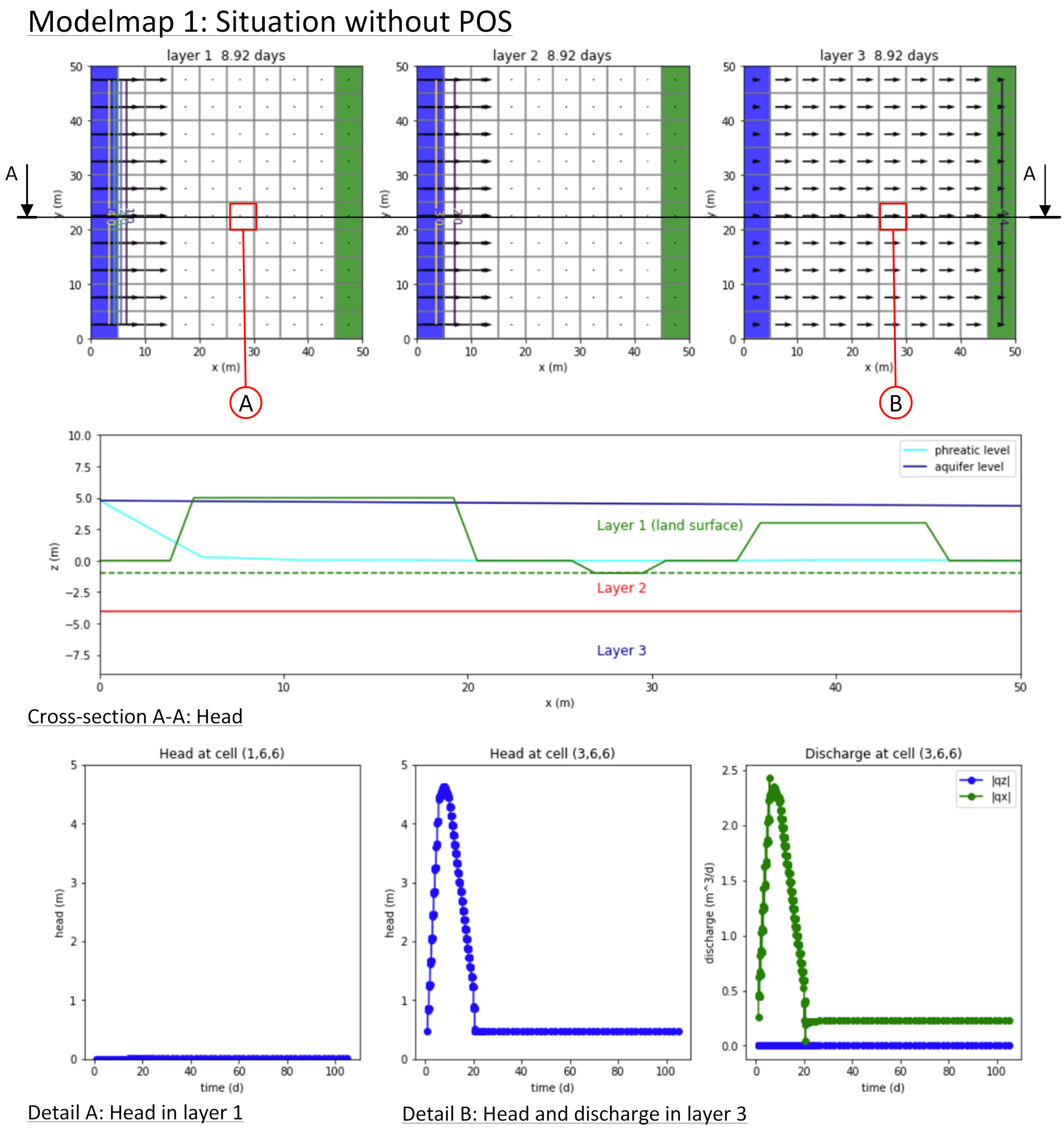


Figure 2.11: Modflow results without Pipingontspanner

2.3.4. RESULTS

The output of Modflow is the simulated hydraulic head and discharge for all nodes in the model domain. The data of the nodes are processed into 2D graphs to evaluate the results. Firstly, the situation without the Pipingontspanner implementation is examined to show the basis model on which the measure is implemented. Secondly, the results of the Pipingontspanner model are analysed to understand the influence of the well on the hydraulic head in the polder.

SITUATION WITHOUT PIPINGONTSPANNNER

The results for the base model are presented in Modelmap 1, Figure 2.11. Modelmap 1 displays the head and flow direction for each layer at the maximum river water level (occurring around t=9 days). The streamlines indicate that flow is polder-directed and uniform along the y-axis. From this modelmap, a cross-section A-A is taken at y = 22.5 m that shows the phreatic surface and the head of the aquifer. The polder ground level remains dry as the water is held back by the impermeable dyke and cover layer.

Detail A shows the water level of the polder over time at point (x,y) = (27.5,22.5) m and indicates that only a minimal amount of groundwater reaches the surface by seepage through the cover layer. Detail B describes the head and discharge at point (x,y) = (27.5,22.5) m and shows that the head follows the river water level without large delays. The pressure from the river infiltrates the aquifer and the head slowly slopes down to the constant head boundary at x = 500 m. The maximum discharge that is depicted in detail B can easily be verified with the equation 2.1:

$$Q_{aquifer} = 10 \cdot \frac{4.8}{500} \cdot (5 \cdot 5) = 2.4 \text{ m}^3/\text{s per cell width.}$$

This base situation is vulnerable to the Uplift mechanism of piping as the maximum upward pressure in the aquifer ($\Delta\phi_{max} = 4.8 - 0 = 4.8$ m at t= 9 days) is larger than the critical potential limit ($\Delta\phi_{c,u}$) provided by the cover layer. The critical potential limit can be determined with:

$$\text{FOS} = \frac{\Delta\phi_{c,u}}{\Delta\phi_{max}} \geq 1 \qquad \Delta\phi_{c,u} = d_{blanket}\frac{\gamma_{sat,blanket} - \gamma_w}{\gamma_w} = 3\frac{18-10}{10} = 2.4 \text{ m.}$$

SITUATION WITH PIPINGONTSPANNER

The results for the Pipingontspanner model are presented in Modelmap 2, Figure 2.14. In contrast to Modelmap 1, the streamlines of the aquifer in layer 3 are now curved towards the well. The relieved groundwater from the well spreads throughout the seepage basin and slowly starts to fills up. The cross-section A-A displays the phreatic surface in the seepage basin and the head in the aquifer with the last showing a minimum around the well. Detail A depicts the water level in the seepage basin over time with a maximum water level at REF +1.3 m occurring 10 days after the maximum river level. The maximum discharge through the well (Q_z) is 85 m^3/d or around 0.001 m^3/s.

Details C and D describe the head and flow around the well in the aquifer. The detail D shows a positive discharge of 33 m^3/d towards the well, whereas detail C displays a negative flow of 18 m^3/d at the well. This phenomenon can potentially be explained by a local decrease in the head at the well due to an equivalent increase of velocity head causing a radial flow towards it.

When comparing the head of the aquifer near the well in detail C for Modelmap 1 and 2, the head difference can be determined. The influence of the well is reflected in the head difference and the stored water volume in the seepage basin (Figure 2.12). The maximum head difference is 1.8 m, which reduces the upward pressure in the aquifer from 4.8 to 3 m. The critical pressure ($\Delta\phi_{c,u}$) is strengthened by the water in the basin resulting in $\Delta\phi_{c,u} = 2.4 + 0.5 = 2.9$ m. It can be concluded that the Pipingontspanner does not prevent Uplift for this generic configuration, however does significantly reduces it.

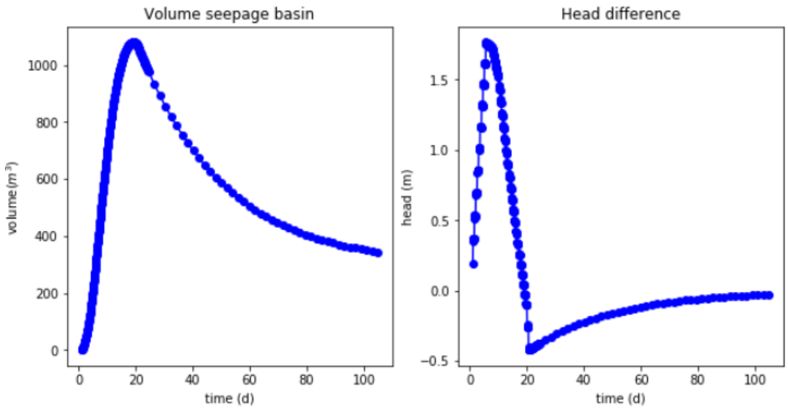


Figure 2.12: Results influence well: basin volume and head difference

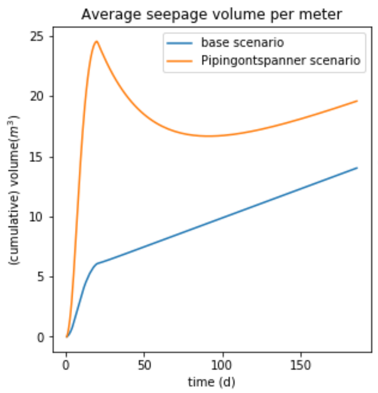


Figure 2.13: Results seepage volume

Appendix A.1 analyses the amount of seepage entering the polder via the aquifer, which is important for the water management as an increase could have consequences for Waterboards. When comparing the situation with and without the Pipingontspanner in Figure 2.13, the seepage volume into the polder is higher for the Pipingontspanner. The well draws more water from the river, of which most is stored in the seepage basin, but when the river level becomes lower than the basin level a part of the volume flows back into river. This scenario is only possible when the basin level exceeds the river level and the hydraulic resistance of the river bottom is low. Overall, the Pipingontspanner did cause an additional seepage volume of 6 m^3 per meter dyke per flood wave.

Appendix A.2 shows the influence of different cell sizes by comparing the results for a cell size of 5, 2.5 and 1.25 m. The results show that the maximum discharge in the relief well is approximately equal for all cases, but the aquifer head decreases with a smaller cell size. This phenomena is caused by an increasing velocity in the well due to the smaller cell size. The head reduction across the well is proportional to the velocity in the well and thus results in a lower aquifer head. Based on these findings, a cell size of 1.25 m is selected for the hydraulic model as the number of calculation points is sufficient and the cell size is close enough to the actual diameter of the well.

2.4. FINAL REMARKS

The analytic and semi-empirical formulations currently available are not able to describe the flow through the relief well. The time-dependent relation between the river and the seepage basin could not be modelled with these formulations. However, the numerical hydraulic model is able to describe the Pipingontspanner implementation using Modflow and some essential model choices:

- The empty model space and basin are modelled as soil with a high porosity. This means that results related to the flow velocity inside the basin cannot be used accurately. However, the main variable of interest, the basin volume, can be accurately obtained using this method.
- The well is modelled as soil with an hydraulic resistance equivalent to the well. This assumption produces a constant discharge through the well for each model cell size. Consequently, the flow velocity varies per well cell size and influences the maximum achieved head difference. The last becomes more accurate when the cell size decreases to the well diameter.
- The model domain is set to 50 by 50 m, which was sufficient for a single well. However, this selection will influence the head of the system in the case of multiple wells and will be extended in that case.

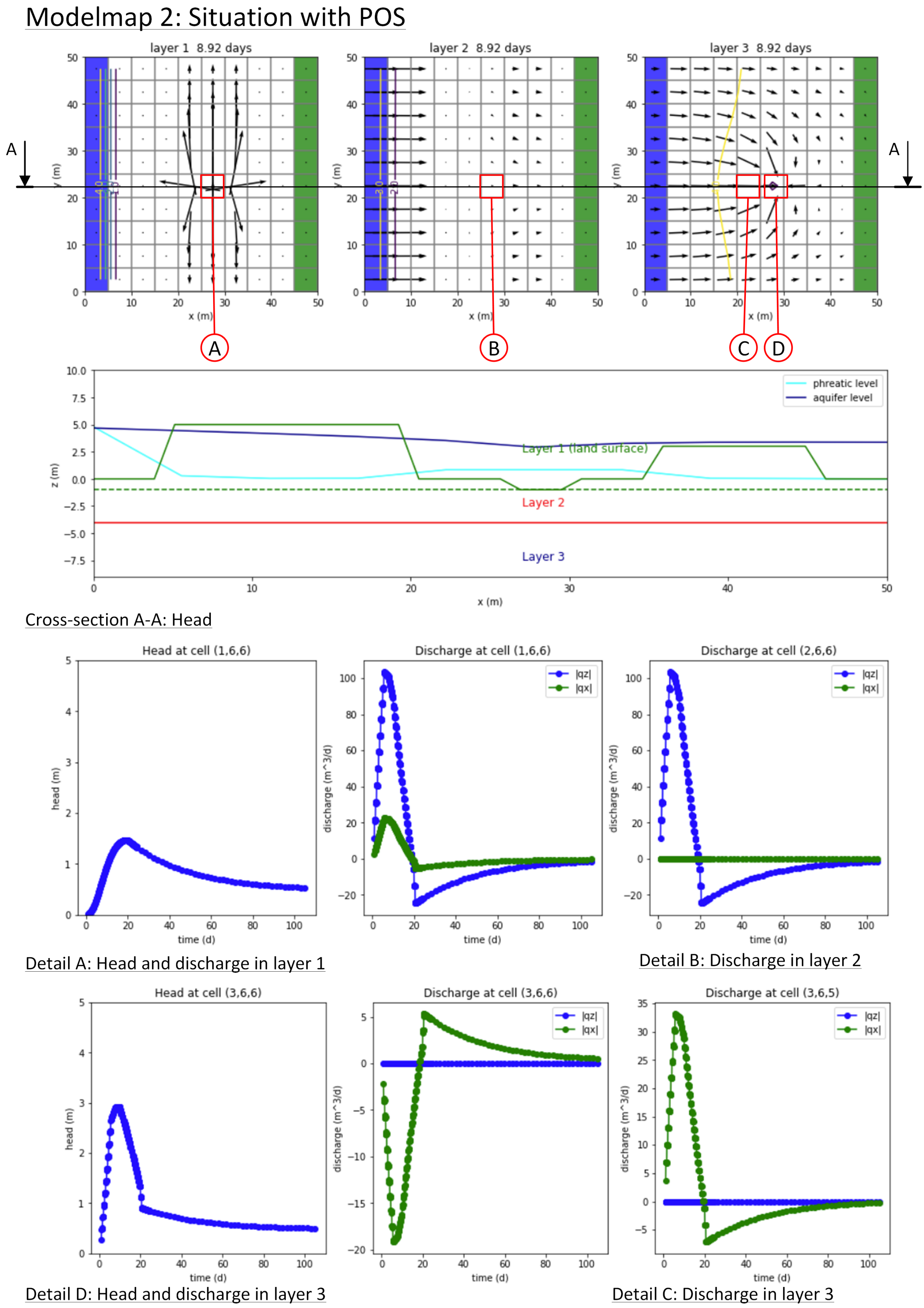


Figure 2.14: Modflow results with Pipingontspanner

3

Development of Pipingontspanner model

The second and third research questions are treated in this chapter and revolve around the design of the Pipingontspanner. The focus lies on describing and developing a Pipingontspanner model using the groundwater flow model of Chapter 2. The Pipingontspanner model is an instrument that calculates and weighs different design options to obtain an optimal configuration. The next chapter continuous by evaluating this model under various conditions.

The verification of the reliability is an integrated part of the design and is included in the Pipingontspanner model. Section 1.2.1 described the Piping failure mechanisms, but does not include the influence of the Pipingontspanner on the piping reliability. This influence is examined in section 3.1 and answers research question 2. Sections 3.2 and 3.3 describe and create a model for the Pipingontspanner and answer research question 3.

3.1. Reliability Flood defence

The Pipingontspanner is based on a drainage measure that relieves pressure and blocks sediment transport, and with that, it prevents the three mechanisms in piping (Uplift, Heave and Piping). The reliability of the flood defence is determined by assessing those mechanisms. Besides the piping mechanisms, the overall reliability of the flood defence is influenced by the addition of the drainage component to the flood defence system. In the following two sections, the reliability for piping and the verification models are determined.

3.1.1. Piping reliability drainage techniques

Drainage techniques are the solution to lower the pressure head in the aquifer and thus help reduce the piping occurrence. However, by the addition of the drainage system, the flood defence also inherits its reliability and uncertainty. Potential failure causes of drainage systems can be related to degradation, execution errors, vandalism, or model errors. The Pipingontspanner relies on the drainage system to achieve a certain drawdown. Piping failure can occur when the drainage systems provide insufficient drawdown resulting in the following possible failure scenario's:

1. Piping failure occurs, but the calculated drawdown is achieved.
2. Piping failure occurs due to an insufficiently functioning well system (calculated drawdown is not achieved).

The response of the dyke and well system to a high water event is presented in the event tree below and corresponds to the different failure scenario's. Piping failure ($P(D_f)$) is defined as a failure by the Uplift, Heave and Piping sub-mechanism.

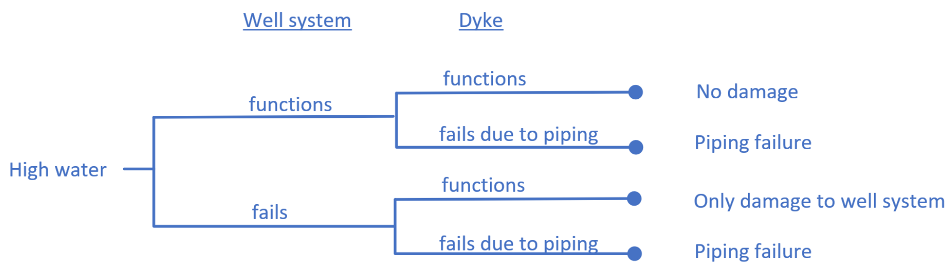


Figure 3.1: Event tree Pipingontspanner

The failure of a well system ($P(W_f)$) is determined by a decomposition of the well elements. The well consists of a pipe connected to the surface and a filter structure. The failure mechanisms for the pipe connection are bursting, damaging by external causes, blockage and freezing. The failure mechanisms for a filter construction depend on the type of filter but are related to the particle flow transport. In such a case, the filter could be damaged during construction or exposed to a load more substantial than the design load. Parts could also fail due to degradation in the absence of the required maintenance.

The event tree can be transformed into a fault tree, which is presented in Figure 3.2. The fault tree shows connections between failure elements and allows the determination of the overall failure probability for piping.

A failure probability analysis uses the decomposition of the well to specify the well system failure probability ($P(W_f)$). For each part in the decomposition, a failure mechanism and probability are assigned. Once the design for the well is set, the failure probability ($P(W_f)$) can be determined with the help of the guidelines from the drinking water industry (Niemeijer et al., 2017). The failure probability per element is largely influenced by the design, management and maintenance.

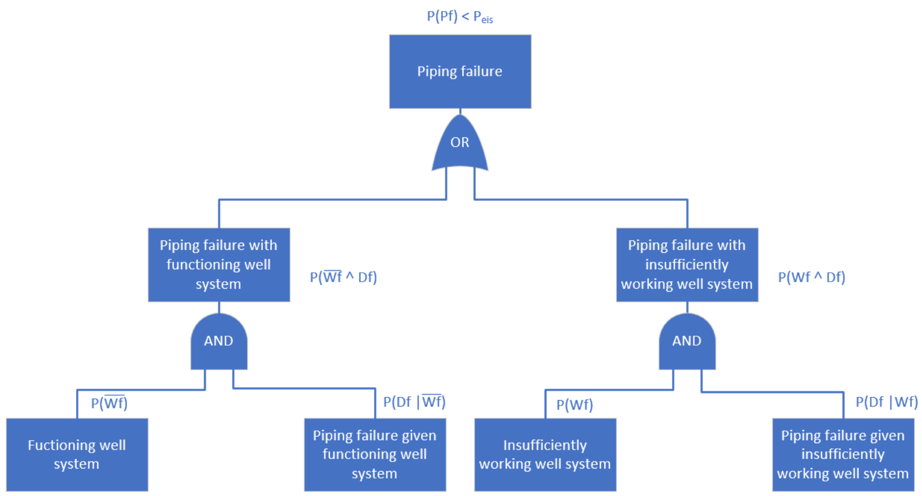


Figure 3.2: Fault tree Pipingontspanner

3.1.2. VERIFICATION MODELS PIPINGONTSPANNER

The failure probability of piping ($P(D_f)$) is determined with a semi-probabilistic method, in accordance with the WBI2017, using the Uplift, Heave and Piping verification models that are described in this section. A semi-probabilistic method uses the characteristic values (5 or 95-percentile) of stochastic parameters to obtain the safety factor in each verification model. The safety factor is transformed into a failure probability by means of the calibrated formulas below ($P_f = \Phi(-\beta)$).

$$\beta_u = \frac{ln(\frac{FOS_u}{0.48}) + 0.27\beta_{norm}}{0.4} \qquad \beta_h = \frac{ln(\frac{FOS_h}{0.37}) + 0.3\beta_{norm}}{0.48} \qquad \beta_p = \frac{ln(\frac{FOS_p}{1.04}) + 0.43\beta_{norm}}{0.37} \tag{3.1}$$

Generally, piping failure is determined with the verification models of Section 1.2.1 with equations 1.3, 1.4 and 1.5. However, these verification models need to be adapted to account for the Pipingontspanner influence.

UPLIFT AND HEAVE

The Uplift mechanism describes the forces on the cover layer, while the Heave mechanism focuses on the water level gradient over the cover layer. Both mechanisms depend on the water pressure underneath the cover layer (ϕ_{exit}). The Pipingontspanner counteracts the Uplift and Heave mechanism using two aspects:

1. a hydraulic head reduction underneath the cover layer ($\phi_{exit,red}$);
2. an increased phreatic water level formed by the seepage basin ($h_{p,exit}$).

The current form of the Heave verification model (Eq.1.4) includes these parameters and results in equation 3.2.

$$Z_h = i_{c,h} - i = i_{c,h} - \frac{\phi_{exit,red} - h_{exit}}{d} \qquad FOS_h = \frac{i_{c,h}}{\frac{\phi_{exit,red} - h_{p,exit}}{d}} \tag{3.2}$$

For the Uplift mechanism, the second reduction aspect can be interpreted in two ways. Firstly, the water in the basin acts as a weight on top of the cover layer and produces a downward pressure. Secondly, the water in the basin infiltrates the layer below and increases the pore water pressure in the cover layer. The main difference between both interpretations is the permeability of the cover layer, because the infiltration time is limited to about 20 days. Since most piping situations occurs for impermeable cover layers, the first case is chosen for this study.

The Uplift verification model (Eq.1.3) is altered to include the weight from the water inside the seepage basin. This aspect is incorporated by adding the weight of the water in the basin (h_{sb}) to the resistance term, which results in equation 3.3.

$$Z_u = m_u \cdot (d\frac{\gamma_{sat} - \gamma_w}{\gamma_w} + h_{sb}) - (\phi_{exit,red} - h_{p,exit}) \qquad FOS_u = \frac{d\frac{\gamma_{sat}-\gamma_w}{\gamma_w} + h_{sb}}{\phi_{exit,red} - h_{p,exit}} \tag{3.3}$$

Figure 3.3 shows the locations where Uplift and Heave will appear with the implementation of the Pipingontspanner. Both mechanisms are most likely to occur at the edge of the wells influence radius, where the head drawdown is minimum. In practice, this position occurs along a centre line between two wells. Heave will follow Uplift and is therefore evaluated on the same plane. Additionally, Heave occurs when objects are protruding into the cover layer, such as the well in the case of the Pipingontspanner. The locations of the critical points on the Uplift and Heave plane is unknown and thus is examined for the entire plane.

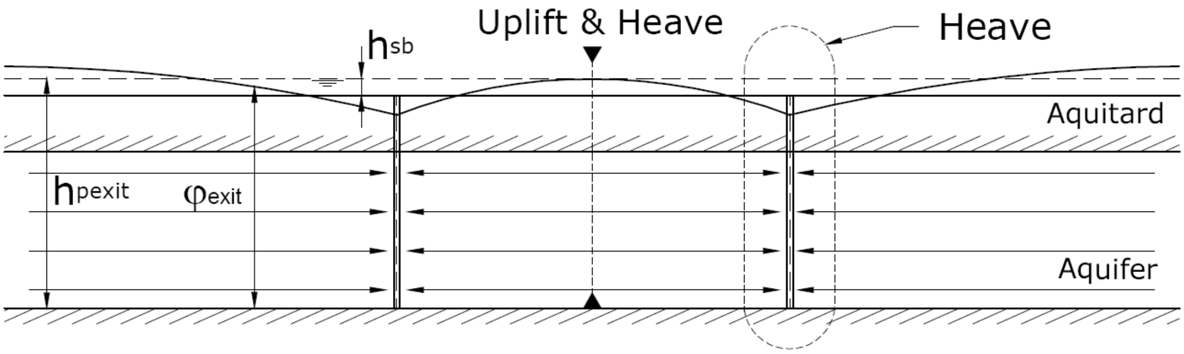


Figure 3.3: Assessment location Uplift and Heave mechanisms

PIPING

The influence of drainage measures is not included in current Piping verification models. The Sellmeijer formulation is applicable but would neglect the reduction effects of the Pipingontspanner. The erosion process of Piping can also be evaluated with numerical solutions such as D-Geoflow (3D) and Mseep (2D). However, both of these solutions cannot model the groundwater flow of the Pipingontspanner or be integrated into the groundwater flow model of Chapter 2.

An alternative approach (Schoonen and Mols, 2015) uses a new instrument and automated pre- and post-processing to acquire the safety of Piping. The automated processes use the groundwater flow model Modflow to asses Heave and an iterative loop to describe the pipe development. A scheme for this process is depicted in the diagram below. This approach is solely based on the seepage gradient and the critical heave gradient. The properties of the soil and erosion mechanisms are not explicitly included, in contrast to the numerical models Mseep and D-Geoflow.

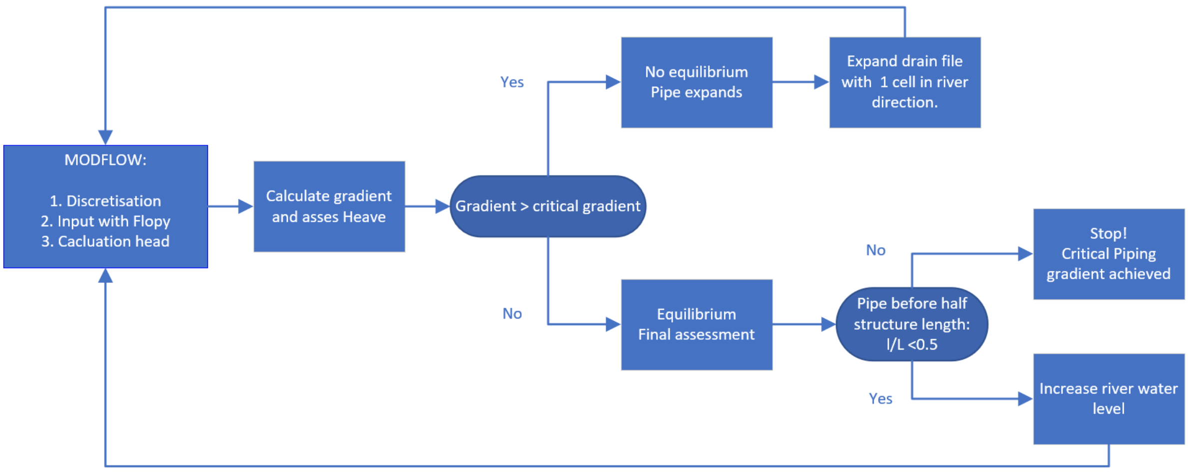


Figure 3.4: Approach to implementing Pipe development (adapted schematic (Schoonen and Mols, 2015))

Currently, it is unknown how this approach compares against the Sellmijer 2011 formulation or the numerical methods (D-geoflow or Mseep). Ultimately, the Sellmeijer formulation (Eq 1.5) is used in this study to assess piping because it is prescribed by the WBI2017 assessment guidelines, even though it does not account for drainage measure effects.

3.2. PIPINGONTSPANNER MODEL

The design of the Pipingontspanner depends on different fields (hydrology, hydraulic and geologic) that come together in one design. The fact that a hydraulic model is required to estimate the effects of design changes causes difficulties in the development of alternatives. A parametric design approach is chosen to create alternatives and determine the best option according to specified criteria. The technical design process presented in Figure 3.5 is used to create the Pipingontspanner design and only focuses on alternatives within the Pipingontspanner concept.

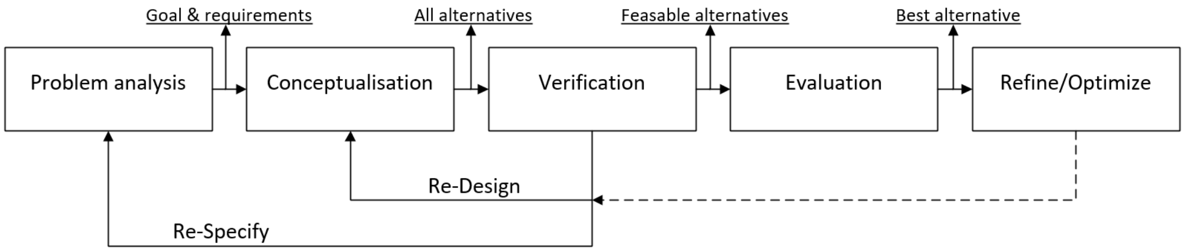


Figure 3.5: Systematic parametric design (Eggert, 2005)

The parametric design consists of the conceptualisation, verification and evaluation phase, where alternatives are created and chosen. In the conceptualisation phase, alternatives are generated with different design parameters such as dimensions or materials. Followed by the verification phase, where the performance of each alternative is determined and compared against defined requirements. During the evaluation phase, the best alternative is chosen based on design criteria such as the maximisation or minimisation of a certain property. The aforementioned steps are included in a parametric model that produces the best alternative.

In the following sections, the problem analysis, conceptualisation and verification phases are specified. The setup of the parametric model is treated in the next section.

3.2.1. PROBLEM ANALYSIS

The problem analysis maps the challenges to overcome and describes a dyke section with piping problems. The Pipingontspanner principle is based on the problem defined in Section 2.3.1 and is summarised by Figure 3.6.

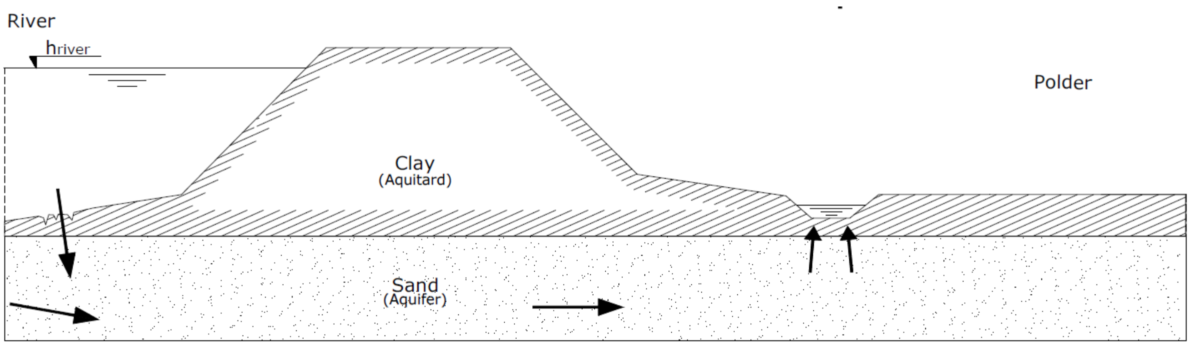


Figure 3.6: Schematic problem Pipingonstpanner

A primary dyke section bordering a river that is sensitive for the Piping mechanism, which is characterised by a permeable aquifer and a shallow cover layer behind the dyke. The governing hydraulic load is a river flood wave on top of the mean high water level. This load causes the cover layer to lift up and starts a piping process.

The primary purpose of the measure is to reduce the probability of piping to the required reliability norms. These (national) norms are the leading requirement in the design and often the reason why dyke sections need improvements. The Pipingontspanner is in an early development stage, and the main goal of this design is to assess the feasibility on a global scale. Therefore, safety and financial requirements are centralised to acquire the economic feasibility of the measure.

The design requirements for the Pipingontspanner consist of reliability requirements and boundary conditions. The hydraulic and geometrical boundary conditions of the problem in Section 2.3.1 still hold. Furthermore, the following requirements and boundary conditions are added to the design:

1. **Reliability.** The combined flood defence (dyke and Pipingontspanner) needs to comply with the current national safety norms. These norms are included in the Water Act of 2017 and specify per segment and failure mechanism the required failure probability (Staatscourant, 2016). Note that piping is not the only geotechnical mechanism affected by the measure, which means that other mechanisms such as macro stability need to be checked as well.
2. **Boundary conditions.** The following conditions are bound to a typical dyke section in the Netherlands and need to be integrated into the design.

    (a) The amount of seepage discharge is limited and determined by each waterboard and per case (Niemeijer et al., 2017). The use of drainage measures affects the seepage discharge to the hinterland. The water balance inside a polder is a complex composition of precipitation, evapotranspiration, seepage and discharge to external water systems. The pumping stations maintain a certain level according to the water level agreement inside the polder. An increase of seepage discharge influences this water budget and cause a nuisance for waterboards to keep the fixed levels.

    (b) The space behind the dyke is limited and therefore a boundary for the design. This area could consist of property boundaries, houses or other objects that possibly restrict the size of the seepage basin.

3.2.2. CONCEPTUALISATION

The conceptualisation is the generation of concepts or alternatives, which could potentially form the solution to the problem. This process is also referred to as synthesis. This section analyses the system behind the Pipingontspanner concept to identify the mechanisms and design variables that are used to create the alternatives.

GENERAL SYSTEM

The general design consists of two components: a conveying part and a storage part, which interact to form the response of the water level in the seepage basin. This system causes a change in the hydraulic head for the hinterland as a result of the Pipingontspanner.

In the conveyance and storage system, there are three design elements which can be manipulated: the well, the seepage basin and the seepage dyke. The storage part is defined by the seepage basin (surface area, A_{sb}) and the seepage dyke (allowable height, h_{sb}), in which the water only varies as a function of time and not space. The conveying part is represented by the well (surface area, A_w, permeability, $K_{v,well}$), which forms a connection between the storage basin and the changing head in the aquifer. In the well, the discharge only varies as a function of time as there is no storage volume present.

This system resembles to the *small basin approximation* (Eq 3.4) when resistance in the conveyance part is neglected. This approximation deviates slightly from the Pipingontspanner, where the permeability of the well is a form of resistance and thus a damped system. However, the small basin equation provides an excellent insight into the relation between wells discharge and storage surface and in case of small discharges the influence of resistance remains small.

$$Q_{well} = A_{sb}\frac{dh_{sb}}{dt} \tag{3.4}$$

The overview below shows the variables and relations present in the design that can be adapted to different situations.

Variables present in the design:

- basin width (w_{sb})
- spacing between wells (L_w)
- height seepage dyke (h_{sd})
- radius well (r_w)
- conductivity well ($K_{v,well}$)

Relations present in the design:

- seepage basin surface area ($A_{sb} = L_w \cdot w_{sb}$)
- basin volume ($V_{sb} = A_{sb} \cdot h_{sb}$)
- well cross-sectional area ($A_w = \pi r_w^2$)
- discharge well and basin surface ($Q_w \sim A_{sb}$)

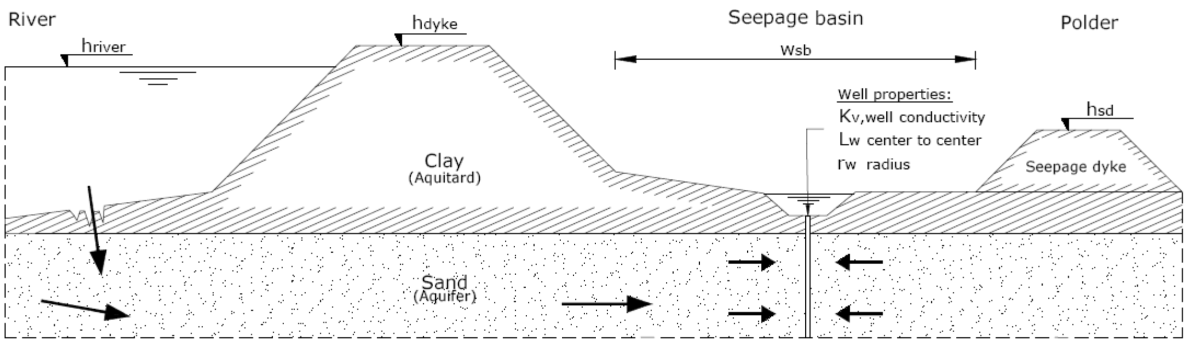


Figure 3.7: Schematic Pipingontspanner variables

ALTERNATIVES

The variables in the general system are used to generate different configurations of the Pipingontspanner. Each variable is given a certain range of values that combine into multiple unique alternatives. From the list of available variables, only the basin width (w_{sb}) and well spacing (L_w) are selected to vary in the alternatives. More design variables would lead to overcapacity of alternatives and increase the calculation time substantially. The remaining variables are assigned a constant value or a design rule that allows them to adapt to different situations. Figure 3.8 shows the design variables.

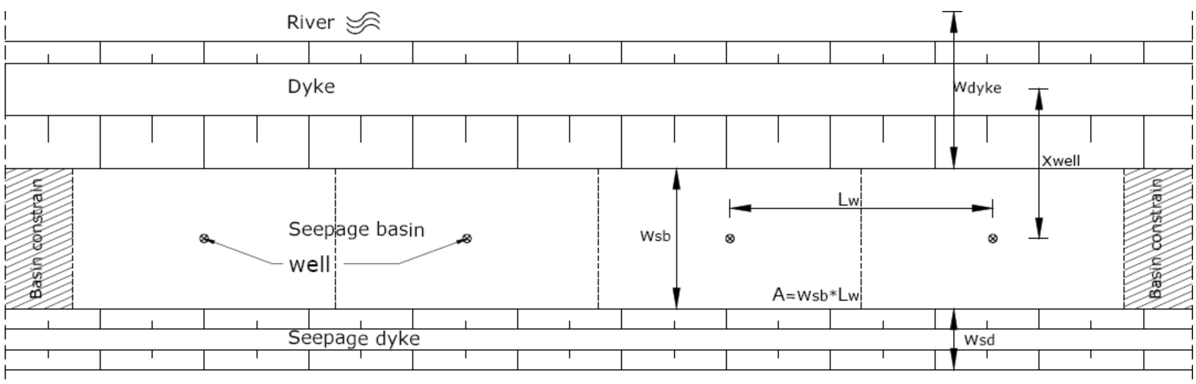


Figure 3.8: Overview Pipingontspanner design variables

The basin width and well spacing will vary from 5 to 20 m with a step of 1.25 m. This initial range results in 144 unique alternatives that will be subjected to verification and evaluation.

The height of the seepage dyke can be related to the maximum water level in the basin ($h_{sb,max}$) to prevent overflow into the polder. This water level is given by a steady-state situation with the highest river level and is expressed by an exponential decay between the two given water levels (h_{river}, h_{polder}) (Niemeijer et al., 2017).

The characteristics of the well are determined by the radius and conductivity. In the formulation of the groundwater flow model, the rule of Sichardt is used to estimate the conductivity of the well based on the radius. In reality, the conductivity mostly depends on the type of the well, but in this initial design the previous method will continue to be used to maintain simplicity in the model.

The radius of the well is assigned a constant value of 0.25 m. The location of the well is placed in the middle of the basin since until now it is still unsure what the influence of this position is on the head reduction and reliability.

The three design relations are transformed into uniform design rules that can describe the structure. The initial design can be specified according to:

1. distance well from river: $x_{well} = \frac{w_{sb}}{2} + w_{dyke}$
2. seepage dyke height: $h_{sd} = h_p - (h_p - h_{r,max})e^{-\frac{x_{well}}{\lambda_h}}$
3. conductivity well: $K_{v,well} = (2\pi r_w \cdot D \cdot 10\sqrt{K_{h,aquifer}})/A_{cell}$

In the optimisation phase, the design relations can be altered to produce a better result. For example, the dyke height can be lowered (optimised) by discharging the water at a lower level from the basin, which also prevents overflow. The reduction in height results in a smaller footprint with reduced construction costs.

3.2.3. Verification

In the verification phase, the design alternatives are assessed on the requirements specified in the problem analysis. These requirements consist of piping reliability, maximum seepage discharge and maximum measure width. The groundwater flow model calculates the effects of each alternative and those results are used to verify whether the design is feasible considering the requirements.

The reliability of the flood defence needs to satisfy the norm required by the Water Act. The addition of a drainage system to the flood defence influences the overall piping reliability. Section 3.1 shows how the overall piping reliability can be determined with verification models. The maximum width behind the dyke (x_{max}) is a geometry constraint. The maximum seepage discharge ($Q_{s,max}$) originates from the responsible Waterboard and is assessed with the groundwater flow model.

These aspects are translated into verification rules that can determine feasible alternatives. The feasible alternatives are specified according to:

1. Maximum width behind the dyke: $x_{seepagedyke} \leq x_{max}$
2. Maximum seepage discharge: $Q_{s,alternative} \leq Q_{s,max}$
3. Reliability: $P_{Pipingfailure} \leq P_{piping,req}$

3.2.4. EVALUATION

The evaluation phase determines the best alternative from a set of feasible alternatives. The best alternative can be defined by different criteria such as robustness, in which a high reliability is requested, or the lowest construction cost. Frequently, there are multiple objectives, and that results in a trade-off behaviour with compromises between the different goals. A trade-off function can describe this trade-off behaviour.

The goal of this system is to reduce the probability of piping to the required norm for the best economical value. A decision-maker criterion for this goal is the maximum value for money over the entire lifespan, which can be described by the Life Cycle Cost (LCC) of the structure. The LCC is the sum of all costs over the lifespan of the structure including design, construction, operation, maintenance and removal costs. The system trade-off function is coupled to the LCC formulation of the Piping measure and is described by:

$$\text{Trade-off function} = \min LCC(w_{sb}, L_w) \tag{3.5}$$

The Life Cycle Cost of the Pipingontspanner is defined in Chapter 5. However, an example is presented in the table below to show the structure of a LCC function in Net Present Value (NPV).

Table 3.1: Life Cycle Cost example Pipingontspanner

Cost type\Phase	Unit	Design	Construction	Operation	Maintenance	Removal
Fixed cost	euro	3000	6000			1000
Returning costs	euro/lifetime			2500	3500	
Cost per well (n)	euro/well		500			200
Cost per area (A)	euro/m^2		10			
LCC/Dyke length	euro/50m	$10,000 + 700 \cdot n + 6000 + 10 \cdot A$				

3.3. PYTHON IMPLEMENTATION PIPINGONTSPANNER MODEL

The parametric design approach is integrated into a parametric model and includes the conceptualisation, verification and evaluation phase. The basis for the model is a Python script that incorporates the groundwater flow model with the verification models and evaluation trade-off function.

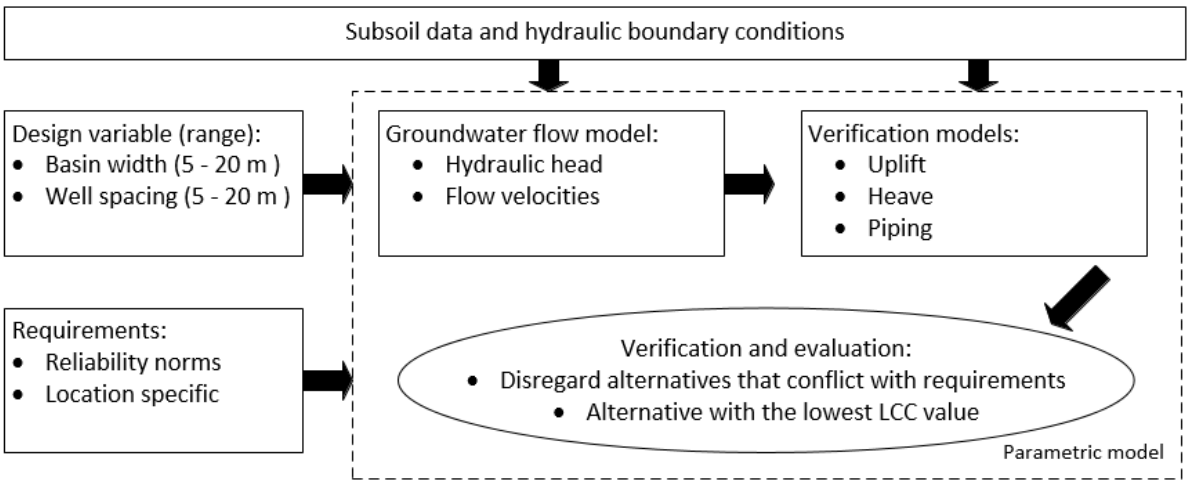


Figure 3.9: Overview parametric design

This numerical approach is used to calculate different combinations of design variables and results in the best alternative. An overview of the parametric model scheme is depicted in Figure

3.9. Additionally, the model can be used to provide insight into the different configurations of the Pipingontspanner.

The Python script combines the groundwater flow model with the design rules, verification models and evaluation function from the previous sections to calculate the design effects on the Pipingontspanner. The groundwater flow model, described in Chapter 2, uses a single well within an area of 50 m by 50 m and a cell resolution of 1.25 m. However, the solution should prevent piping along the entire dyke segment. The previous situation of a single well is extended to a system of wells, which requires a minimum of four wells for the model to simulate the head between wells accurately. The new situation, presented below, contains four wells in an area of 200 m (y-direction) by 50 m (x-direction), which will be the basis of the groundwater flow model.

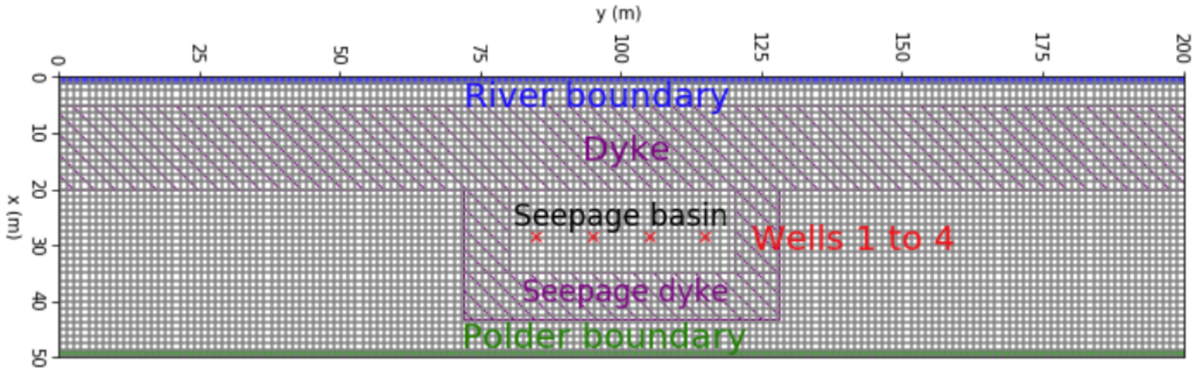


Figure 3.10: Overview model domain and location wells

The input for the parametric model consists of design variables, soil parameters, boundary conditions and site-specific elements. The last concerns the layer system and geometry and is based on the situation defined in Section 2.3. The chosen design variables for the Pipingontspanner design are well spacing (L_w) and basin width (w_{sb}), which are depicted in the figure below. The location and chosen scenario determine the remaining parameters and boundary conditions in the model.

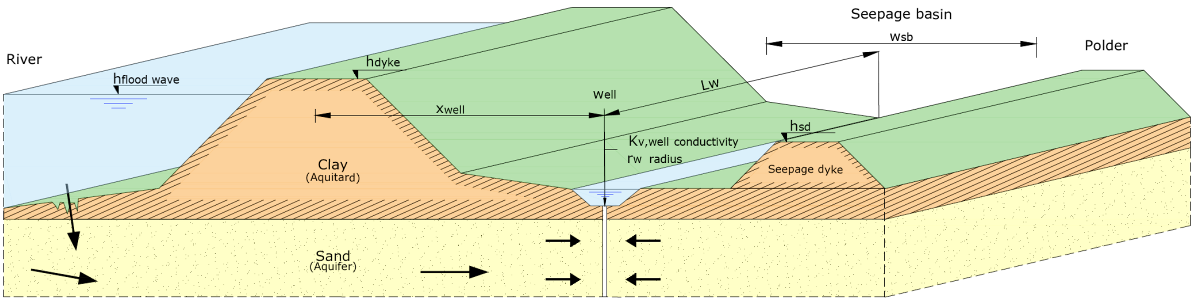


Figure 3.11: Summary Pipingontspanner variables

In the verification, alternatives are disregarded that do not satisfy the verification rules of Section 3.2.3. The reliability requirement needs the verification models of Section 3.1.2 to assess the Uplift and Heave mechanisms. An iterative loop is used to find the lowest values for Uplift and Heave in the results of the groundwater flow model. The plane between two wells is analysed for all time steps resulting in the governing reliability value for each alternative.

The feasible alternatives that pass the verification are passed on to the evaluation to determine the best alternative. The decision for the best alternative is determined with the trade-off function (Eq. 3.5).

3.4. FINAL REMARKS

The parametric model can describe the Pipingontspanner concept according to the design variables, verification rules and evaluation rules. The design approach is based on a parameter optimisation instead of explicit design rules and incorporates some preliminary design choices. The following important points are made concerning the parametric design:

- The drainage measure is part of the total flood defence and depends strongly on the current dyke system. The reliability analysis showed that the addition of drainage systems introduces some additional risk that needs to be taken into account when determining the required failure probability.
- The feasibility of the design depends among others on the two added boundary conditions. Firstly, the basin width (w_{sb}) is limited when objects and structures reduce the expansion of the basin. Secondly, the seepage discharge indicates if drainage measures are applicable at all for this dyke section.
- The Uplift and Heave verification models are adapted to the Pipingontspanner by taking the two Pipingontspanner reductions aspects into account. Sellmeijer's Piping assessment is used even thought it does not take the effects of drainage measures into account. Currently, there no other suitable options that integrate with the groundwater flow mode.
- The effects of the aquifers head drawdown on other geotechnical failure mechanisms need to be examined.
- A more detailed design is obtained by examining the Pipingontspanner effects and optimising/refining the design (next step in the design cycle).

4

Pipingontspanner evaluation

The applicability and effectiveness of the Pipingontspanner are evaluated in this chapter by analysing the Pipingontspannner model from Chapter 3, which answers research question 4 and 5. These questions focus on the technical conditions to implement the Pipingontspanner and the influence of the concept on the macro stability. The sections below describe a sensitivity and macro stability analysis to help understand and refine the concept.

4.1. Sensitivity analysis

The sensitivity of the Pipingontspanner model is analysed to establish the effectiveness of the piping mitigation measure under a variety of conditions. These conditions range from design variations to different soil compositions, which leads to a qualitative view of the applicability.

4.1.1. Parameters

From the available input parameters in the Pipingontspanner model, the design, hydraulic and soil elements are selected as the varying conditions in uncorrelated theoretical cases. These parameters and their chosen range are presented in Table 4.1. The sensitivity analysis is performed with the model described in Section 3.3 and the model parameters stated in Table 4.2.

Table 4.1: Parameters in sensitivity analyses

Type of data	Parameter		Range	Unit
Hydraulic elements	Flood wave rising front	F_{rf}	4-10	d
	Flood wave height	$h_{floodwave}$	3-6	m
Soil characteristics	Hydraulic conductivity aquifer	$K_{h,aquifer}$	5-20	m/d
	Hydraulic conductivity aquitard	$K_{v,aquitard}$	0.0001-0.01	m
	Specific storage coefficient	S_s	1E-5 - 1E-2	1/m
Design elements	Well spacing	L_w	5-20	m
	Basin width	w_{sb}	5-20	m
	Well position	x_{well}	7.5-22.5	m

The hydraulic load consist of two parts: the maximum water level and the shape of the flood wave. The maximum water level is the mean high water level plus the flood wave height, as depicted in Figure 4.1. The design focuses on the time-varying conditions of the flood wave, and thus the mean high water level remains constant. The flood wave height ($h_{floodwave}$) varies from 3 to 6 m.

The shape of the flood wave is determined by the duration of the rising front and falling back. A steeper front than back shows a diffusive wave and is suspected to influence the concept to a large extent. The overall duration of the flood wave is 20 days. If the rising front equals 10 days, the flood wave is symmetrical, and there is no diffusive character. If the rising front equals 5 days, the flood wave is asymmetrical, and the maximum water level occurs twice as fast. The duration of the rising front (F_{rf}) is varied from 5 to 10 days to examine the influence of the flood wave shape.

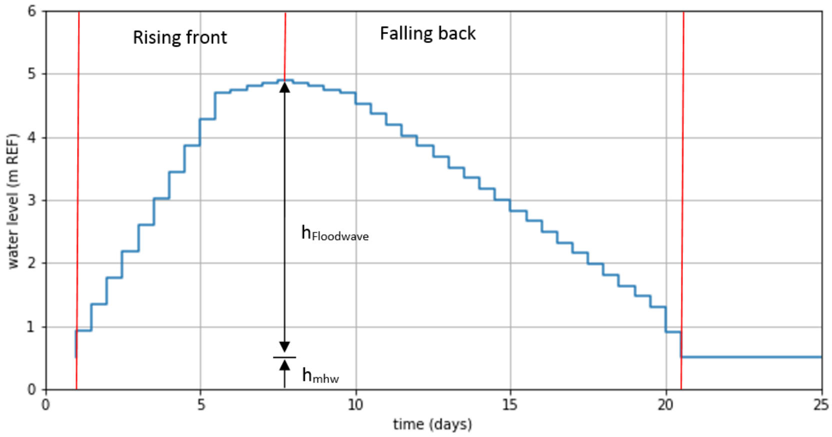


Figure 4.1: Flood wave parameters

The hydraulic aspects of the soil can be described by the transmissivity of the aquifer and the hydraulic resistance of the aquitard. Both of these soil parameters are influenced by the conductivity and thickness of the layer. For this analysis, the conductivity of the aquifer ($K_{h,aquifer}$) and the aquitard ($K_{v,aquitard}$) is chosen as the varying property. The aquitard hydraulic resistance influences the seepage through the cover layer and the aquifer conductivity influences the flow to the well system.

Another parameter that affects the well system is the specific storage factor (S_s), which determines how fast the water that is relieved from the aquifer as a result of drainage. A parameter range of 10^{-5} to 10^{-2} is used to examine the storage factor effect on the Pipingontspanner. The typical storage values in the Netherlands are $S_s = 2 \cdot 10^{-5} - 6 \cdot 10^{-4}$ (Niemeijer et al., 2017).

The design variables of the Pipingontspanner (Well spacing (L_w), Basin width (w_{sb})) are examined for their efficiency in reducing the occurrence of Uplift. Both variables have a parameter range of 5 to 20 m. The influence of the well position inside the basin is unknown and will be varied between the two dykes with a range of $x_{well} = 20 - 35$ m. In previous chapters, this position was centred between the main dyke and seepage dyke.

Table 4.2: Model parameters

Property	Layer type	Top [m NAP]	Bottom [m NAP]	Thickness [m]	K_h [m/d]	T [m^2/d]	K_v [m/d]	C [d]	S_s [1/m]	S_y [-]
Layer 2	aquitard	-1	-4	3	0.0005	0.0015	0.0005	6000	0.001	0.05
Layer 3	aquifer	-4	-9	5	10	50	3	1.67	0.0001	0.2
Other parameters:			Basin width:	15 m		Flood wave height:		4.9 m		
			Well spacing:	15 m		Flood wave rising front:		5 d		

4.1.2. RESULTS

The Pipingontspanner model processes each parameter series and from those results three characteristic elements of the well system are extracted: discharge well (Q_z), basin water level (h_{sb}) and aquifer head (h_{exit}). The effectiveness of the Pipingontspanner is obtained by assessing the Uplift mechanism with these characteristics. In the following sections, an overview of the well discharge, water levels and Factor of Safety (FOS) for Uplift is presented for each parameter.

HYDRAULIC ELEMENTS

The hydraulic elements control to which extent the Pipingontspanner is loaded and how fast. The combination of the flood wave height and shape will show when the Pipingontspanner performs better and how resilient the design is.

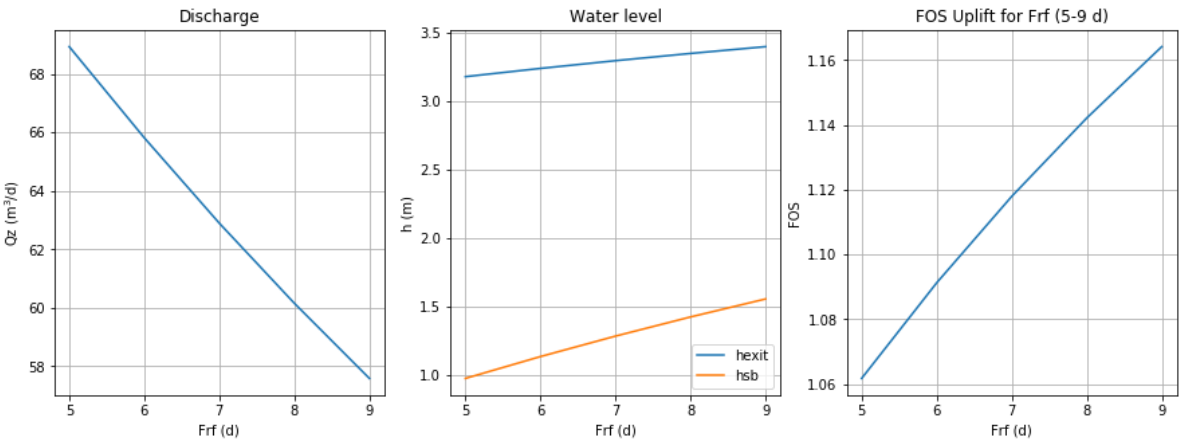


Figure 4.2: Sensitivity flood wave shape

The flood wave shape sensitivity is presented in Figure 4.2 and shows that the FOS increases when the front of the flood wave rises slower. A longer duration of the rising front allows more water to flow into the basin before the critical river water occurs. This results in higher basin water levels and lower discharges, which contribute to the Uplift FOS.

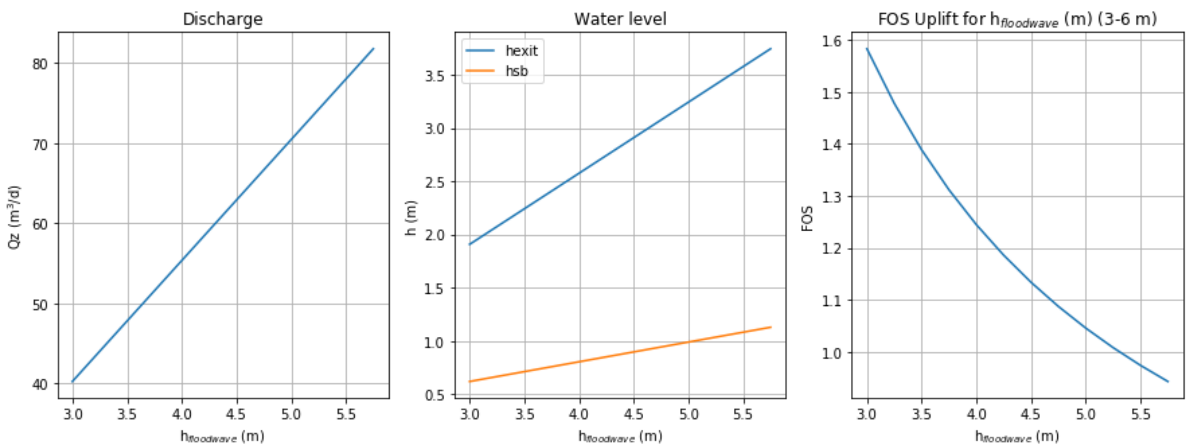


Figure 4.3: Sensitivity flood wave height

The height of the flood wave directly influences the head in the aquifer below the cover layer. A higher river level increases the pressure on the cover layer and reduces the Uplift safety factor. Figure 4.3 shows that the Pipingontspanner cannot keep up with the increasing water level and results in a reducing factor of safety with an exponential curve.

SOIL CHARACTERISTICS

The soil characteristics determine how fast the water can flow towards the well and consequently influences the well discharge. A lower well discharge results in a lower seepage basin water level and head reduction. Moreover, the aquifer head develops at a slower rate due to the lower conductivity of the soil.

The sensitivity for the aquifers conductivity is depicted in Figure 4.4 and shows that the factor of safety increases when the aquifer becomes more permeable. A higher conductivity increases the flow towards the well and thus the discharge and basin water level increase exponentially. The head should reduce when the discharge increases, but a more permeable soil develops the head faster and higher. Overall, the permeability has a significant effect on the Pipingontspanner.

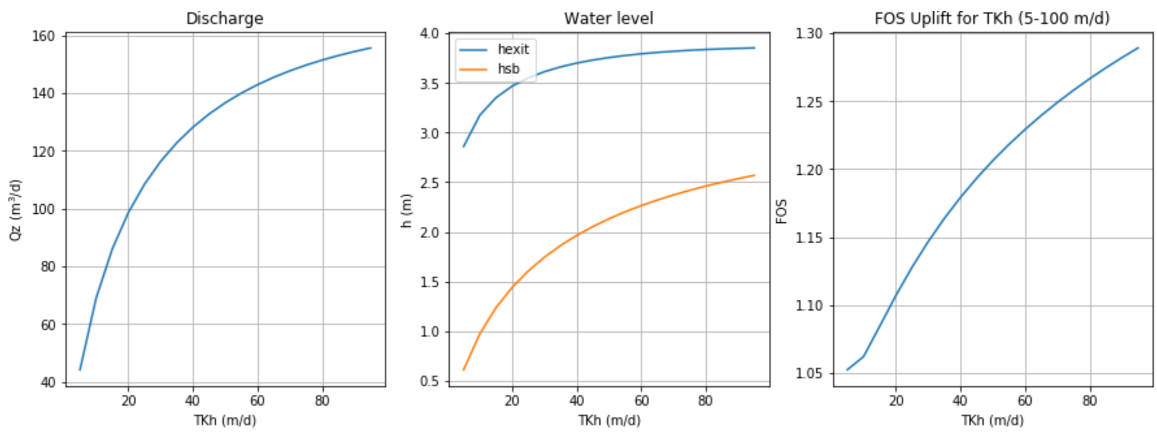


Figure 4.4: Sensitivity transmissivity aquifer

The cover layer conductivity is presented in Figure 4.5 and displays a slight improvement in the factor of safety. A high permeability increases the seepage through the cover layer into the basin, which decreases the discharge and basin water level. The head decreases when the seepage through the cover layer increases. The overall variability has a limited effect on the Uplift mechanism.

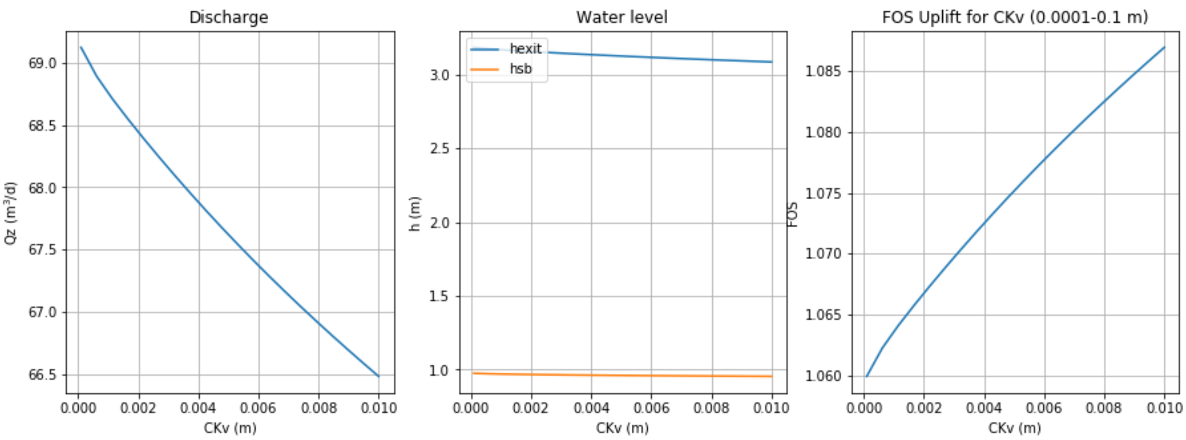


Figure 4.5: Sensitivity hydraulic resistance aquitard

The specific storage factor is represented by equation 3.1 and consists of the hydraulic conductivity and the soil consolidation factor. A lower S_s value reduces the discharge and correspondingly the basin water level and head reduction. Figure 4.6 shows that the discharge is mildly influenced for the specific storage factor range that occurs in the Netherlands $(2 \cdot 10^{-5} - 6 \cdot 10^{-4})$.

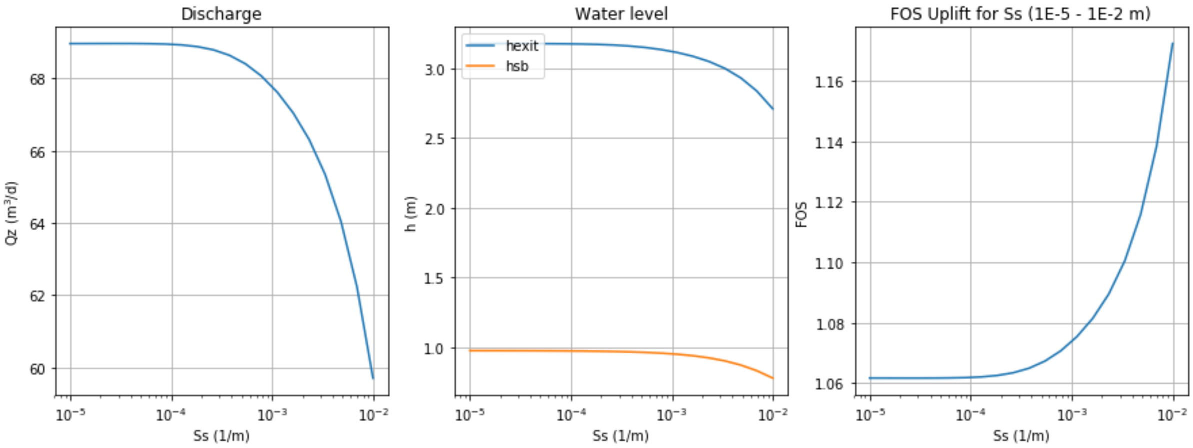


Figure 4.6: Sensitivity specific storage factor

DESIGN ELEMENTS

The combination of the seepage basin width (w_{sb}) and well spacing (L_w) shows how the design can be configured to create a more effective solution. The result of the two combined parameters for Uplift is shown in Figure 4.7.

The colour map shows the most efficient combination, which is in the lower left corner at w_{sb} = 5 m and L_w = 5 m. At first instance, the colour map pattern looks diagonally symmetrical. However, the pattern is not fully symmetrical as one can see from the colour map cross-sections in Figure 4.7.

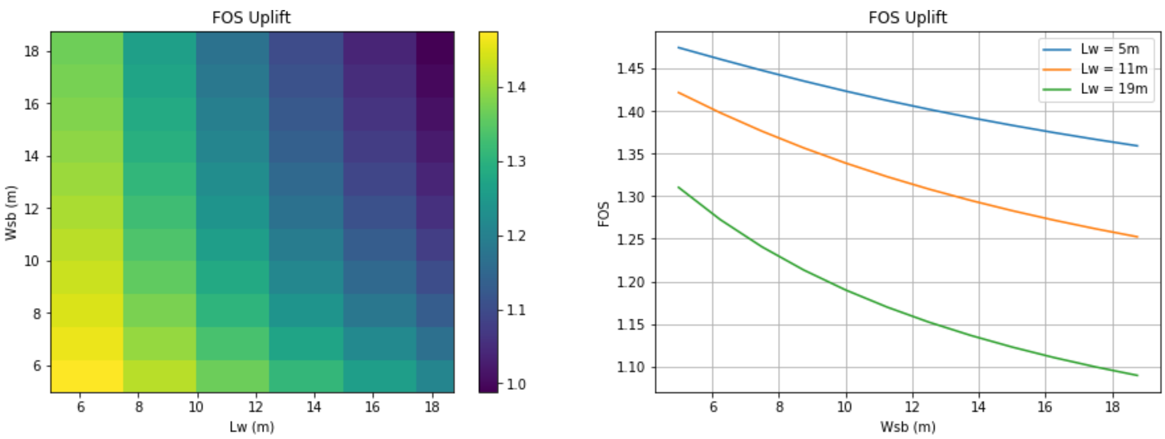


Figure 4.7: Sensitivity variables: w_{sb}&L_w colour map

A closer look at the characteristics of the Pipingontspanner model (Figure 4.8) shows that both the discharge and seepage basin water level are symmetrical, however, the aquifer head shows a deviating behaviour. The factor of safety for Uplift consists of the cover layer thickness, seepage basin water level and aquifer head. Since the thickness is constant, the asymmetrical result of the colour map is directly related to the head in the aquifer. It seems that the combination of a large well spacing with a large width causes an unexpected behaviour by deviating from the pattern shown in the discharge and basin water level.

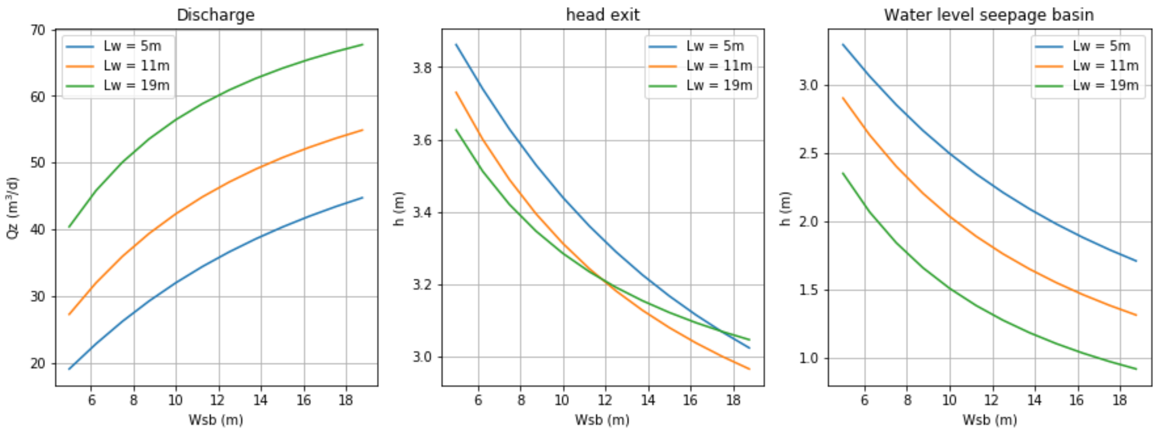


Figure 4.8: Sensitivity variables: w_{sb} & L_w

A second observation is that the Uplift factor of safety, which depends on both the head reduction and basin water level, is mainly steered by the basin water level. The maximum head reduction is 1.8 m, and the maximum basin level is 3.25 m. The combination with the highest factor of safety produces very high basin water levels (3+ m), which is not desirable as it also requires a high seepage dyke. A potential water level limit in basin must be considered, but also reduces the effectiveness of the Pipingontspanner.

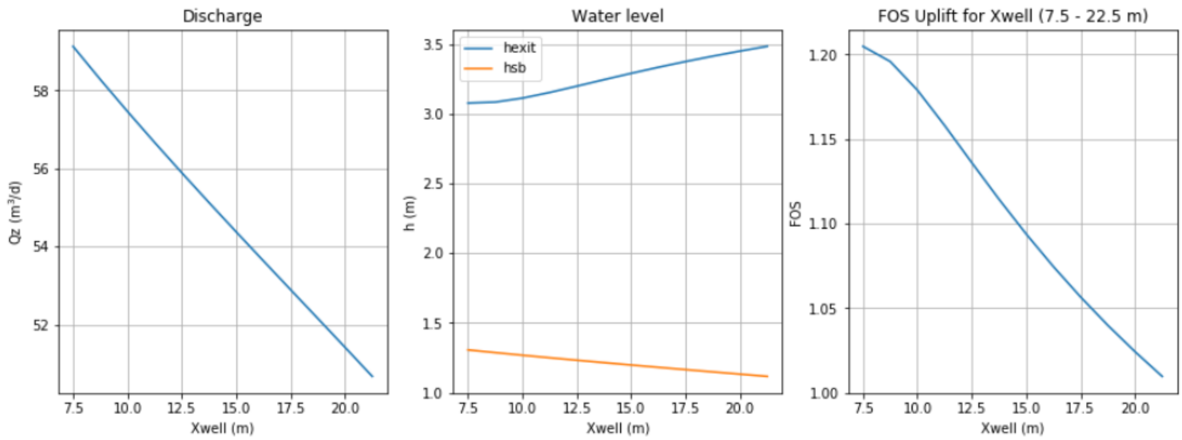


Figure 4.9: Sensitivity variable: x_{well}

The well position was assumed to be in the middle of the basin in the previous chapters. The results of the well position variation (Figure 4.9) showed a negative trend in efficiency for an increasing well distance (x_{well}). The discharge decreases from 60 to 50 m^3/s and correspondingly the basin water level (h_{sb}) decreases and aquifer head increases (h_{exit}). The well position causes a linear decrease in safety when moving away from the river dyke.

Ultimately, the Pipingontspanner is more effective when the well is positioned closer to the main dyke toe. The potential Uplift point is often near the dyke toe and thus positioning the well near this point produces the highest safety factor.

4.1.3. PARAMETER INFLUENCE

The goal of the sensitivity analysis is to provide insight on the effectiveness and applicability of the design. The results from the previous section display the performance of the Pipingontspanner design under various conditions, from which conclusion are drawn regarding the applicability of the piping measure in this section.

EFFECTIVENESS

The design variables showed that the highest safety factors occur for small well spacing's and a limited basin width.. The basin area for those cases is small, which results in high basin water levels that contribute to the high safety factors. Basin water levels that approach the river level are unwanted as it would require a seepage dyke with the proportions of the main dyke. The extreme basin water levels are filtered from the suitable alternatives by suggesting to neglect the cases with a basin water level higher than 75% of the river-polder water level difference.

In general, a larger basin width would be preferred over a high basin water level as this could introduce additional geotechnical risks such as piping and macro instability of the seepage dyke. These conclusions are depicted in the results of the design variables in Figure 4.10. Additionally, the positioning of the well near the toe of the dyke proved to be the most favourable option and is incorporated into the Pipingontspanner model.

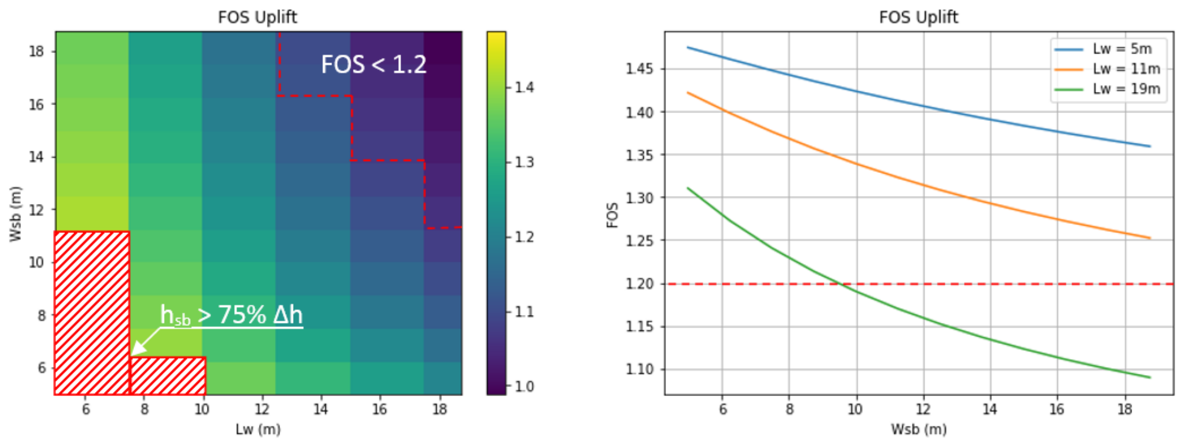


Figure 4.10: Operating conditions: w_{sb} & L_w

OPERATING CONDITIONS

The hydraulic and soil conditions are essential for the applicability of the measure, and an indication of their relative influence can be derived from the sensitivity analysis. The following conclusions are drawn for the hydraulic and soil aspects:

- Hydraulic elements:
  - A symmetrical flood wave shape has a slightly beneficial influence on the efficiency of the POS design. The steeper front of the asymmetrical flood wave reduces the Uplift safety up to only 10%. When the river level slowly rises, there is more time to fill up the basin before the critical river level is reached and that increases the factor of safety for Uplift. The change in the shape of the flood wave does not significantly affect the system of the Pipingontspanner.
  - An increase in the flood wave height (naturally) has a negative influence on the safety. The sensitivity analysis shows that the head in the aquifer increases twice as fast as the water level in the basin. The relation between the factor of safety and the flood wave

height is slightly exponential and reduces the safety about 20% per meter.

- Soil characteristics:
  - A higher permeability of the aquifer increases the safety by 7% per increase of 20 m/d. At values equal to or higher than 80 m/d, the head in the aquifer reaches an almost constant level. Overall, the Pipingontspanner concept works better with more permeable aquifers.
  - The permeability of the cover layer has a small influence on the well discharge and water levels. A high permeability increases the natural seepage to the basin and a low permeability blocks seepage, which allows the head to rise higher and faster. Overall, the cover layer permeability influence on the POS concept is negligible as the discharge only decreases slightly.
  - The specific storage factor has an influence on the POS performance for values higher than S_s = 1E-3 1/m but is negligible for lower values. The typical S_s values occurring in the Netherlands are below this threshold, thus the storage factor influence is limited.

The Pipingontspanner works well when the aquifer is permeable and the hydraulic load increases slowly. It is mostly affected by parameters that describe the flow system towards the well, which is characterised by:

- The transmissivity (thickness x permeability) of the aquifer;
- The water supply from the river bottom.

The water supply to the aquifer is not included in the sensitivity analysis, because the Pipingontspanner model is based on situation in which the river is in direct contact with the aquifer. This situation occurs when scour holes or thin bottom layers are present and is considered conservative. A delay in water supply decreases the development of pressure and improves the Uplift factor of safety.

4.2. MACRO STABILITY ANALYSIS

The Pipingontspanner affects the phreatic line in the dyke body and hinterland with the introduction of the well and the basin. Consequently, the macro stability of the dyke changes and affects the failure probability of the flood defence. This section examines what kind of effect the Pipingontspanner has on the macro stability of the main dyke. The focus is on the comparison of two scenario's and not specifically on the final results of the slope stability.

The failure mechanism of inward macro stability assesses the sliding of soil parts towards the hinterland. When the dyke body is infiltrated with water, the pore pressures increase and could lead to a failure of the inner slope, as a result of a decrease in the effective stress and shear resistance.

4.2.1. SETUP

The slope stability analysis is performed with Limit Equilibrium Models (LEM) in the program D-Geo Stability (version 18.1). The method of Bishop and Uplift- Van are used to calculate the stability safety factor. Bishop and Uplift - Van both check the moment and vertical equilibrium of the slip surface. Additionally, Uplift - Van checks the horizontal equilibrium. The sliding surface of bishop is circular in contrast to Uplift - Van which has a horizontal element between the circular elements. Since the scenario is piping sensitive, the Uplift-Van method is expected to provide the critical slip surface. The LEM approach has disadvantages compared to numerical approaches, but would be sufficient enough to show the effects of the Pipingontspanner on macro stability.

The generic case, described in Section 2.3.1, is used in the slope stability analysis with the corresponding geometry and soil conditions. The aquifer head and phreatic surface are obtained from the groundwater flow calculation results presented in Section 2.3.4. The aquifer head used in this calculation exceeds the potential limit and thus represents a situation that would not occur in practise. There are two different scenarios simulated in this analysis:

1. **Base scenario** A high river level with a fully saturated dyke body and the phreatic surface equal to ground level in the hinterland. The aquifer head is acquired from the base groundwater flow model (Figure 2.11).
2. **Pipingontspanner scenario** A high river level with a fully saturated dyke body and the phreatic surface according to the groundwater flow model results for the Pipingontspanner (Figure 2.14). The phreatic and aquifer head is obtained for the critical point of piping (t = 9 days).

The soil properties are provided in Table 4.3. and represent generic values for the different soil types. The phreatic surface in the dyke body is a generic line from the assessment level to the polder level and represents a fully saturated dyke body. A surcharge of 10 kN/m is applied on the dyke crest and represent traffic load and maintenance works. An overview of the scenario setup is depicted in Figure 4.11 and 4.12.

Table 4.3: Soil properties D-Geo stability

Layer	Type	Top [m NAP]	Thickness [m]	Soil properties γ_{dry} [kN/m^3]	γ_{sat} [kN/m^3]	c'_d [kN/m^2]	ϕ'_d [°]
2,3,4,5	Soft clay	5	6	14	14	1	25
1	Sand	-4	5	17	19	0	30

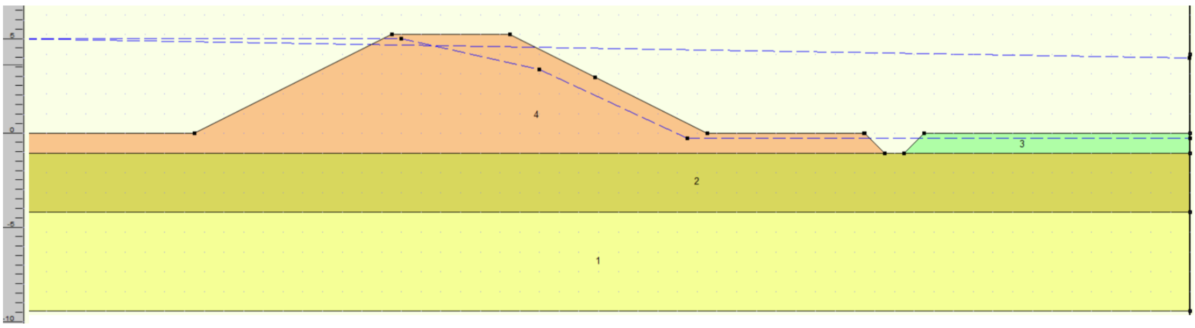


Figure 4.11: Input D-Geo Stability Base scenario

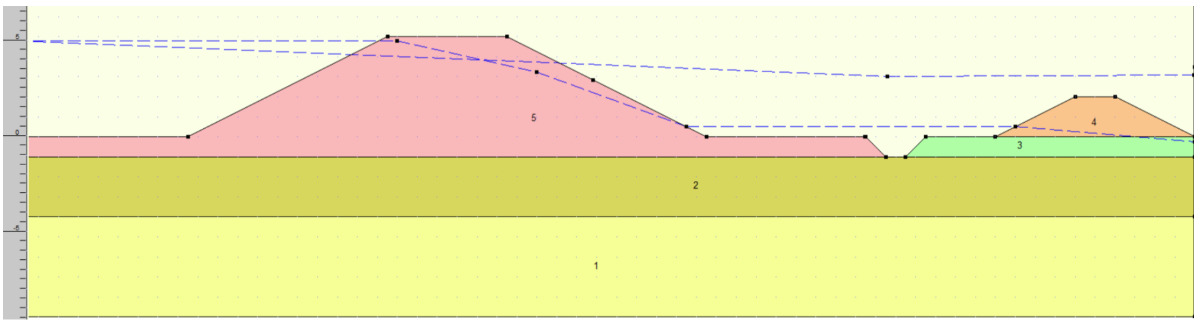


Figure 4.12: Input D-Geo Stability Pipingontspanner scenario

4.2.2. RESULTS

The safety factor for the critical slip surfaces of the Bishop and Uplift - van method are presented in Table 4.4 for both scenario's. The critical slip circle for scenario 1 and 2 are displayed in Figures 4.13 and 4.14, respectively. The lowest safety factor is found for the base scenario with FOS = 0.39. The gradient of the inner slopes are 1:2 and therefore increase the slope instability in contrast to more moderate dyke gradients 1:3 - 1:4. The results show that the safety factor for scenario 2 (Pipingontspanner) improves significantly compared to the base scenario.

Table 4.4: Macro stability results

Scenario	Factor of safety (-)		Radius (m)	
	Bishop	Uplift - Van	Bishop	Uplift - Van
1. Base	0.45	0.34	14.2	19.62
2. Pipingontspanner	0.62	0.53	15.29	15.4

The Pipingontspanner measure has a positive effect on the macro stability and increases the factor of safety up to 35%. When the basin water level of scenario 2 rises to 1 m instead 0.5, the Uplift - van safety factor increases to 0.63. The measure is not designed to prevent macro stability but the configuration can take this aspect into account by providing higher basin water levels that reduce instability.

A similar effect occurs for the measure Piping berm, which acts as a support berm and improves the stability. Additionally, Piping berms can easily expanded to create a sufficient counter pressure for stability. This is not the case for the Pipingontspanner, as small changes in the design would affect the performance for piping substantially, as is shown in the previous section.

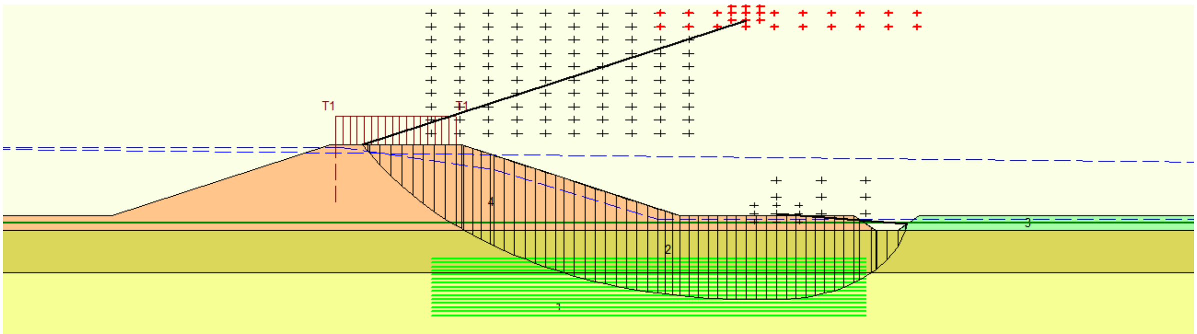


Figure 4.13: Result stability Uplift - Van Base scenario

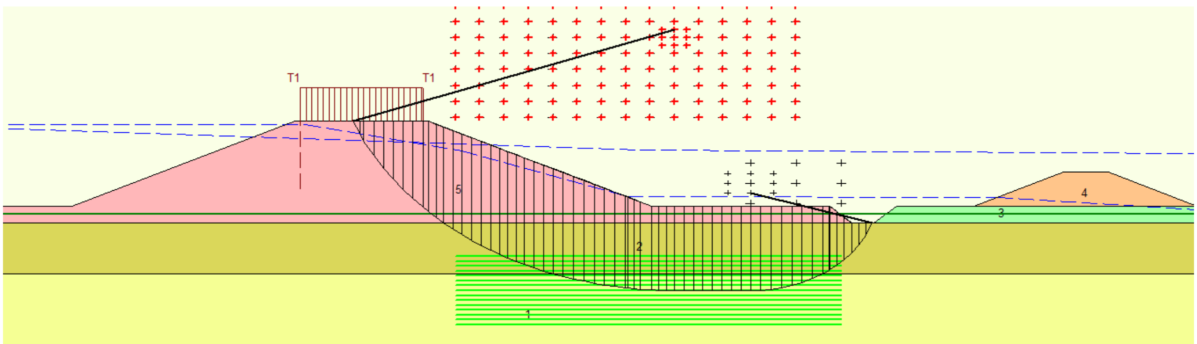


Figure 4.14: Result stability Uplift - Van Pipingontspanner scenario

4.3. FINAL REMARKS

The sensitivity and macro stability analysis showed the applicability and performance of the Pipingontspanner model and helped to understand and refine the concept. The Pipingontspanner works well if the aquifer is permeable and the hydraulic load increases slowly. The cover layer permeability and storage coefficient do not affect the performance of the Pipingontspanner. The concept is mostly affected by parameters that describe the flow system towards the well (transmissivity and riverbed resistance). The following points could be deducted regarding the configuration of the Pipingontspanner:

- A well near the inner dyke toe is the most effective position regarding the Uplift safety.
- High basin water levels ($h_{sb} > 75\%\Delta H$) are found for small values of well spacing and basin width ($L_w, w_{sb} < 10$ m), which could cause a secondary piping mechanism and should be avoided. Additionally, this basin level would almost require a seepage dyke size similar to the main dyke.
- The sensitive parameters have a considerable influence on the Pipingontspanner model and would pass on their uncertainty to the model, which creates less reliable results.

Lastly, the Pipingontspanner contributed to improving macro stability by providing additional counter pressure on the cover layer. Additionally, the well relieves the upward aquifer pressure, which creates additional resistance for macro instability. However, the measure is not easily expandable for more resistance against slope instability as these changes could affect the piping prevention capability.

5

Measure cost and application

The economic feasibility of the Pipingontspanner concept is demonstrated in this chapter by first defining the cost of the measure and secondly comparing these costs with a traditional piping measure in a case environment. The result shows the economic aspects of the Pipingontspanner and by this means answering sub-question 6.

5.1. General costs comparison

In current practices, there are many solutions to mitigate piping problems found in dyke segments throughout the Netherlands. Each type of measure has unique benefits that could match the requirements of a problematic dyke segment and forms a suitable solution. A decision among the different types of measures is not only based on the requirements and benefits but depends, for a large part, on the costs of the solution. These costs reflect the economic feasibility of a measure and illustrate whether the measure poses a notable alternative. Typically, a cost-benefit comparison is used to determine the best solution among the considered alternatives.

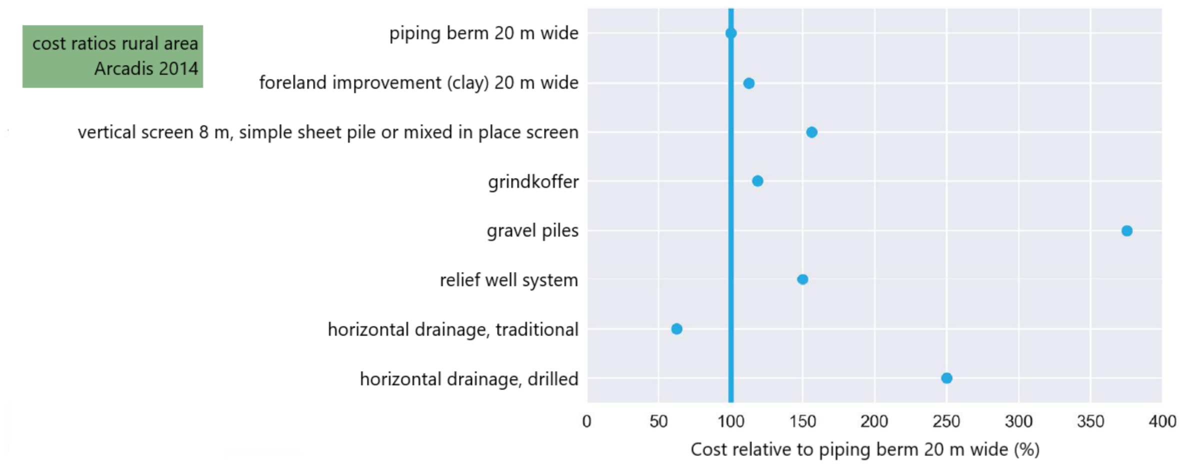


Figure 5.1: Cost ratios based on SSK estimates of realised projects, translated (Arends and Niemeijer, 2014)

From all the types of piping mitigation measures in the Netherlands, the drainage measures are relatively new and not widely applied yet (Luijendijk et al., 2017, Niemeijer et al., 2017). Figure 5.1 shows how the costs of drainage measures compare to the traditional Piping berm measure (depicted as 100%) for a rural area. The indicated measures are expressed relatively to the piping berm cost and based on a estimate of the construction costs without any land acquisition. The gravel piles method is based on a project that involved more than flood protection safety, which results in a higher cost price than the other drainage measures.

The construction costs of drainage measures appear to be higher than the piping berm measure in Figure 5.1. Moreover, the maintenance and monitoring costs are not included in the study of the drainage techniques (Arends and Niemeijer, 2014), which are generally higher for drainage measures. There are still inaccuracies in the design method and reliability aspects of drainage measures (Niemeijer et al., 2017), which results in a conservative design with a higher construction price. Alternatively, the price differences could be explained by variations in location-specific aspects that drive up the costs for drainage measures.

The cost definition of the Pipingontspanner combined with a case study is a tool to determine the economic feasibility since it allows the comparison with an alternative measure on the same conditions. The costs function developed in the following section is used in the case study (Section 5.3) to compare with the traditional Piping berm method in a specific case.

5.2. COST ANALYSIS PIPINGONTSPANNER

The Pipingontspanner costs are determined for a general design that can be adapted to different scenarios. A specific design would allow for a more accurate comparison with alternatives but would not show how the costs adapt to different circumstances. The type of costs used in this analysis is Life Cycle Cost, where the operation and maintenance phases are expected to form a considerable share of the total costs. The costs will be represented by a generic cost function that is developed for the required design lifetime using the Net Present Value approach.

GENERAL DESIGN

The Pipingontspanner design elements needs to be detailed before the structure costs can be estimated. The main layout of the design and the design variables are defined in Chapter 3, but the design of the individual elements are left unspecified. Figure 5.2 shows the layout of the design and the two main elements that require further refinement. Appendix B describes these elements and provides the design specifications and unit prices for each element.

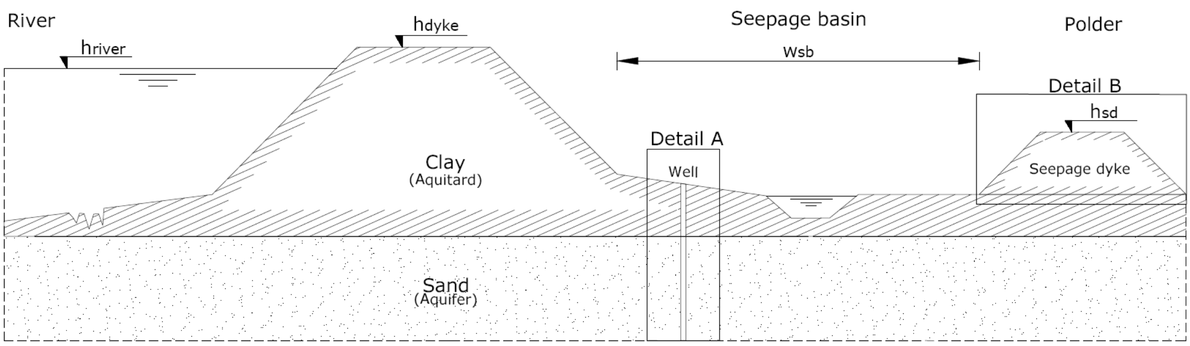


Figure 5.2: Pipingontspanner general design

The first element is the well (Detail A, Figure 5.3) and generally consists of a screen in the aquifer that is surrounded by a filter (USACE, 1992). The well screen and filter allow water to flow towards the surface and prevent the loss of foundation material. The connection of the screen to surface level is called riser and has a backfill that blocks the flow alongside the pipe. The well is closed off with a wellhead and equipped with valves for maintenance and emergency scenario's. A suction drill technique is used to place the filter and well structure.

The inspection, monitoring and maintenance aspects of a relief well are an essential part of the design and ensure the proper functioning of the well. A well with a reduced efficiency could lead to a structural failure during high water events. For example, the clogging of the well screen could result in a lower well efficiency and increases the surrounding hydraulic head. The maintenance aspects are coupled to the design of the well and seepage dyke, and thus a maintenance strategy is provided in Appendix B with the corresponding costs.

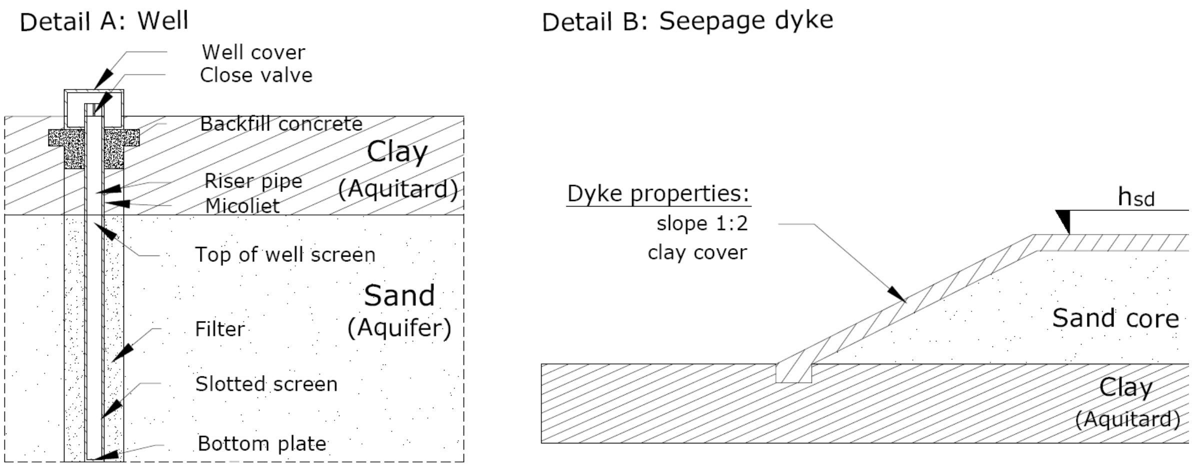


Figure 5.3: Pipingontspanner general design details

The second element is the seepage dyke (Detail B, Figure 5.3), which retains the water inside the basin by forming an impermeable wall. This wall is formed as a clay dyke with a core of sand according to the guidelines for new dykes (Jonkman et al., 2017). The slope on either side has a gradient of 1:2 and the dyke exterior is covered with vegetation.

GENERIC COST FUNCTION

The generic cost function describes the Life Cycle Costs based on the general design and the corresponding unit prices for each of the elements. The function is implemented in the Pipingontspanner model to evaluate the cost of each alternative as discussed in Section 3.2.3).

The required lifetime for structures near a primary flood defence is generally 100 years (Jonkman et al., 2017), but the Pipingontspanner only has a design lifetime of 50 years. The limited lifetime means that the structure will be entirely renewed around 50 years. The Net Present Value method is used to incorporate a second life cycle into the cost function and is given by the formula:

$$NPV = \sum_{n=1}^{T} \frac{C_t}{(1+r)^{t_i}} - C_0 \tag{5.1}$$

- T Total life cycle [years]
- t_i Number of interest periods [-]
- r Net discount rate (0.02) [-]
- C_t Net cash flow during period t_i [€]
- C_0 Initial investment [€]

The total costs (TC) of the Pipingontspanner for a 100 year lifetime are implemented with the NPV according to equation 5.2. The general shape of the Life Cycle function is given by equation 5.3, but differs between two the intervals resulting in two separate functions with LCC1 and LCC2 the costs during the first and second life cycle respectively. The second life cycle re-uses elements of the first life cycle that have a life expectancy of more than 50 years. The well will be completely renewed after 50 years, but the seepage dyke has with some maintenance has a life expectancy 100 years or more. Moreover, the design phase can re-use elements from the first design interval such as measurements.

$$TC = \frac{LCC_1}{(1+r)^1} + \frac{LCC_2}{(1+r)^{50}} \tag{5.2}$$

$$LCC_n = C_{design} + C_{construction} + C_{maintenance} + C_{removal} \tag{5.3}$$

The Life Cycle function elements are presented in Table 5.1 and show how the design, construction, maintenance and removal costs come together. The specific construction and maintenance cost functions are described in Appendix B and based on the design variables (h_{sd}, L_w and w_{sd}).

Table 5.1: Life Cycle Cost function Pipingontspanner

Phase	Part	Function
Construction (Cc)	Seepage dyke	C_{cSd}
	Well	C_{cW}
	Seepage basin	C_{cSb}
	Project supervision	$C_{cPm} = 15\% \cdot (C_{cSd} + C_{cW} + C_{cSb})$
	Construction cost	$C_c = C_{cSd} + C_{cW} + C_{cSb} + C_{cPm}$
Maintenance (Cm)	Seepage dyke	C_{mSd}
	Well	C_{mW}
	Maintenance cost	$C_m = C_{msd} + C_{mW}$
Design (Cd)	Well, basin and dyke	$C_d = 20\% Cc$ (Construction Costs)
Remove (Cr)	Well, basin and dyke	$C_r = 10\% cc$ (Construction Costs)
Life Cycle Costs	Unforeseen	$Cu = 10\%(C_d + C_c + C_m + C_r)$
	LCC period 1	$LCC1 = 1.1 \cdot (C_d + C_c + C_m + 0.5 \cdot C_r)$
	LCC period 2	$LCC2 = 1.1 \cdot (0.75 \cdot C_d + C_c + C_m + C_r)$
	Total costs (TC)	$TC = LCC1/1 + LCC2/(1.02)^{50}$

The construction costs include a 15% addition for the preparation, management and supervision of the project. The design and removal costs are hard to determine precisely for a global design, and thus are based on the construction costs. The design costs are composed of measurements, design and conditioning aspects, which are estimated to be roughly 20% of the construction costs. The removal of the structure at the end of the lifetime is estimated to be 10% of the construction costs. Finally, unforeseen costs are accounted for into the function.

COSTS SIMULATIONS

The LCC1 cost function of the previous section is used to analyse the influence of the design variables and shows how each element contributes to the overall cost. A composition of the Life Cycle Costs is displayed in the Pie chart below (Figure 5.4) for a well spacing and basin width of 11.25 m. The maintenance cost contributes to 51.2% of the total costs, which is about 1.6 times the construction costs. The composition of the construction costs shows that the seepage dyke is one of the primary cost drivers with a share of 45.8%. Furthermore, the well and seepage basin combined equal the cost of the seepage dyke with a share of 41.1%.

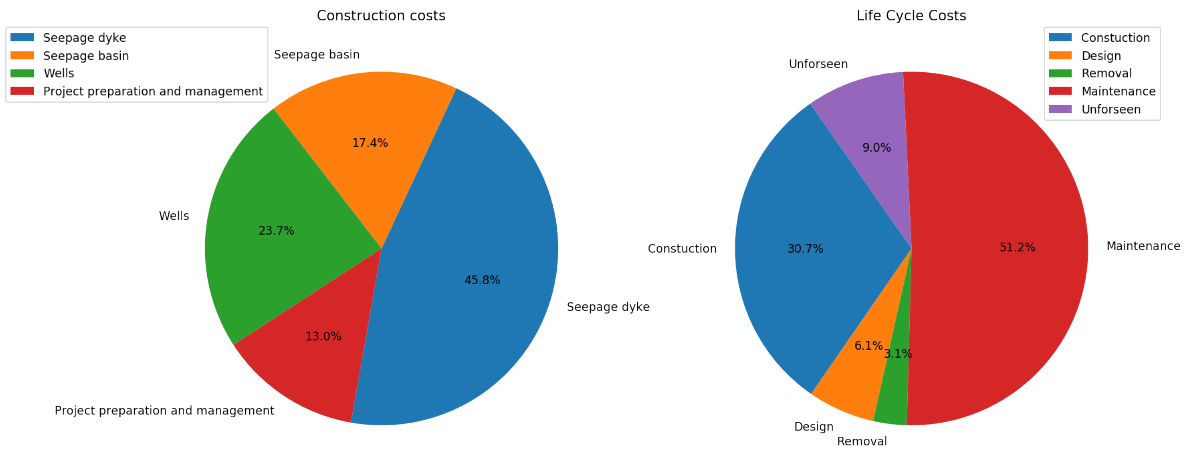


Figure 5.4: Life Cycle Cost composition

The leading variables in the cost function are the seepage dyke height (h_{sd}), well spacing (L_w) and basin width (w_{sb}) and those are used to examine the financial response to changes in the design. The colour map below shows the predicted Life Cycle Cost for different combinations of the basin width and well spacing. The seepage dyke height is maintained at 2 meters during the cost simulations.

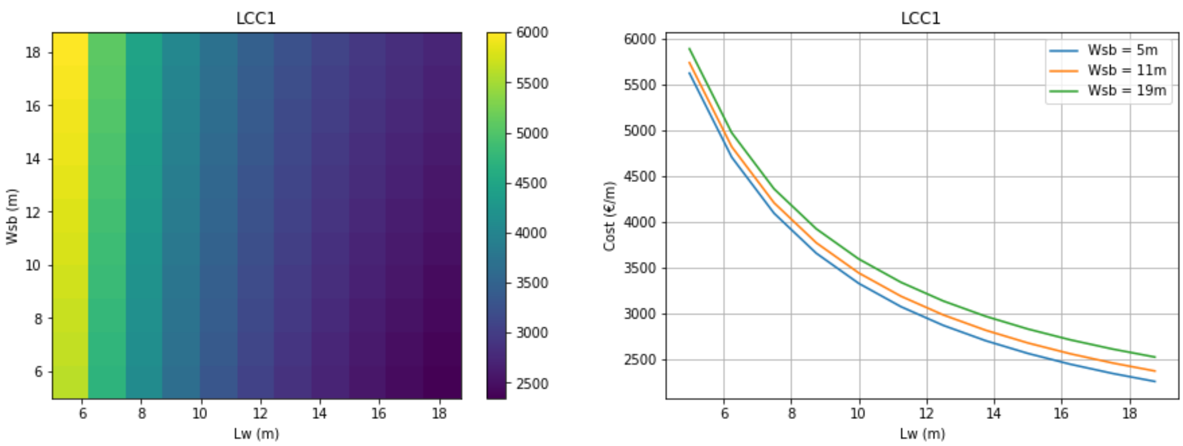


Figure 5.5: Life Cycle Costs for varying design variables (L_w and w_{sb})

The well spacing appears to have the most substantial influence on the overall costs, because of the maintenance and well costs. Both are directly connected to the number of wells, i.e. well spacing, and follow an exponential cost curve. Figure 5.6 depicts the cost of the two aspects for a varying well spacing and shows that the maintenance costs are the main cost driver with a contribution to the Life Cycle Costs ranging from 40 to 60%.

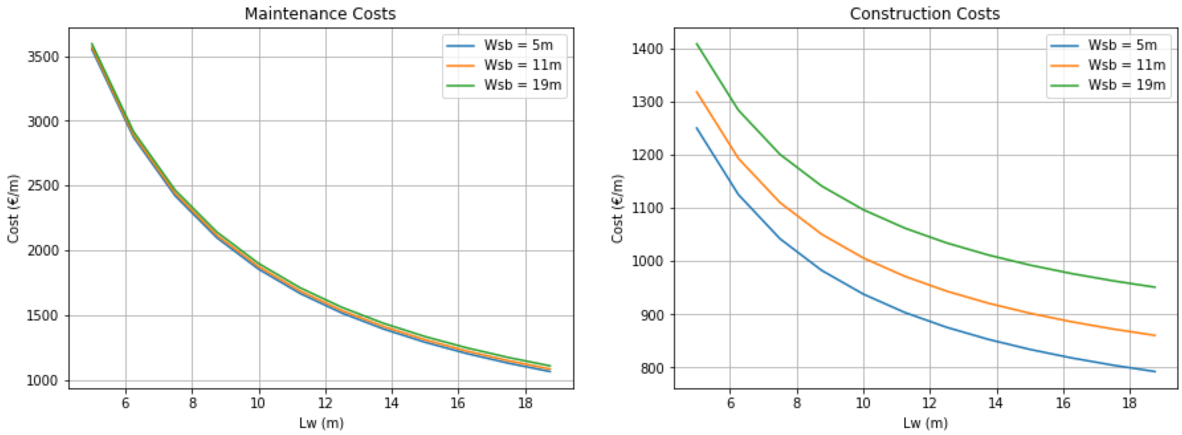


Figure 5.6: Maintenance and construction costs for varying design variables (L_w and w_{sb})

The Life Cycle Costs pie chart showed that construction costs of the seepage dyke was the second largest contribution to the overall costs and could provide an interesting option for optimisation. Figure 5.7 displays the total Life Cycle costs and the costs of the seepage dyke under a varying height (h_{sd}). The seepage dyke height has a significant influence on the costs and is worth to optimise.

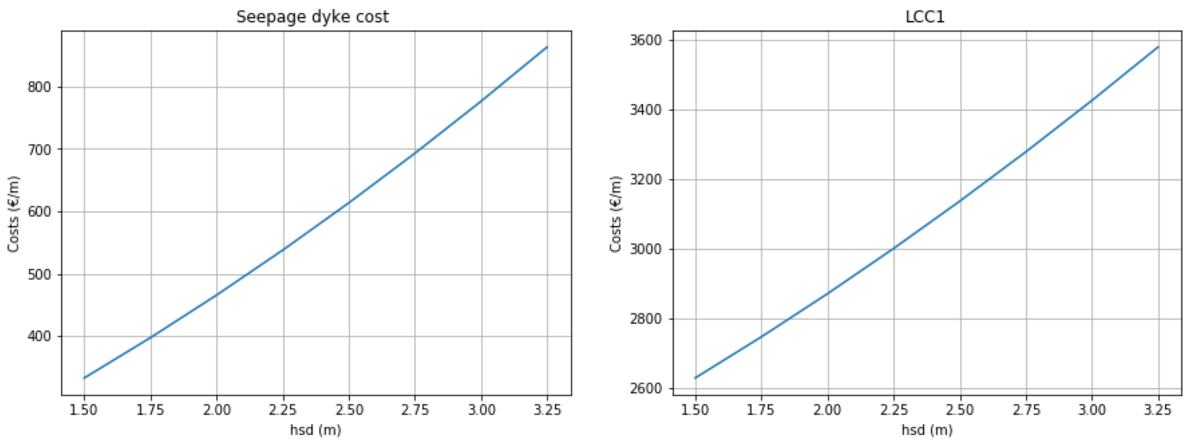


Figure 5.7: Seepage dyke costs for varying dyke height (h_{sd})

The Pipingontspanner costs cannot directly be compared to the measures displayed in Figure 5.1 as those only display construction costs and are based on specific projects. The Pipingontspanner is estimated to be positioned around the relief system at 150% of a piping berm, which is based on a comparison of the design elements. The Pipingontspanner has a straightforward well design that contains fewer elements than a relief system but does include an expensive seepage dyke, which takes up about 50% of the construction costs and 20% of the overall costs. The seepage dyke and maintenance costs are the largest contributors in the design and provide opportunities for improvement.

5.3. CASE STUDY

A case study provides an opportunity to compare the Pipingontspanner with a traditional Piping berm mitigation measure. The piping berm is often used to compare against, as it is universally applicable, reliable and only requires sufficient space behind the dyke. In the following sections, the case of Tiel is described, a preliminary design of the Pipingontspanner is determined and a comparison with the Piping berm is made.

5.3.1. CASE TIEL

The case concerns a dyke segment near the city of Tiel with a localised Piping problem. Currently, the flood defences of the city Tiel do not satisfy the new requirements stated in the Water Act 2017 and a dyke reinforcement project is ongoing (Waterschap Rivierenland, 2017). The total reinforcement project has a length of 3.6 km and is projected to start construction in 2021. The objective in this case study is to solve the piping problems for a green dyke in the dyke segment TG000 - TG003, which is depicted in Figure 5.8 and has a length of 326 m.

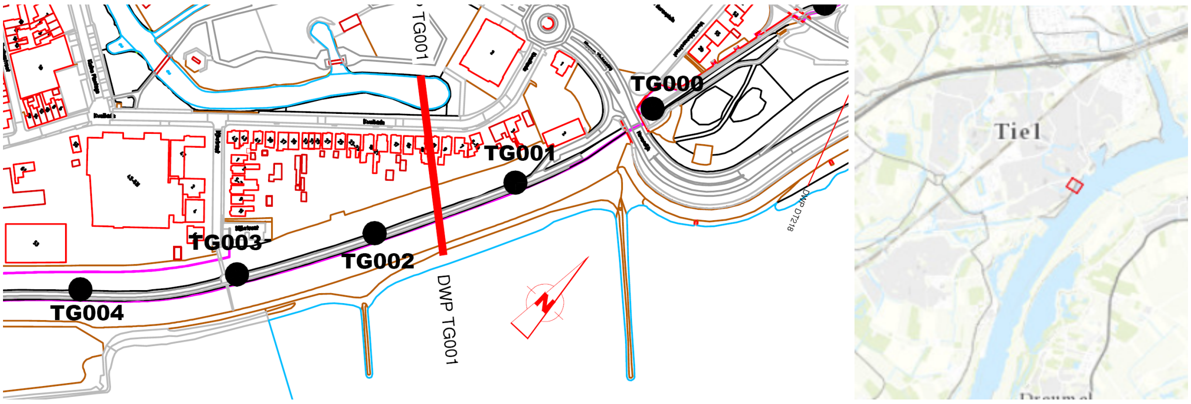


Figure 5.8: Overview case location Tiel (Waterschap Rivierenland, 2018)

The geometry, subsoil and hydraulic data were provided by the Waterboard Rivierenland and consist of cross-sections, cone penetration tests (CPT) and monitoring wells. The soil permeability is not available and is deducted from monitoring wells or soil type. The soil data is presented as a geotechnical cross-section between TG001 and TG002 in Figure 5.9 with the corresponding description in Table 5.2.

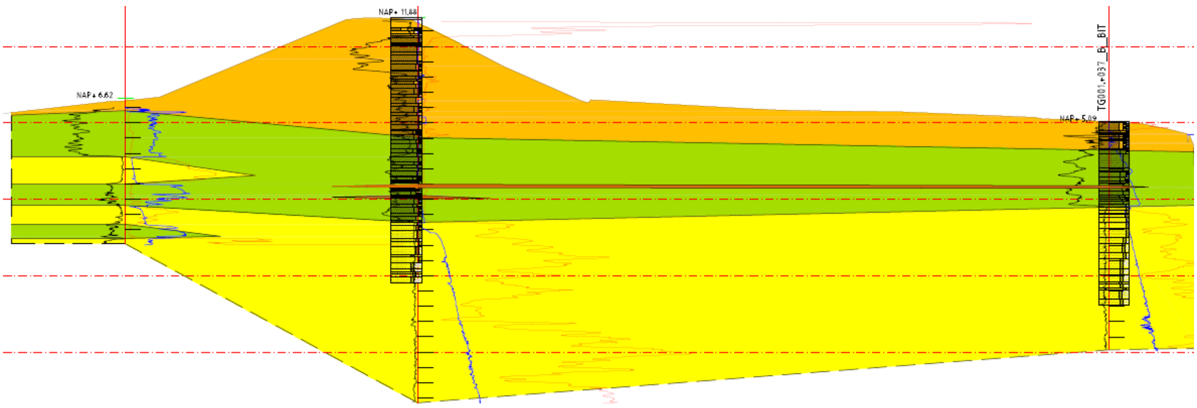


Figure 5.9: Geotechnical cross-section Tiel TG001 (Waterschap Rivierenland, 2018)

Table 5.2: Soil characteristics cross-section TG001 (Waterschap Rivierenland, 2018)

Number	Colour	Type	Consistency description (w: weak; m: mild; s:strong)	Soil properties γ_{dry} [kN/m^3]	γ_{sat} [kN/m^3]
Soil 1	Orange	Sand/Clay cover	Silty: w-m; locally contains debris	17	20
Soil 2	Green	Clay	Silty: w-s	13	19
			Sandy: w-m	17	20
Soil 3	Yellow	Pleistoceen sand	Silty: w-s; coarseness: w-s	18	20
Soil 4	Red	Peat	Clayey: w-s	12.5	13.5

The dyke profile starts a few meter before the outer dyke toe at NAP +5 m and continues with a slope of 1:3 up to the dyke crest at a height of NAP +11.67 m. The inner slope of the dyke has a gradient of 1:2.5 with a toe at NAP +7 m. The hinterland is 65 meters long until it reaches polder surface water.

The river water level and head in the dyke are monitored on cross-section TG001 to examine the response of the hinterland aquifer and aquitard during high water levels. This data can be used to assess piping and calibrate the groundwater flow model. The water level data range from November 2017 to May 2018 and are measured at the River Waal, dyke crest (BIK), dyke toe (BIT) and hinterland (AL) locations. The resulting graphs are presented in Appendix C and Figure 5.10.

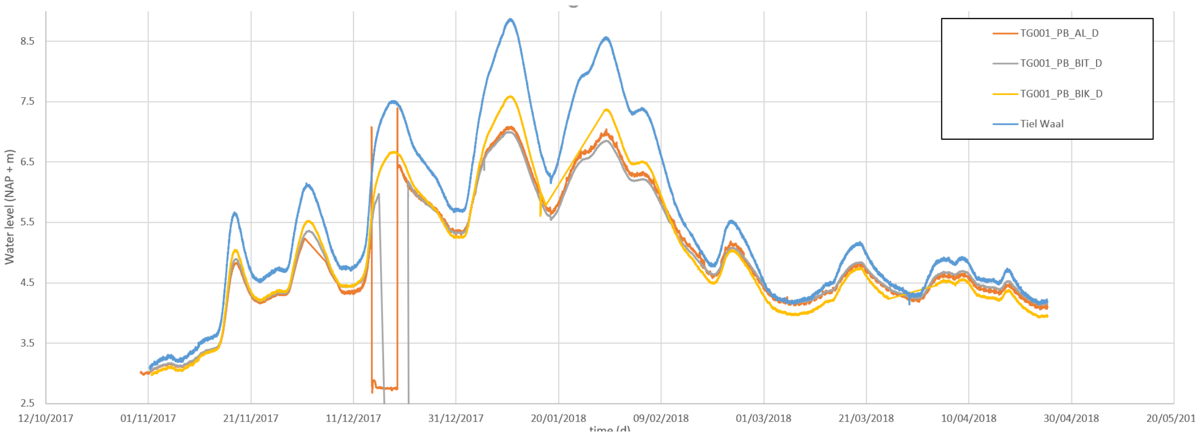


Figure 5.10: Monitoring wells aquifer on cross section (Waterschap Rivierenland, 2018)

The specific dyke section (River kilometre 915) is part of segment 6, which belongs to dyke ring 43 and has a norm failure probability of 1/30,000 per year. The target reliability index is derived from this norm using the probability budget ($\omega = 0.24$) and the length effect ($N = 139$ for $L_{segment} = 46$ km). The piping target reliability is β= 5.3 or P_f= 1/17,375,000 per year with a corresponding water level of 11.5 m according to Hydra-NL (Database June 2017 Boven-Rijn 43-6, Figure 5.11).

The flood wave shape is formed with the general format of water level gradients that is available for different sub-areas in the Netherlands (Rijkswaterstaat, 2017). The water level gradient for the Waal near Tiel is described by sub-area 5 and is displayed in Figure 5.12. The shape is subtracted from the assessment water level of NAP + 11.5 m to acquire the design flood wave.

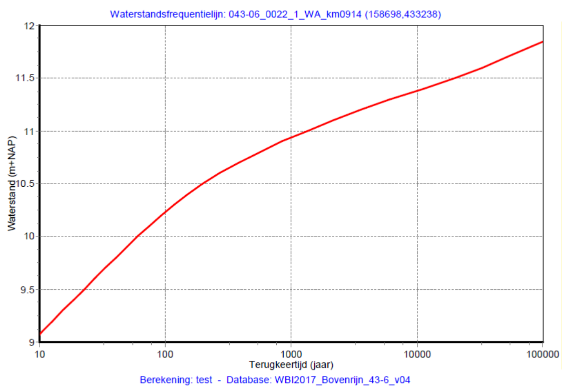
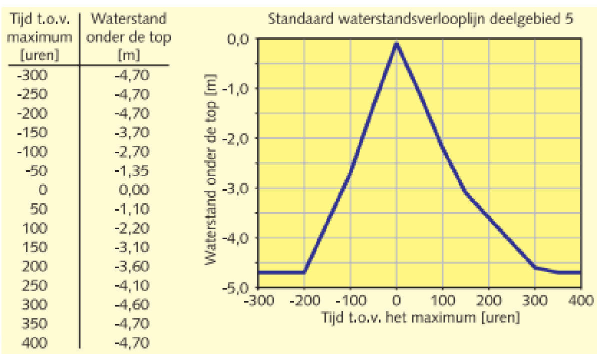


Figure 5.11: Hydra-NL survivor curve June 2017 Boven-rijn 43-6



Tijd t.o.v. maximum [uren]	Waterstand onder de top [m]
-300	-4,70
-250	-4,70
-200	-4,70
-150	-3,70
-100	-2,70
-50	-1,35
0	0,00
50	-1,10
100	-2,20
150	-3,10
200	-3,60
250	-4,10
300	-4,60
350	-4,70
400	-4,70

Figure 5.12: General flood wave shape sub-area 5 Tiel, Netherlands (Rijkswaterstaat, 2017)

A Piping assessment of the current state is performed using a semi-probabilistic method in Appendix D. The factor of safety for Uplift, Heave and Piping is calculated with equations 1.3, 1.4 and

1.5, respectively, and the results are depicted in Table 5.3. The critical head difference for Uplift ($\phi_{c,u}$) and Piping (ΔH_c) is 2.51 m and 2.66 m, respectively. The required failure probability for section 43-6 is $1/17,375,000$ per year (β_{req}= 5.3), which is larger than the combined failure probability of $1/570$ per year ($\beta = 2.92$) and thus does not satisfy the requirements.

Table 5.3: Results Piping assessment

Failure mechanism	Factor of safety semi-probabilistic	(deterministic)	Reliability index (-)	Probability (1/year)
Uplift	0.41	(0.59)	-1.69	0.95
Heave	0.62	(1.04)	-0.20	0.58
Piping	0.48	(1.08)	2.73	0.00031
Combined	$(P_f = P_{f,u} \cdot P_{f,h} \cdot P_{f,p})$		2.92	1.7E-3

5.3.2. PRELIMINARY DESIGN PIPINGONTSPANNER

The Pipingontspanner measure is applied to the problematic segment described in the previous section and is designed with the approach of Chapter 3 and the design specification of Appendix B. Firstly, the Pipingontspanner model is adjusted to the case situation, after which, the model is used to find the preferred alternative of the Pipingontspanner design.

CALIBRATED GROUNDWATER FLOW MODEL

The basis of the Pipingontspanner model is a MODFLOW groundwater flow model that calculates the occurring pressure head throughout the model domain. The subsoil and hydraulic boundary conditions are essential in obtaining an accurate model that represents a reliable Pipingontspanner design and safety assessment. The groundwater flow model setup is described in Appendix E and is summarised by:

Model domain

The area of interest is the dyke section between TG003 and TG000, which is 326 m long. The cross-sectional width of the dyke is limited by the river on one side and a seepage pool on the other side with a total width of 100 m. The model domain is assumed uniform in the dyke length axis, and thus only a part of the total length is required to be modelled. A domain length of 200 m is sufficient to model two edge well and two undisturbed wells for a well spacing up to 40 m.

Geometry

The geometry is equal to the cross-section in Figure 5.9, which holds for the entire dyke section. The system is divided into three separate layers: top layer, aquitard and aquifer. The top layer (layer 1) reaches from REF NAP +12 to 4 m. The aquitard (layer 2) starts at NAP + 4 m and has a thickness of 5.7 m. The aquifer (layer 3) starts at NAP -1.7 m and has a thickness of 20 m.

The Modflow software has problems obtaining a solution when calculation cells are drying up during the simulation, which occurs in the top layer. For this reason, the layer 1 and 2 interface is lowered from the original level of NAP +6 m to NAP +4 m, at which the polder water level (NAP +4.2 m) prevents the drying up of the cells. This change causes the phreatic line to lower from the outer dyke toe. Physically, this means that the basin level starts at NAP +5 m (initial conditions) thus a meter lower than the ground level. The design becomes more conservative as the water level in the basin drops and therefor contributes less to the Uplift safety. At this point, the model results are adequately representative, however, a different setup is required to overcome the drying up limitation.

Layer properties
The assigned properties consist of the porosity (S_y), the water storage coefficients (S_s) and the hydraulic conductivity ($K_{h/v}$). Different values are assigned to each layer, as is shown by Table 2.1. The values for these properties are deducted from the soil type according to the guidelines for drainage measures (Niemeijer et al., 2017) and the groundwater pocketbook (Bot, 2011, Table 2.1 and 2.5).

Table 5.4: Layer properties Tiel

Parametrisation layer model Tiel										
Property	Layer type	Top [m NAP]	Bottom [m NAP]	Thickness [m]	Kh [m/d]	T [m^2/d]	Kv [m/d]	C [d]	Ss [1/m]	Sy [-]
Layer 1 (part a)	dyke body	12	4	8	0.05	0.4	0.05	160	-	0.05
Layer 1 (part b)	empty space	12	4	8	86400	691200	86400	9.3E-05	-	0.99
Layer 2	cover layer	4	-1.7	5.7	0.05	0.285	0.05	152	0.001	0.05
Layer 3	aquifer	-1.7	-21.7	20	5	100	1.5	13.33	0.0001	0.2

Boundary conditions
The polder water level (h_p) is a constant head at NAP + 4.2 m and is located at an influence length ($L_i = 3 \cdot \lambda_h = 378$ m) from the river border. A head-dependent flux boundary simulates this constant head at x = 100 m and calculates the flow with Darcy's law and the gradient between the two points.

The river level varies according to the river flood wave shape depicted in Table 4.1, which is translated to a step function with a step size of 12 hours. The flood wave starts at NAP +6.7 m and reaches the maximum river level of NAP + 11.5 m in 8 days.

Model calibration
The simplified schematisation and estimated parameters allow the model to deviate from reality, which influences the reliability of the piping assessment and Pipingontspanner design. For that reason, relieve well measurements are used to validate and calibrate the groundwater flow model. The river level and aquifer head in the winter season of 2017/2018 is depicted in Figure 5.10.

The groundwater flow model is calibrated with the dampening factors and water level gradients from the measurement, which are based on a river peak in the data set (29-01-18). The water level gradients of the phreatic water and aquifer during this river peak are presented in Figure 5.13. The dampening factors and water levels for the same point in time are depicted in Table D.2a.

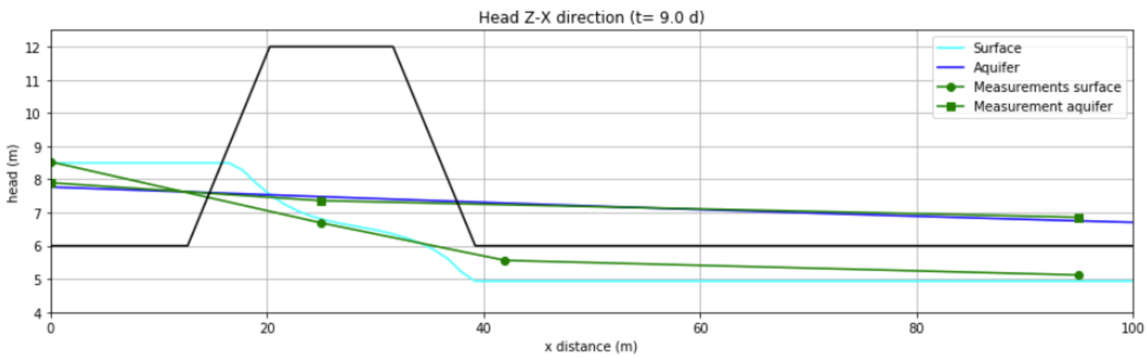


Figure 5.13: Calibrated Groundwater flow model with data monitoring wells

The groundwater flow model is adapted using the riverbed resistance (C) and the aquifer conductivity parameters ($K_{h,v}$), which control how fast the water enters the aquifer and how the gradient develops over the dyke, respectively. The river flood wave and initial water levels are adapted to the period of 18/01 to 09/02, in which the initial water levels are especially important for the surface level development. The groundwater flow model showed similar results when increasing the riverbed resistance to 2 days and decreasing the permeability of the dyke and aquitard to 1E-2 m/d.

A comparison between the model and measurement gradients, depicted in Figure 5.13, shows that only the surface gradient deviates from the measurements. The modelled phreatic line in the dyke is influenced by the changes made to solve the dry-cells problem, which misrepresents the water level in the hinterland. The model accuracy is considered sufficient for a global assessment. The model is based on a design flood wave with the initial water level conditions of 20-01-18 and the calibrated parameters, resulting in the scenario depicted in Figure 5.14.

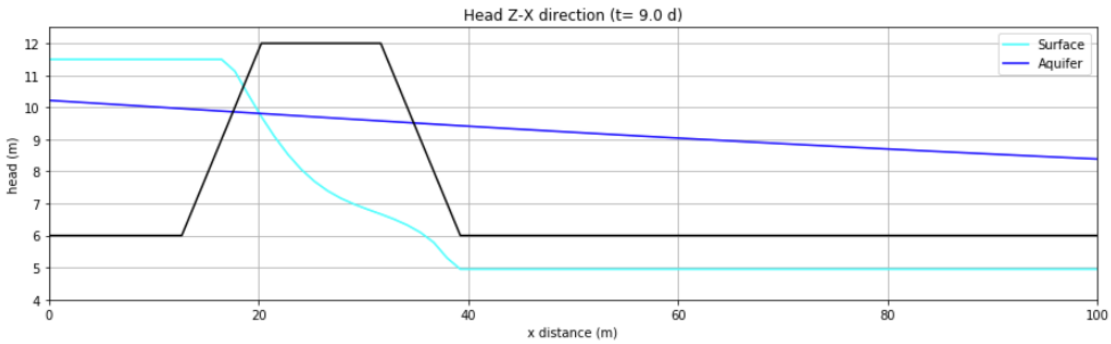


Figure 5.14: Calibrated Groundwater flow model assessment scenario

The alternatives are calculated with mean and calibrated parameters after which the safety assessment is performed with the characteristic values. The semi-probabilistic approach requires the characteristic values of Table D.3 and includes a 95-percentile value for the dampening factor instead of the calibrated value. The aquifer head is multiplied with a 1.075 factor in the safety assessment to obtain the correct water level. The model factor is determined by the ratio of the estimated assessment level at the exit point divided by the calibrated water level at the exit point (10.1/9.4 = 1.075).

PIPINGONTSPANNER MODEL

The Pipingontspanner model uses the setup of the previous section to calculate alternatives, verify the compliance of the requirements and select the best alternative according to the evaluation criteria. The alternatives are generated with the design variables well spacing (L_w) and basin width (w_{sb}) and consist of 130 configurations that vary from $L_w = 10 - 37.5$ m and $w_{sb} = 10 - 42.5$ m.

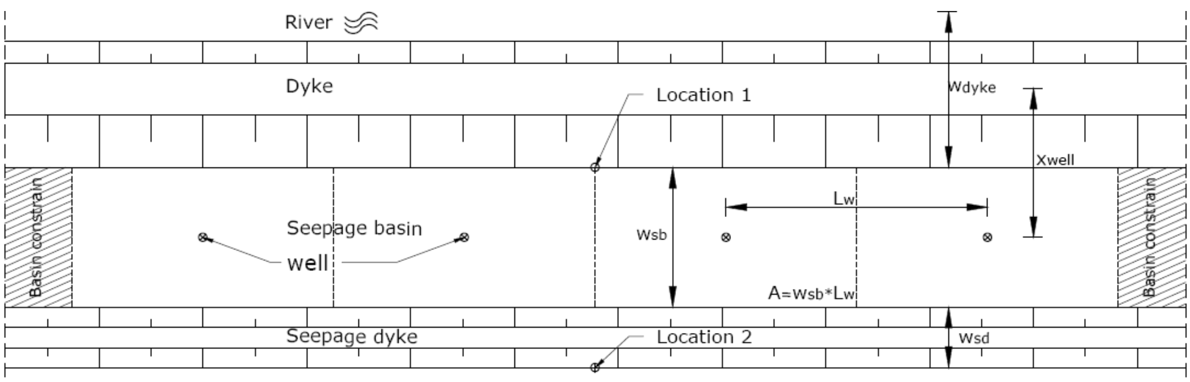


Figure 5.15: Overview Pipingontspanner design variables

The model provides the total cost and the piping failure probability for each alternative, after which the results that do not comply with the reliability requirements are dismissed. The failure probability of the alternatives at the inner dyke toe (location 1) and behind the seepage dyke (location 2) is displayed in Figure 5.16. The required failure probability is 5.76E-8 per year, which needs to be achieved on both locations.

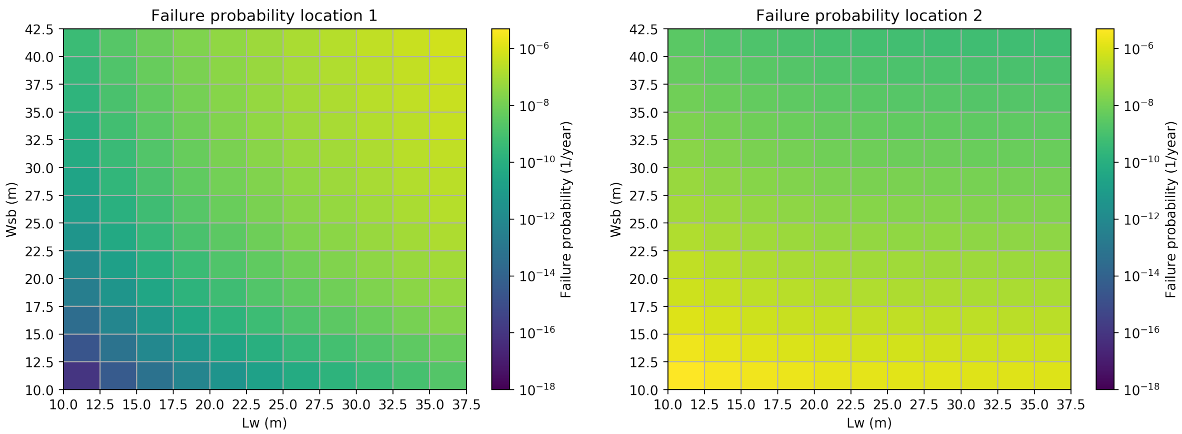


Figure 5.16: Failure probability location 1 (inner dyke berm x=52 m) and 2 (behind seepage dyke $x = 52 + w_{sb} + w_{sd}$)

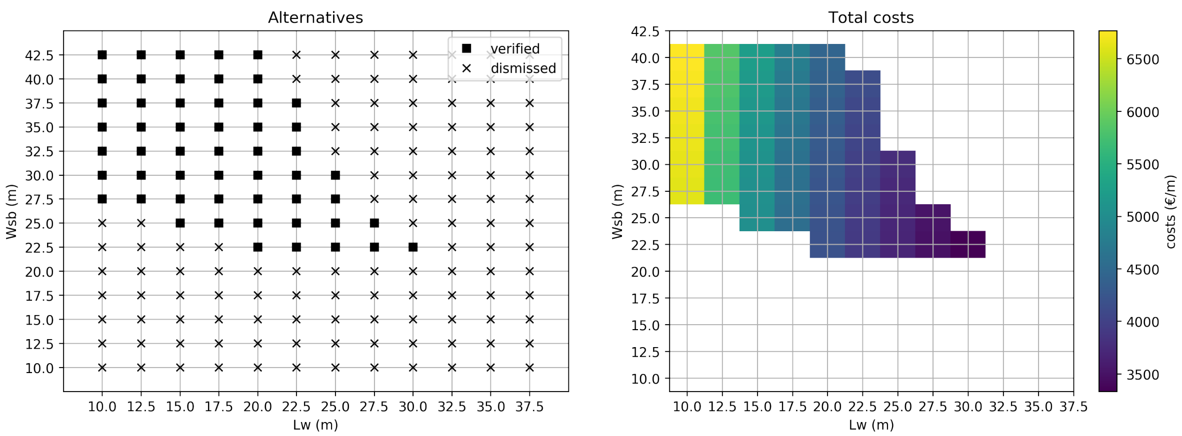


Figure 5.17: Alternatives and resulting total costs

The remaining options are presented in Figure 5.17, where the dismissed alternatives are represented by a cross and without costs. The most economical alternative is a Pipingontspanner with a well spacing of 30 m, a basin width of 22.5 m and a total cost of 3332 €/m. The groundwater flow results with and without the preferred alternative are presented in Appendix E.

Table 5.5: Preferred alternative details

Design parameter	Well spacing L_w	Basin width w_{sb}	Well distance x_{well}	Seepage dyke height and width h_{sd}	w_{sd}
Value	30 m	22.5 m	15 m	2.3 m	10.2 m
P_f location 1:	5.5 E-8	per year	P_f location 2:	3.6E-08	per year

5.3.3. PRELIMINARY DESIGN PIPING BERM

The Piping berm is the alternative of the Pipingontspanner and will be used to determine the economic feasibility based on the case Tiel. The piping berm measure is applied to the problematic segment TG003 -TG000 and is designed according to the guidelines of constructive designs for dyke reinforcements (Technische Adviescommissie Waterkeringen, 1994).

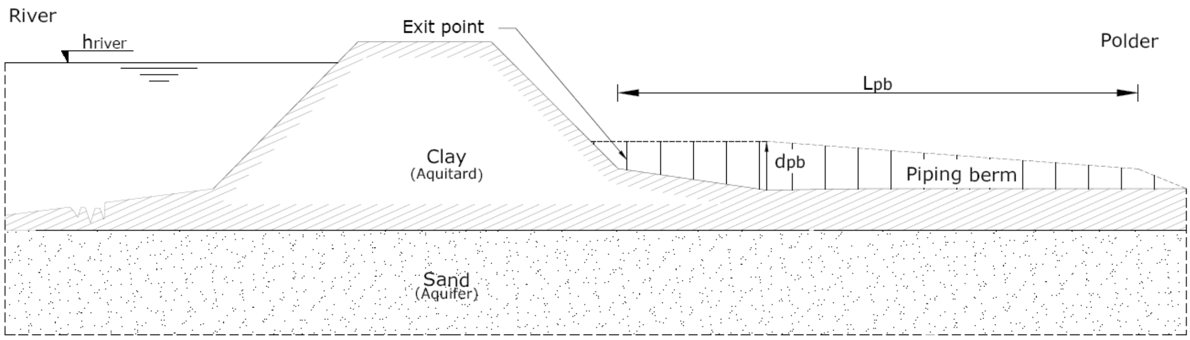


Figure 5.18: Overview Piping berm design

The piping berm adds weight on the cover layer in the form of soil and thereby prevents the Uplift mechanism. The thickness of the berm should counteract the Uplift force on the cover layer resulting in a higher factor of safety for Uplift. The length of the berm is determined by the required seepage length of the Piping mechanism. The berm is constructed with a core of sand (γ_d = 18 kN/m^3) and a 0.5 m thick clay cover layer with grass that protects the sand core against erosion (Technische Adviescommissie Waterkeringen, 1994). The berm design is displayed in Figure 5.18 and shows a slope of 1:2 at the end of the berm.

The required thickness at the exit point is 2 m for a Uplift safety factor of 1 (with characteristic values). The required seepage length is 121 m and results in a berm length of 73 m at which the Piping safety factor equals 1. However, the berm length can also be calculated with the required failure probability according to the new WBI assessment method, which reduces the length to 57 m. This means that the combined probability of Uplift, Heave and Piping at a length of 57 m is smaller than the required probability (P_f = 2.48E-8 < 5.76E-8 = $P_{f,req}$) instead of a Piping safety factor of 1.

The piping berm cost is determined according to the same setup as the Pipingontspanner (Subsection 5.2) and is based on the unit prices of the seepage dyke in Appendix B. The cost composition for the piping berm is depicted in Table 5.6 and renders a total cost of 6825 €/m.

Table 5.6: Cost definition piping berm

Phase	Part	Unit price		Quantity	Cost (€)
Construction (Cc)	Sand core	15	€/m^3	$(L_{pb} \cdot (h_{pb} - 0.5)) + (h_{pb} - 0.5)^2$	870
	Clay cover	30	€/m^3	$((L_{pb} \cdot h_{pb}) + h_{pb}^2) - A_{sandcore}$	892.50
	Acquiring land and site preparation	20	€/m^2	$L_{pb} + 2h_{pb}$	1200.00
	Vegetation (grass)	0.75	€/m^2	$L_{pb} + \sqrt{2} \cdot h_{pb}$	44.34
	Project preparation & management	15% Cc	€/LC	1	451.03
Maintenance (Cm)	Berm inspection (height, vegetation)	0.1	€/m^2/y	$100(L_{pb} \cdot 1)$	570.00
	Restore height and vegetation	10	€/m^2/5y	$20(L_{pb} \cdot 1)$	1140.00
Design (Cd)	Design piping berm	10% Cc	€/LC	1	345.79
Removal (Cr)	Remove piping berm	20% Cc	€/LC	1	691.57
Unforseen costs (Cu)	(Cd + Cc + Cm + Cr)			10% (Cd + Cc + Cm + Cr)	620.52
Life Cycle Costs	Total costs (TC)			(Cd + Cc + Cm + Cr + Cu)	6825

5.3.4. EVALUATION PIPING MEASURES

This section evaluates the costs of both measures but also considers the benefits that set the different designs apart. The life cycle cost of applicable Pipingontspanner designs range from 3332 to about 6700 and offers cheaper alternatives than the Piping berm which has a cost of 6825 euro. The cheapest Pipingontspanner alternative is 3332 euro and shows that the measure is economically interesting.

The length of the measure could have a significant impact on the hinterland by conflicting with houses and other objects in the vicinity of the dyke. The Piping berm reaches 57 m inland and is double the Pipingontspanner length. The piping berm is a simple and effective measure but has an impact on the hinterland, especially for urban area's. Recently, the assessment criteria adopted the improved Sellmeijer 2011 formulation, which is known for producing twice as large seepage lengths (Arcadis, 2012, Luijendijk et al., 2017). This development is especially unfavourable for piping berms as the length increases considerably. For the case of Tiel, the Pipingontspanner offers a shorter length and leaves room for nature development, farming and recreation inside the seepage basin. The additional seepage volume in to the polder that the Pipingontspanner introduces is negligible compared to the volume in the base situation (Appendix E). When the available space behind the dyke is an important aspect, a sheet pile, geotextile or coarse sand barrier measure would offer a better solution than the Pipingontspanner.

Figure 5.1 showed an overview of construction costs ratios for drainage measures compared to a piping berm, in which most of the drainage measures are equal to or more costly than a piping berm. The construction cost ratios of Pipingontspanner compared to the Piping berm is 50% and differs with the image presented by other drainage measures, which compare against a 20 m wide berm. In general, cost prices of different projects are hard to compare as local conditions can change drastically. In the case of Tiel, these conditions produce a considerable Pipingberm length and height, which reflects in general the construction costs. Ultimately, a single case is not representative of the economic feasibility, however, it is a first indication of the measure's potential.

5.4. FINAL REMARKS

The goal of this chapter was firstly to establish the economic feasibility of the Pipingontspanner by comparing it to the piping berm, and secondly to show the applicability of the measure in the realistic scenario presented by the case study. The cost analysis and case study have demonstrated that the Pipingontspanner is applicable and economically competitive compared to a piping berm. The following important conclusions are drawn concerning the Pipingontspanner design:

- The current economic selection criterion prefers a solution with a minimum number of wells, due to the high well maintenance and monitoring costs. However, a smaller well spacing would be favoured over a larger basin in a geotechnical perspective, because the inhomogeneity influences in the subsoil could cause variability in the well performance.
- The distribution of the failure probability in location 2 differs strongly from location 1 for the possible design configurations (Figure 5.16). Additionally, location 2 was the main driver in the design choice as the highest failure probability always occurred at this point. Location 2 was not investigated during the sensitivity analysis but could lead to a refinement of the design.
- A possible design improvement could be to move the well location towards the middle of the basin to have a evenly distributed failure probability in location 1 and 2.
- The Pipingontspanner design is an iterative process and the results of this case study show the first cycle.

6

Conclusions & recommendations

6.1. Conclusions

The goal of this research was to establish the technical and economical feasibility of the Pipingontspanner concept as a piping measure. As the new safety regulations are expected to identify more piping prone dyke sections in the Netherlands, this new concept is worthwhile to investigate. A problem analysis highlighted the components of the Pipingontspanner that form challenges in realising the concept and pinpointed the research to the questions below:

1. How can the seepage discharge be modelled with the required accuracy level?
2. Which requirements can be used to assess the reliability of the Pipingontspanner concept?
3. Which design approach defines the Pipingontspanner concept?
4. Under which conditions is it possible to implement the Pipingontspanner as a piping mitigation measure?
5. What is the impact of the Pipingontspanner on the macro stability of the dyke?
6. Under which conditions is the Pipingontspanner economically favourable compared to other piping mitigation measures?

To answer the questions above, the following topics are covered in this report: the calculation method, design approach, applicability and cost effectiveness of the Pipingontspanner. The conclusions are summarised as follows:

1. Analytical solutions for pumping wells and empirical solutions for relief wells are available and well documented, but differ on key points from the Pipingontspanner concept, leaving only numerical methods to describe the seepage discharge. One of the key points is the inability to include the time dependent interaction between the river and the seepage basin. The groundwater flow software Modflow can describe the flow in the Pipingontspanner concept, but may only be used to predict the possible effects as there is no validation data. The non-uniformity of subsoil parameters has a considerable influence on the results of a groundwater flow model, demonstrated by the sensitivity analysis, and causes inaccurate results of the model. The purpose of the model is to provide a representative overall view of a functioning Pipingontspanner, which leaves the accuracy to representative subsoil parameters, hydraulic conditions and geometry. However, the case study showed that, with the current model setup, adaptations in the geometry might be necessary to prevent the drying up of the top layer cells, which would result in early termination of the simulation.

2. The reliability of the flood defence is influenced by the failure mechanism for piping (Uplift, Heave and Piping) and the well that is introduced by the Pipingontspanner. The calculated drawdown of the well can be impaired by deformation, vandalism, clogging or blockage and either of which could result in Piping failure. The well is designed with protective measures that reduce the likelihood of these mechanisms. The Uplift assessment model, as described by the assessment guidelines, does not account for the counter pressure of the basin, but it is incorporated by adding the basin water level as a weight in the resistance term. This approach only holds for an impermeable aquitard, as the phreatic line is not affected by the basin. Furthermore, the critical location for the piping assessment is not known in advance and depends on the location of the well and the width of the measure. The possible critical locations are the inner dyke toe, around the well and at the end of the measure.

3. The Pipingontspanner concept is led by two mechanisms: relieving water from the aquifer using a well and applying counter pressure with the relieved water. The water from the well is stored in a basin that is enclosed by an additional smaller (seepage) dyke. In the absence of design guidelines or analytic formulations, a parametric design can be used to determine the optimal configuration using the groundwater flow model. The design variables used in this approach are the dimensions of the basin and the spacing between the wells, which determine the extent of pressure relief and counter pressure. The remaining variables in the design are chosen constant or made dependent on location properties as their influence is easier to estimate. However, the case study revealed that the location of the well, which was assumed constant in this study, can be a valuable design parameter and shows that the design approach of the Pipingontspanner is an iterative process.

4. The applicability of the measure is determined by geometric restrictions behind the dyke, seepage discharge limits and local soil conditions. A sensitivity analysis of the Pipingontspanner model showed how the concept performs in a variety of subsoil and hydraulic conditions. The results indicated that the safety for Uplift increases for a higher permeability of the aquifer and a slowly increasing hydraulic load. The cover layer permeability and storage coefficient do not affect the performance of the Pipingontspanner. Additionally, high basin water levels ($h_{sb} > 75\%\Delta h$) are found for small values of well spacing and basin width ($L_w, w_{sb} < 10$ m), which could cause a secondary piping mechanism.

5. The Pipingontspanner increases the dyke slope stability by increasing weight on the passive side with the water from the storage basin. Additionally, the well in the basin relieves the upward aquifer pressure, which creates additional resistance for macro instability. The Pipingontspanner concept is not designed to prevent macro instability, but the configuration can take this aspect into account by providing higher basin water levels that reduce instability.

6. The cost drivers of the Pipingontspanner are well maintenance, well monitoring and seepage dyke construction costs. The number of wells and the seepage dyke height increase if the basin width is limited, which drives up the total cost substantially. The cost analysis and case study have shown that the Pipingontspanner is cost effective compared to a traditional Piping berm if the required seepage length is large. Based on these findings, the Pipingontspanner is economically favourable in (rural) areas with a sufficient amount of hinterland space and a permeable aquifer.

In conclusion to answer the main research question, the Pipingontspanner is a piping mitigation measure that is economically competitive with the piping berm and technically possible under the conditions of:

1. a permeable aquifer with a minimum transmissivity of 25 ($T_{aquifer} \geq 25$ m^2/d);
2. a minimum hinterland space of 10 m behind the dyke ($w_{sb} \geq 10$ m);

6.2. RECOMMENDATIONS

As the Pipingontspanner is a new concept with many angles, not all could be covered in the scope of this study. Therefore future studies could investigate the following topics:

The social acceptance and implementation of the Pipingontspanner, which were identified as challenges in the problem analysis and need to be investigated to discover potential pitfalls in the design. Furthermore, additional uses of the empty basin space could be examined to explore the potential added value of the measure compared to other alternatives.

The current setup of the groundwater flow model is sufficient to estimate the global effects of a Pipingontspanner, but a more advanced model is required to create a location-specific design. The dry-cell limitation in the current setup needs to be resolved in order to have more flexibility in modelling the problem. Additionally, the inhomogeneity of the subsoil and a variable geometry along the dyke need to be included into the model to increase the accuracy of the model. A pilot project of the Pipingontspanner could validate the results that are obtained with the groundwater flow models and confirms the functioning of the measure.

The cost comparison to the Piping berm demonstrated that the Pipingontspanner is economically interesting, but it does require a comparison with a larger variety of measures before its general economic feasibility can be confirmed.

Lastly, there are more options that can be investigated to optimise the design of the Pipingontspanner. Some examples are:

- In the current study, the wells are placed in a straight line, but a different configuration might be more effective.
- The addition of an extra well in the cross direction could help in cases with large piping problems.
- The seepage dyke height could be reduced by discharging the water from the basin at a certain height.

A

DETAILED RESULTS GROUNDWATER FLOW

This Appendix provides a summary of the groundwater flow calculations and shows additional results for the seepage discharge into the polder and different cell resolutions (5x5, 2.5x2.5 and 1.25x1.25 m). The groundwater flow model is fully described in Section 2.3. The schematisation of the POS model is depicted in Modelschem (Figure A.1) on the next page. The last is described for a cell size of 5x5 m to show a clear image of the model decisions.

Geometry:

- The model domain is 50 x 50 metres.
- The geometry is uniform along the y-axis.
- The situation is divided into 3 layers: top layer, aquitard and aquifer.
- The top of the main and Seepage dyke are REF + 5 m and REF +3 m, respectively.
- The seepage basin is 15 x 50 x 3 m (w_{sb} x l_{sb} x h_{sd}).
- The aquitard is 3 m thick with a top at REF -1 m.
- The aquifer is 5 m thick with a top at REF -4 m.

Layer properties:

- The layer properties are given in Table 2.1, which originate from an example case (Waterontspanner) on the river Lek near Langerak.
- Layer 1 is divided into 2 part (dyke and empty space), which are modelled using the conductivity properties of the soil.

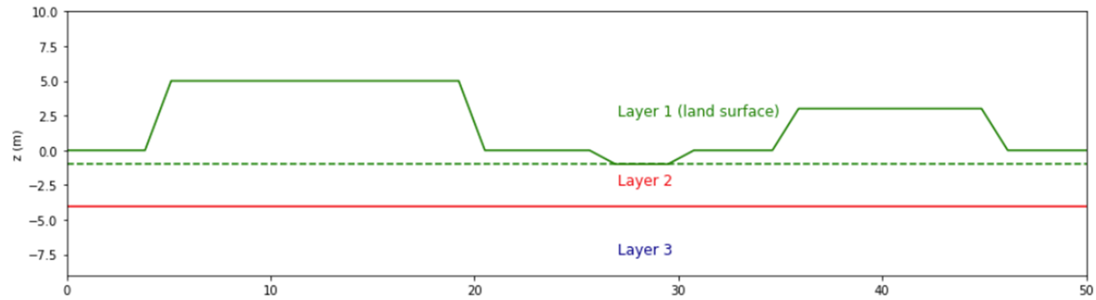
Boundary conditions:

- The outer edge of the model is a no-flow boundary.
- The river boundary is river flood wave on top of the mean high water level (REF +0.5 m).
- The polder boundary is a constant head of REF 0 m at x = 500 m. This boundary is simulated using a head dependent flux boundary.
- The well is located at point (x,y) = (27.5,22.5 m).

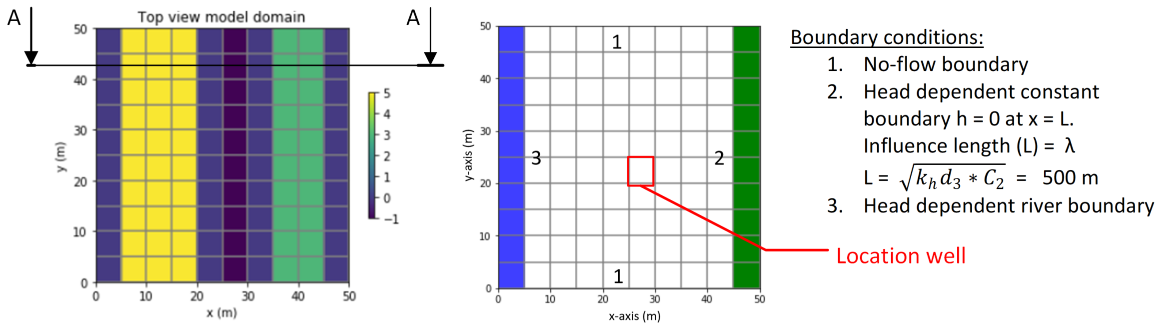
Well:

- The well is incorporated into the model using the hydraulic properties of a cell. The cell is given the equivalent vertical hydraulic conductivity of the well. The conductivity (K_v) of the well for a 5x5 m cell is 10 m/d.

Modelschem: Situation with POS (grid 5 x 5)



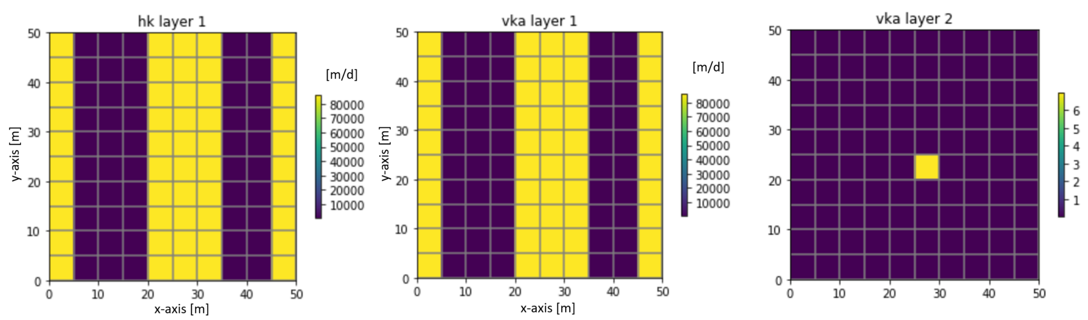
Cross-section A-A: geometry



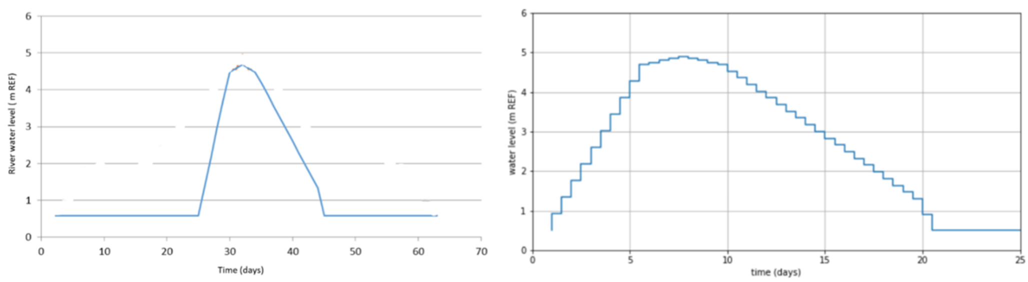
Model domain: top view

Model boundaries: top view

Parametrisation layer model										
Property	Layer type	Top [m NAP]	Bottom [m NAP]	Thickness [m]	kh [m/d]	T [m^2/d]	kv [m/d]	C [d]	Ss [1/m]	Sy m/m
Layer 1 (part a)	dyke body	10	-1	11	0.005	0.055	0.005	2200	-	0.05
Layer 1 (part b)	empty space	10	-1	11	86400	950400	86400	0.00013	-	0.99
Layer 2	cover layer	-1	-4	3	0.0005	0.0015	0.0005	6000	0.001	0.05
Layer 3	aquifer	-4	-9	5	10	50	3	1.67	0.0001	0.2



Layer properties: overview table and cell assignment



River boundary: schematisation: example flood wave (River lek, langerak) and Modflow input

Figure A.1: Schematisation summary

A.1. SEEPAGE DISCHARGE

The amount of seepage entering the polder via the aquifer is important for the polder water management and an increase could have consequences for Waterboards. The seepage discharge of the Pipingontspanner needs to be analysed to determine the additional volume of water that enters the polder. The seepage discharge is measured in the aquifer underneath the dyke at x = 12.5 m, in which a polder directed value is positive. When comparing the situation with and without the Pipingontspanner in Figure A.2, the maximum seepage discharge increases substantially from 0.5 to 2.6 m^2/d. The well draws more water from the river, but also flows back into the river at t = 22 days.

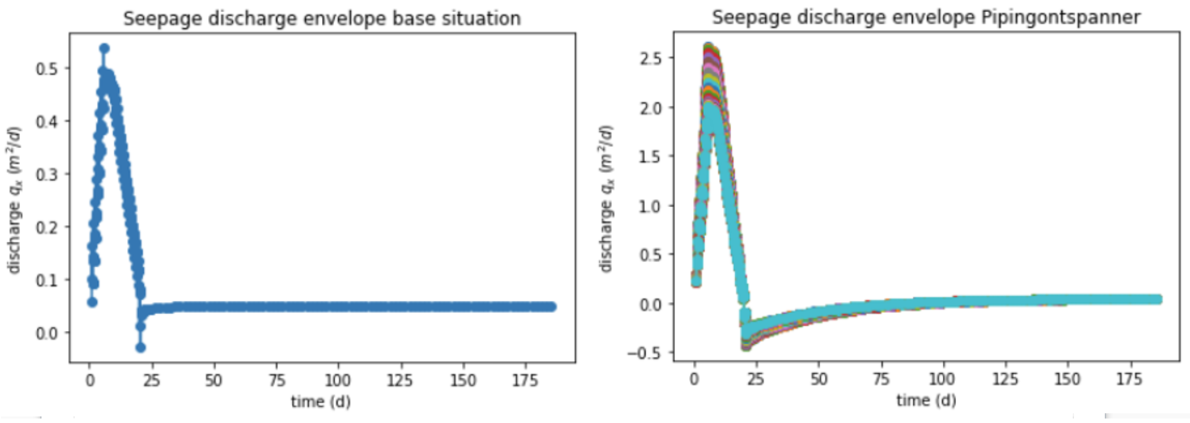


Figure A.2: Seepage discharge envelope base and Pipingontspanner situation (x= 12.5 m, y= 0 - 50 m)

The Waterboard is interested in the total additional volume of seepage water that enters from the river with a Pipingontspanner, which is depicted in Figure A.3. The well draws 4 times more water from the river until t = 22 days, of which is most stored in the seepage basin. However, a part of this volume flows back into river if the river level becomes lower than the basin level. This scenario is only possible when the basin level exceeds the river level and the hydraulic resistance of the river bottom is low. Overall, the Pipingontspanner did cause an additional seepage volume of 6 m^3 per meter dyke per flood wave event.

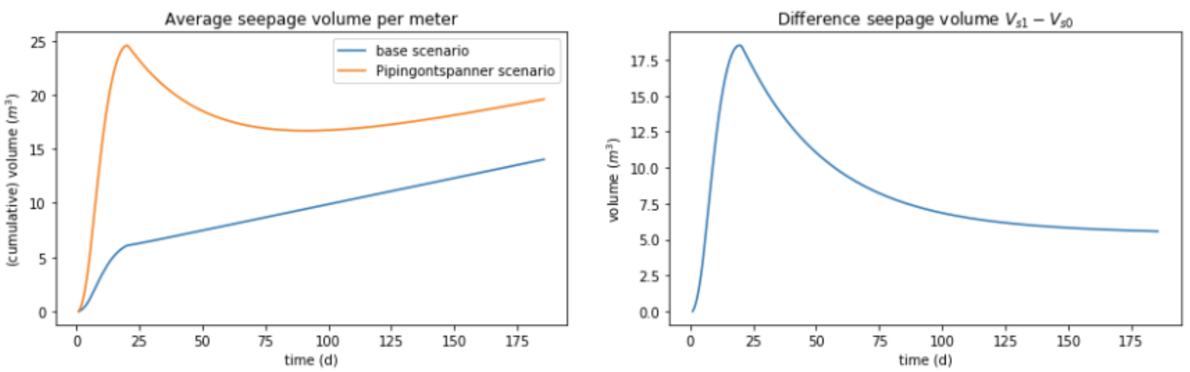


Figure A.3: Seepage volume base and Pipingontspanner situation (x= 12.5 m, y= 0 - 50 m)

The amount of seepage water is highly dependent on the soil properties and thus needs to be checked in each case. In general, the Pipingontspanner will draw less water from the river than active pumping wells and could even transfer the water back to the river in the right circumstances.

A.2. RESOLUTION CELL SIZE

The calculation results for the Pipingontspanner (POS) groundwater model are depicted on the next pages in Modelmap 2, 3 and 4 for cell resolution 5x5, 2.5x2.5 and 1.25x1.25 m, respectively. A calculation with a smaller cell size results in a higher accuracy because more evaluation points result in less interpolation errors between cells. The following notes can be made for differences between the results:

- The phreatic level inside the dyke (Cross-section A-A) is described in more detail in Modelmap4 resulting in a more realistic transition of the water line.
- The discharge through the well (detail A) differs slightly between the modelmaps, from 100 m^3/d in Modelmap2 to 95 m^3/d in Modelmap4. The maximum discharge through the well (Detail B) was kept constant for each cell size by changing the well conductivity, however small differences did occur. This can potentially be explained by the increase of calculation nodes in the surrounding area.
- The head in the aquifer (detail D) becomes lower for a decreasing cell size. This phenomena is caused by an increasing flow velocity in the well due to the smaller cell size. The lower head of Modelmap 4 also results in an increase of head difference between a situation with and without Pipingontspanner.

In conclusion, it is recommended to use a smaller cell size to accurately calculate the head difference produced by the measure, while designing the Pipingontspanner. The detailed phreatic level in the dyke body is depicted below using the cell resolution of Modelmap 4.

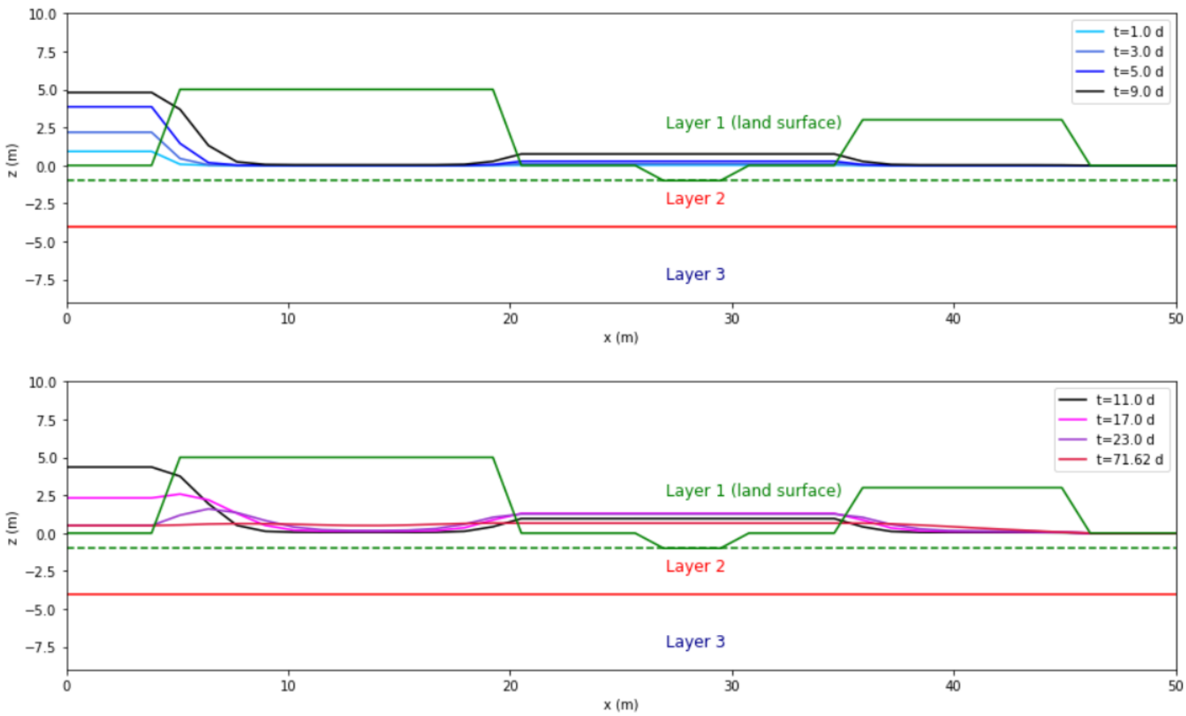


Figure A.4: Modelmap4: phreatic level over time

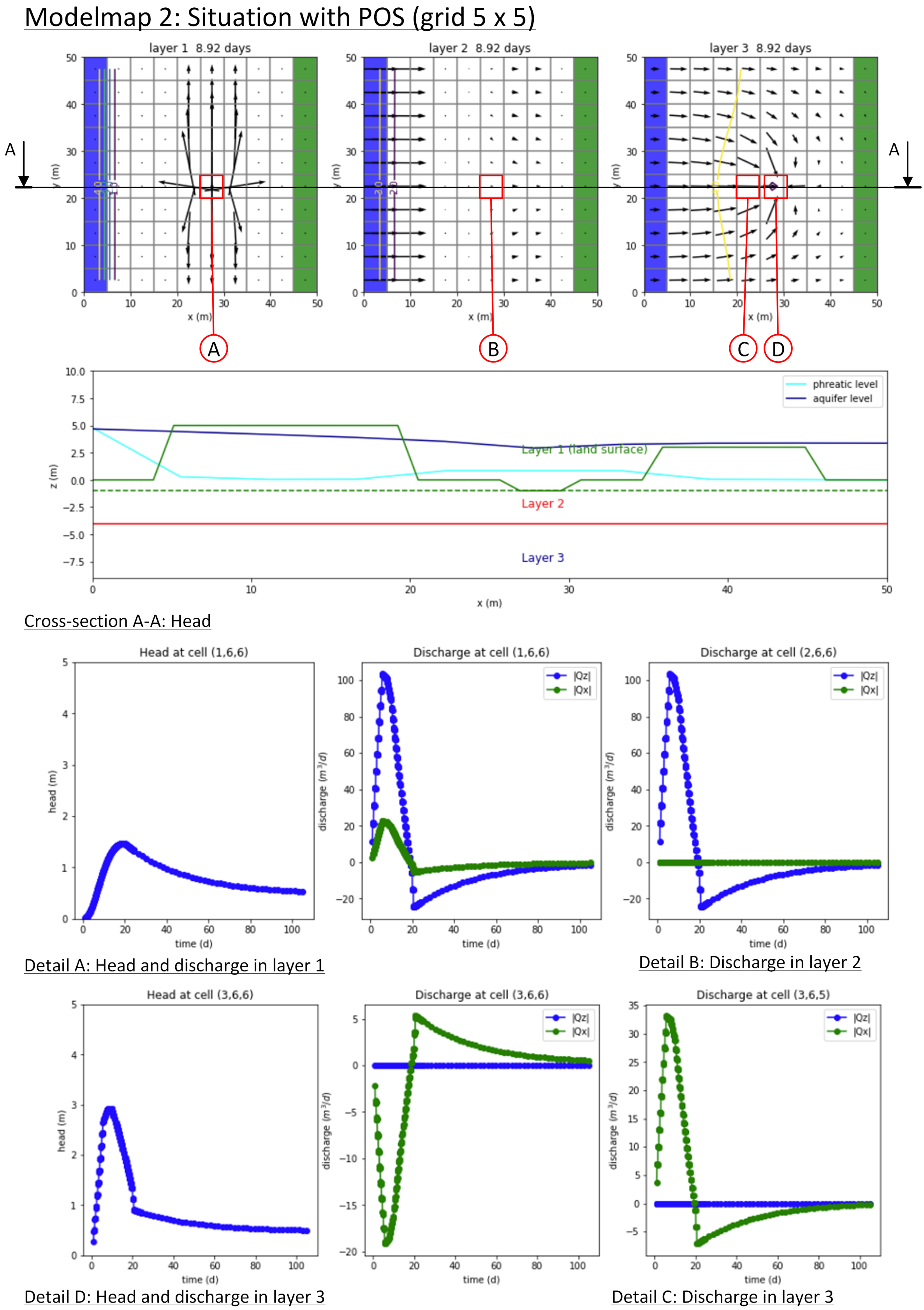


Figure A.5: Modelmap2: Modflow results with cell resolution 5x5 m

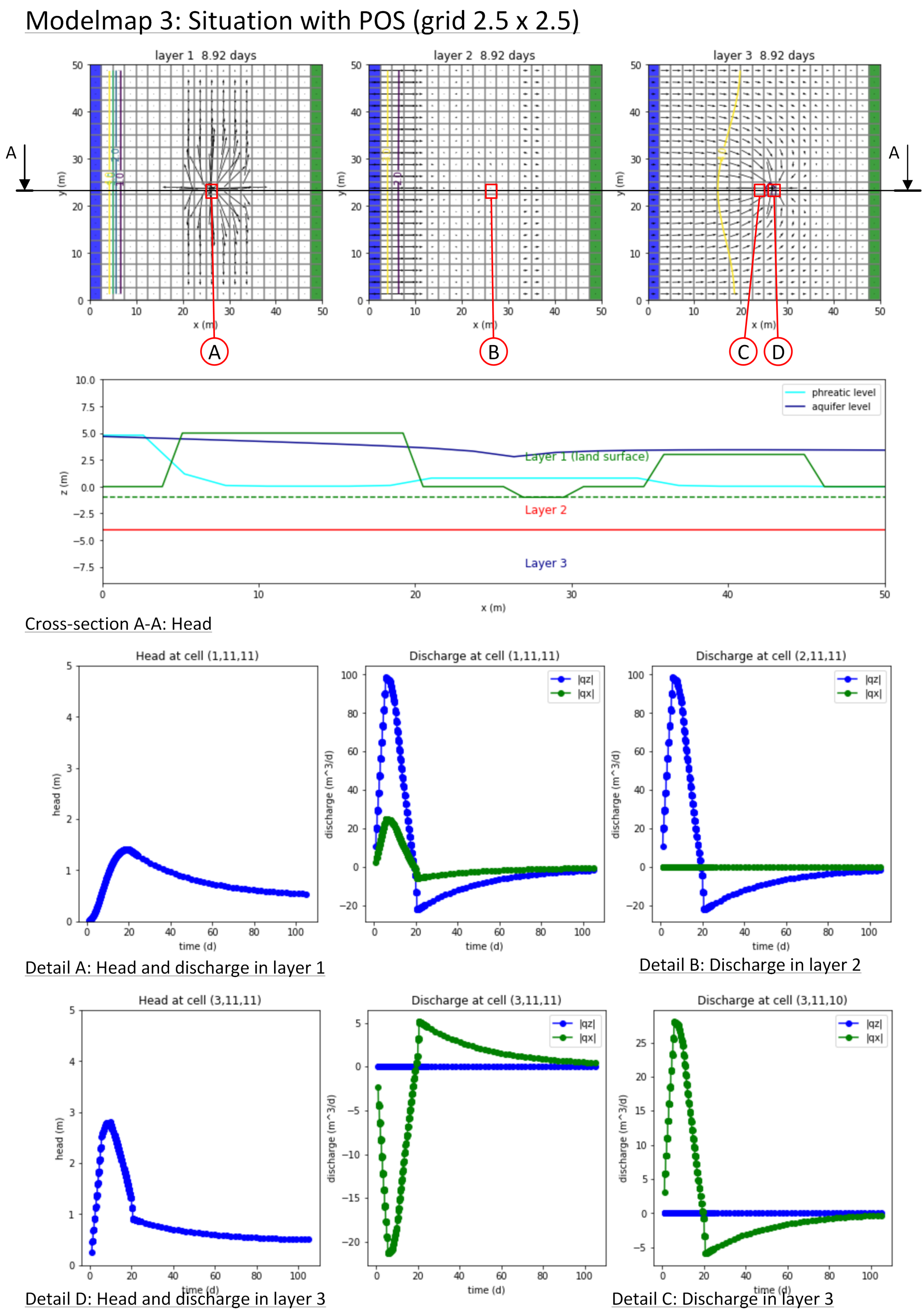


Figure A.6: Modelmap3: Modflow results with cell resolution 2.5x2.5 m

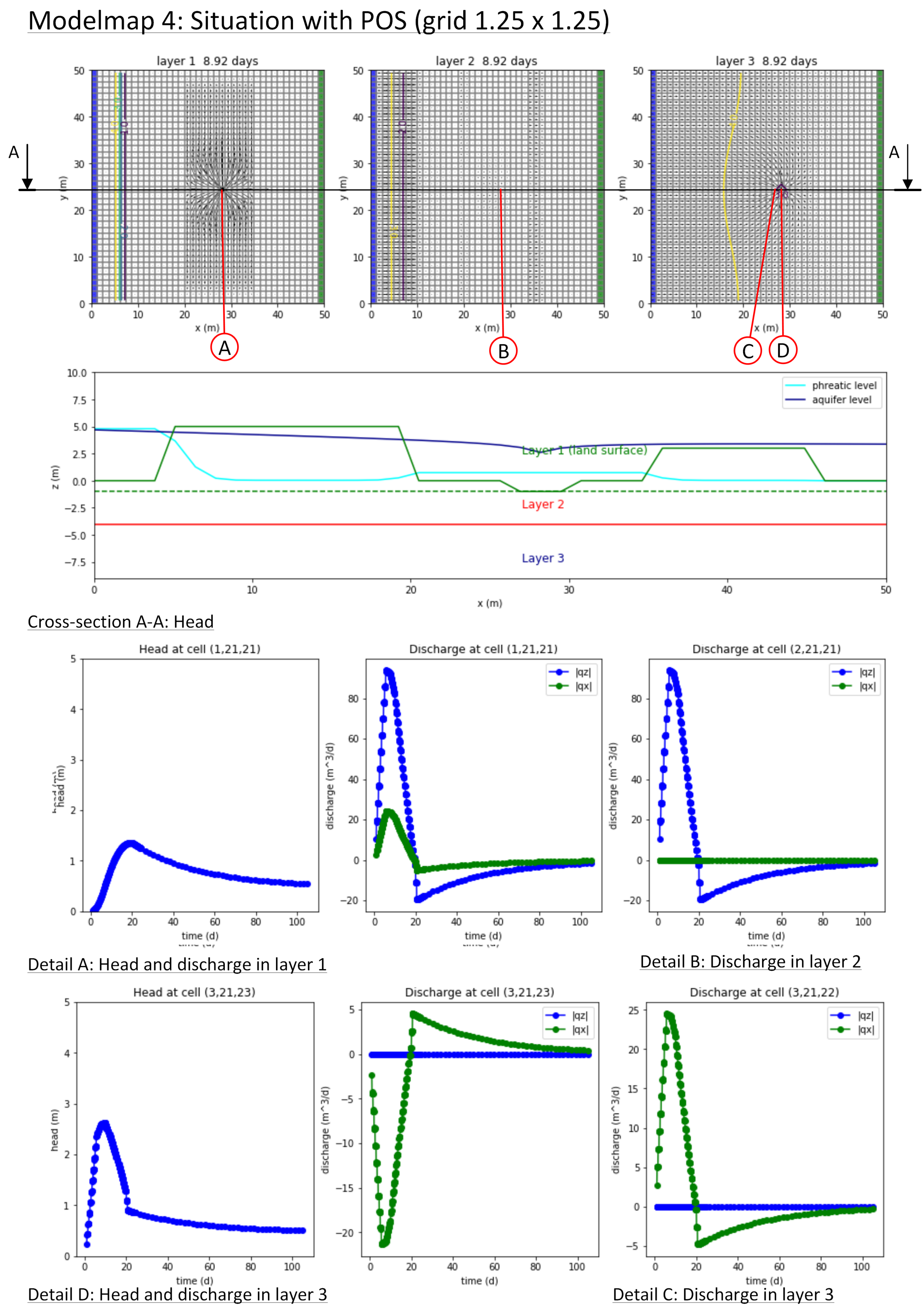


Figure A.7: Modelmap3: Modflow results with cell resolution 1.25x1.25 m

B

PIPINGONTSPANNER DESIGN

This Appendix determines the preliminary design and unit costs of the Pipingontspanner, which are used in the cost analysis of Chapter 5. Furthermore, this preliminary design contributes to the technical feasibility of the concept. The maintenance strategy is part of the life cycle design and included in this appendix.

B.1. PRELIMINARY DESIGN SPECIFICATION

The different objects in the Pipingontspanner are depicted in the object breakdown structure (Figure B.1) and shows the two main elements that are described here: seepage dyke and well. The preliminary design does not focus on the already existing main dyke and flooding area. The concepts described and developed in Chapter 3 are used as a basis for the design.

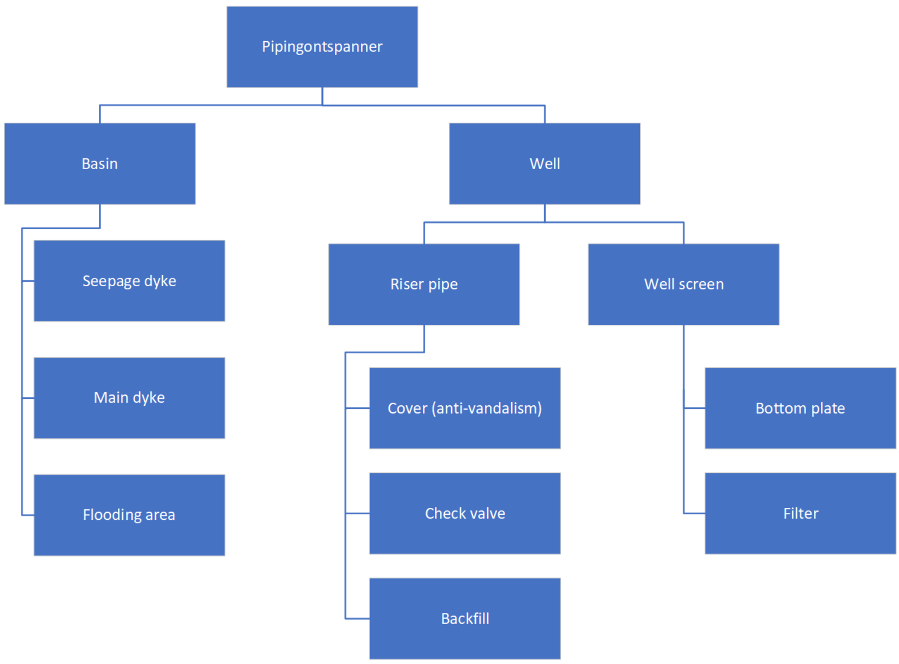


Figure B.1: Object tree Pipingontspanner

B.1.1. SEEPAGE DYKE

The main function of the seepage dyke is to retain water inside the basin by forming an impermeable wall. This wall is formed by a clay dyke with a core of sand and is depicted in Figure B.2, according to the guidelines of new dykes (Jonkman et al., 2017). The dyke forms a horizontal hydraulic resistance using a clay layer of 0.8 m thick with a moderate dens clay. There is no significant erosion of the dyke material since the flow velocities are low during the gradually filling and emptying of the basin. The gradient of the dyke slopes are 1:2 to minimise the footprint of the dyke.

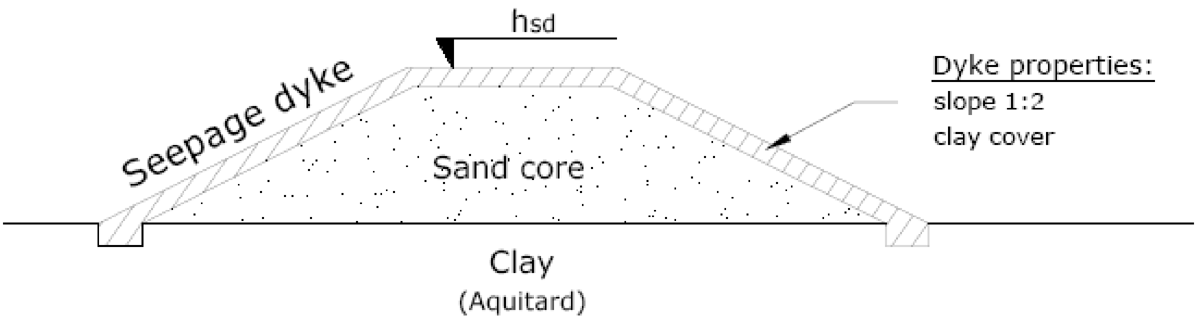


Figure B.2: Design seepage dyke

B.1.2. WELL

The well is a vertical connection between the aquifer and the seepage basin that transfers water in both directions. The standard layout of a well consist of a pipe with a screen that is surrounded by a filter (USACE, 1992). This filter prevents the loss of foundation material and consist of granular material. The riser pipe connects the screen to the surface level and is surrounded by a backfill that blocks the flow along side the pipe. The top of the well can be closed off with a valve and is protected by a cover.

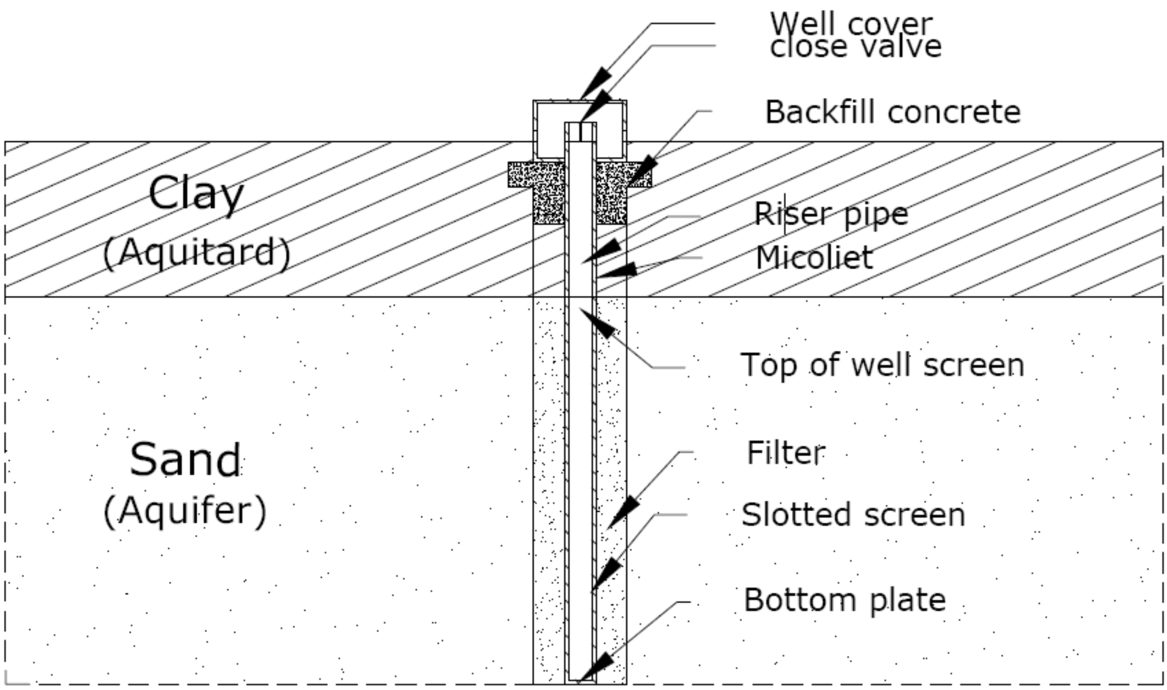


Figure B.3: Well layout

WELL SCREEN

There is a wide range of materials available for well screens and riser pipes such as galvanised iron, stainless steel, fiberglass or plastic material (PVC/HDPE). The site conditions, life time and costs determine the most suitable material. From the mentioned alternatives plastic is the cheapest and fully resistant against groundwater chemicals, but is a relatively weak material. HDPE is one of the plastic variants that has a higher strength and is the most economical choice for a 50 year lifetime requirement.

A HDPE well screen typically has openings in the form of vertical cuts and a bottom plate glued to the screen (USACE, 1992). The size of these opening depends on the filter grain size and should be as large as possible. A large flow area minimises the entrance velocity and correspondingly minimises the transport capacity of foundation material. However, the import of filter material should by prevented, which limits the maximum flow area.

FILTER

A filter is designed according to the retention and permeability criterion and based on the foundation material. The retention criterion states that a filter is stable when the diameter of the filter material ($D_{15,filter}$) is lower than five times the diameter of the aquifer material ($D_{85,base}$) (Stability criterion: $D_{15,base} \leq 5 \cdot D_{85,filter}$). The permeability criterion prevents that water cannot flow through the filter properly and is given by the 15 percentile diameter of both materials (Permeability criterion: $D_{15,filter} \leq 3-5 \cdot D_{15,base}$) (Schiereck, 2005).

A single layer system is preferred as multiple layers are difficult to construct around the well. Given a coarse sand aquifer ($D_{85,base}$ = 0.3 mm), the generally applied filter material is fine gravel ($D_{15,filter}$ = 1.4 mm). The maximum screen slot size is equal to the D_{50} size of the filter material (5 mm).

The representative well radius for the groundwater flow calculation is the screen radius plus half the filter thickness and should be designed for a maximum flow velocity of 0.1 FPS (0.03 m/s) (USACE, 1992). The maximum discharge is calculated with equation 2.7 and result in 703 m^3/d for case Tiel.

$$r_{rep} = r_w + \frac{1}{2} \cdot r_{filter} = 0.2 + \frac{0.2}{2} = 0.3m \qquad u_{max} = \frac{Q_{max}}{\pi r_{rep}^2} = 0.028 m/s \tag{B.1}$$

HEAD LOSS

The flow through the different pipe elements causes a head loss over the well. The total head loss consists of entrance (H_e), friction (H_f) and velocity head (H_v) losses. The groundwater flow model does not incorporate the entrance and friction head losses (USGS, 2005) and increase the maximum occurring hydraulic head. The head losses are added to calculated head and thus accounted for in the safety assessment and overall design.

$$H_w = H_e + H_f + H_v \tag{B.2}$$

The entrance loss consists of the filter and well screen openings, but are relatively small since the velocities are low. The velocity increases over time due to clogging processes, which results in higher losses. The prediction of the long term entrance head loss is difficult and therefor controlled with periodic well maintenance. The friction losses are negligible since the pipe length and velocity is limited. Additionally, the well does not contain pipe bends, which reduce the losses even further. The entrance loss and friction loss combined are smaller enough to be considered negligible.

APPLICATION

The well is constructed with a suction drill technique that allows to create a cylinder without a prefixed pipe. The filter and well pipe are placed within the empty cylinder, which is closed off with a back fill of clay and concrete. The well head is placed on top and includes the valve.

B.2. MAINTENANCE

The inspection, monitoring and maintenance aspects of a relief well are an essential part of the design and ensure the proper functioning of the well. A well with a reduced efficiency could lead to a structural failure during high water events.

The maintenance measures are applied to the different objects in the Object tree (Figure B.1) and the possible failure mechanisms. The USCE manual for well design (USACE, 1992) and the Waterontspanner maintenance plan are the guidelines for these measures. The main failure mechanisms for relief wells are:

- Clogging of the well screen or riser pipe
- Damage to well screen or riser pipe
- Obstruction of the well head
- Failure to close the valve
- Failure of monitoring equipment

WELL

The maintenance objective is to maintain the well efficiency. This efficiency can be monitored with pressure gauges and discharge quantity tests. The efficiency decreases when the hydraulic resistance increases resulting in a lower discharge and head reduction.

The filter and riser pipe are inspected yearly to ensure that the function is fulfilled. The pipe and filter are cleaned preventively every other year. The well head is checked for damages every year and cleaned every two years. Finally, the functioning of the monitoring equipment is checked every year.

Next to the object specific maintenance is the general maintenance that contributes to the proper functioning of the well. The removal of the sand and or other materials next to the well and in the valve. The maintaining of bank slopes and ditches for at least a 1.5 m radius around the well (USACE, 1992).

BASIN

The primary function of the basin is to temporarily retain water that originates from the well. The basin is comprised of the seepage dyke, main dyke and the flooding area between the two dykes.

The flooding area is checked for vegetation and flow obstructing objects in the basin with the yearly general maintenance. The seepage dyke is checked for retaining height, damages and possible settlements every five years. The main dyke falls within the pre-existing maintenance schedule.

B.3. UNIT COSTS

The costs of each element in the Object tree are defined in this section. The unit prices are used to construct the costs analysis and cost functions of Chapter 5. The database of GWWkosten is used to obtain an indication of the costs. An indication of the price range is provided to show the possible variability.

B.3.1. BASIN

Table B.1: Basin unit costs

Basin Phase	Object	Part	Price	Range	Unit
Construction	Seepage dyke	Site preparation and move objects	10	25%	€/m^2
		Sand core (material and application)	15	20%	€/m^3
		Clay cover (material and application)	30	20%	€/m^3
		Acquiring land (Rural farming land)	10	20%	€/m^2
		Terrain finish (equalising, grass)	0.75	25%	€/m^2
	Flooding area	Site preparation and move objects	5	25%	€/m^2
		Acquiring land (Rural farming land)	10	20%	€/m^2
		Terrain finish (equalising, grass)	0.75	25%	€/m^2
Monitoring	Flooding area	Inspection of vegetation and flood area (yearly)	0.1	30%	€/m^2/y
	Seepage dyke	Inspection condition and vegetation (yearly)	0.1	30%	€/m^2/y
Maintenance	Seepage dyke	Maintenance height, form and vegetation (every 5y)	10	30%	€/m^2/5y

B.3.2. WELL

Table B.2: Well unit costs

Well Phase	Object	Parts	Price	Range	Unit
Construction	Well screen	Filter (gravel $D_{50} = 5$ mm)	100	20 %	€/m^3
		Pipe HDPE ($r_w = 0.2$ m)	57	20%	€/m
		Bottom plate HDPE	25	15%	€/s
	Riser pipe	Cover HDPE	300	30%	€/s
		Valve	250	30%	€/s
		Pipe HDPE ($r_w = 0.2$ m)	90	20%	€/m
		Back fill (concrete)	90	20%	€/m^3
		Back fill (Mikolit)	150	5%	€/m^3
	Well placement	Suction drill technique (per meter soil)	100	25%	€/m
Monitoring		Camera inspection (every 10y)	225	30%	€/well/10y
		Pressure sensor (every two wells)	100	30%	€/2wells
		Water level gauge	25	20%	€/well
		Signal check sensors (yearly)	5	20%	€/well/y
		Check sensors (every 2y)	80	5%	€/well/2y
		Check well function (yearly)	120	30%	€/well/y
Maintenance		Cleaning well pipe and head (every 2y)	300	30%	€/well/2y

B.4. LIFE CYCLE FUNCTION

The Life Cycle function elements are presented in Table 5.1 and show how the design, construction, maintenance and removal costs come together. The construction and maintenance costs are based on the design variables (h_{sd}, L_w and w_{sd}) and the unit prices are described in Appendix B.

The construction costs include a 15% addition for the preparation, management and oversight of the project. The design and removal costs are hard to determine precisely for a global design, and thus are based on the construction costs. The design costs are composed of measurements, design and conditioning aspects, which are estimated to be roughly 20% of the construction costs. The removal of the structure at the end of the lifetime is estimated to be 10% of the construction costs. Finally, unforeseen costs are accounted for into the function.

Table B.3: Life Cycle Cost function

Phase	Part	Function
Construction (Cc)	Seepage dyke (Sd)	$w_{sd} = 2 \cdot 2h_{sd} + 1$
		$A_{sand} = 2(h_{sd} - 0.8)^2 + 2(h_{sd} - 0.8)$
		$A_{clay} = 2h^2_{sd} + 2h_{sd} - A_{sand}$
		$C_{cSd} = 30 \cdot A_{clay} + 15 \cdot A_{sand} + 20 \cdot w_{sd} + 0.75 \cdot (2w_{sd} \cdot h_{sd} + 1)$
	Well (W)	$C_{single} = 157(C_d + T_d) + 100 \cdot \pi \cdot 0.12 \cdot T_d + 150(C_d - .75) +$ $45 \cdot \pi \cdot 0.25 \cdot (0.1625 + 0.32) + 575$
		$C_{cW} = C_{single} / L_w$
	Seepage basin (Sb)	$C_{cSb} = (10 + 5 + 0.75) \cdot w_{sb}$
	Preparation (Pm)	$C_{cPm} = 15\% \cdot (C_{cSd} + C_{cW} + C_{cSb})$
	Construction cost	$C_c = C_{cSd} + C_{cW} + C_{cSb} + C_{cPm}$
Maintenance (Cm)	Seepage dyke (Sd)	$C_{mSd} = (1 \cdot 10 + 0.1 \cdot 50)\, w_{sd}$
	Well (W)	$C_{mW} = 3895 / L_w + (0.1 \cdot w_{sb} \cdot 50)$
	Maintenance cost	$C_m = C_{mSd} + C_{mW}$
Design (Cd)	Well, basin and dyke	$C_d = 20\% Cc$ (Construction Costs)
Remove (Cr)	Well, basin and dyke	$C_r = 10\% cc$ (Construction Costs)
Life Cycle Costs	Unforeseen	$Cu = 10\%(C_d + C_c + C_m + C_r)$
	LCC period 1	$LCC1 = 1.1 \cdot (C_d + C_c + C_m + 0.5 \cdot C_r)$
	LCC period 2	$LCC2 = 1.1 \cdot (0.75 \cdot C_d + C_c + C_m + C_r)$

C

Measurement data Tiel

This Appendix provides the measurement data for case Tiel, which is treated in Section 5.3. The data consists of water level measurements and monitoring well measurements. The data is provided for a cross section in the middle of the dyke segment (TG000 -TG003) that is treated in the case study. An overview with the location of the data points is depicted in Figure C.1.

Figure C.1: Overview data points Tiel (Waterschap Rivierenland, 2018)

The hydraulic data is supplied by the waterboard Rivierenland and consists of water level measurements at the surface, the aquitard and the aquifer using monitoring wells. The location of the data points are indicated in the overview above and the data is processed and presented in Figure C.2.

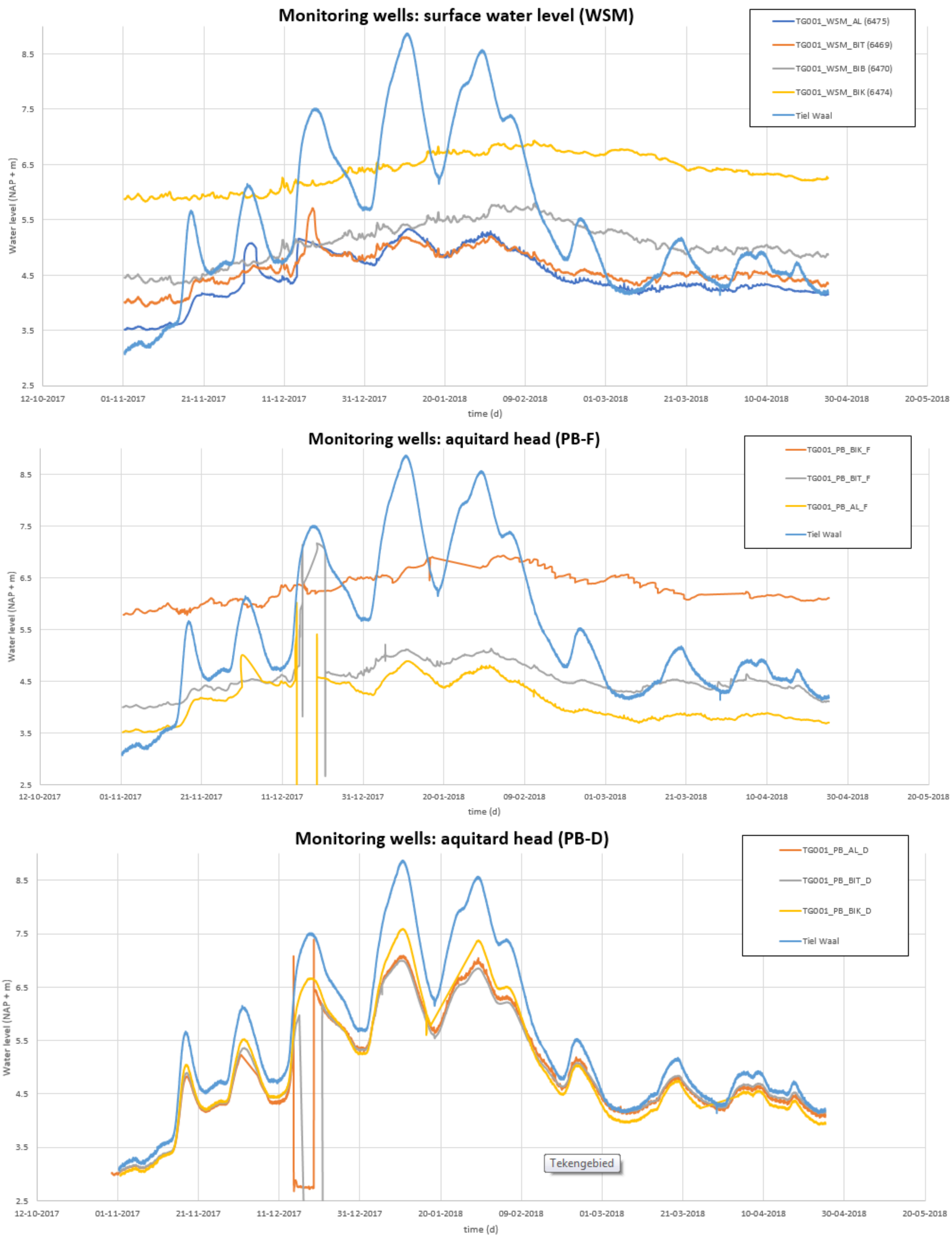


Figure C.2: Monitoring wells TG001 (Waterschap Rivierenland, 2018)

D

PIPING ASSESSMENT TIEL

This Appendix provides the piping assessment for case Tiel, which is treated in Section 5.3.

The failure probability of Piping is determined with a semi-probabilistic method, in accordance with the WBI2017, using the Uplift, Heave and Piping assessment models that are described in Section 1.2.1. The safety factor of each model is translated into a failure probability by means of a calibrated formula (Eq D.1). The characteristic values (5 or 95-percentile) for soil strength parameters are used to obtain the safety factor. The parameters required for the those calculations are presented in Table D.1. The norm reliability (β_{norm}) for section 43-6 is 3.99 or 1/30,000 /year.

$$\beta_u = \frac{ln(\frac{FOS_u}{0.48}) + 0.27\beta_{norm}}{0.46} \qquad \beta_h = \frac{ln(\frac{FOS_h}{0.37}) + 0.30\beta_{norm}}{0.48} \qquad \beta_p = \frac{ln(\frac{FOS_p}{1.04}) + 0.43\beta_{norm}}{0.37} \tag{D.1}$$

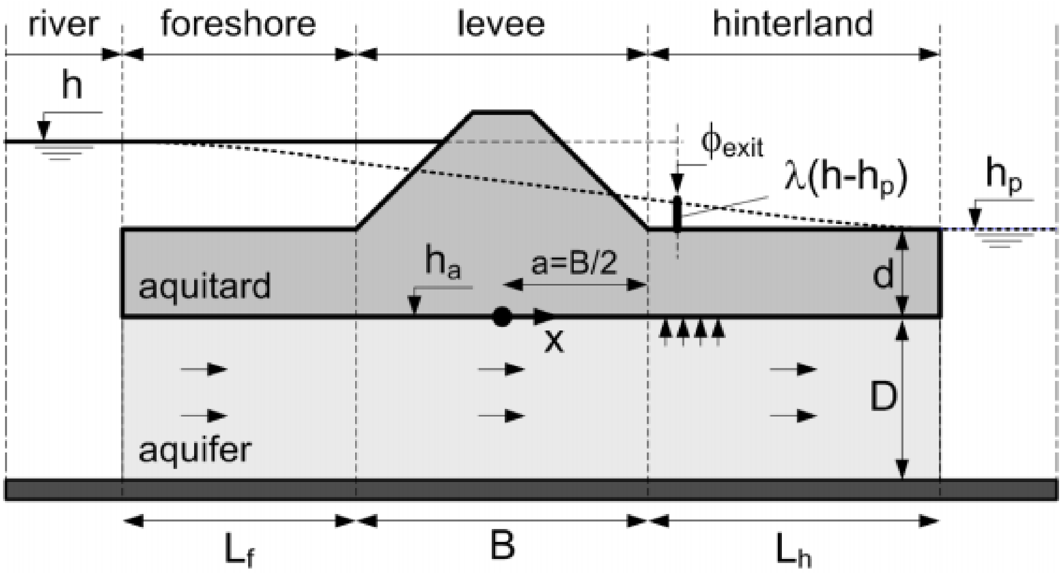


Figure D.1: Definitions relevant for limit state (Schweckendiek, 2014)

Table D.1: Model parameters for Uplift, Heave and Piping (figure symbols (Schweckendiek, 2014))

Parameter	Description	Value	Parameter	Description	Value
h_r	river water level	11.5 m	λ_h	leakage length	124 m
h_p	head at landside	4.2 m	λ	damping (response) factor	0.69
L_f	length of effective foreshore	20 m	ϕ_{exit}	head at exit point	9.27 m
B	width dyke	30 m	i	gradient over the blanket	0.66
L_h	length hinterland	70 m	$i_{c,h}$	critical heave gradient	0.70
L	seepage length	52 m	$K_{h,aquifer}$	specific conductivity aquifer	$6E^{-5}$ m/s
d	blanket thickness	7.6 m	x_{exit}	distance exit point to dyke centre	17 m
D	aquifer thickness	20 m	$K_{h,cover}$	specific conductivity cover layer	$6E^{-7}$ m/s
ΔH	head difference	7.3 m	d_{70}	70%-fractile grain size distribution	$2.2E^{-4}$ m
$\gamma_{s,aq}$	weight solids aquifer	26.5 kN/m^3	d_{70m}	reference value for d_{70}	$2.08E^{-4}$ m
$\gamma_{sat,bl}$	saturated weight blanket	14 kN/m^3	θ	bedding angle	37 °
γ_w	volumetric weight of water	10 kN/m^3	η	White (drag) coefficient	0.25
g	gravitational constant	9.81 m^2/s	υ	kinematic viscosity of water	$1.33E^{-6}$ m^2/s

The WBI2017 schematisation manual provides values for θ, η, d_{70m} and $\gamma_{sat,aq}$. The d_{70} and D parameters are obtained from the D-soil subsoil schematisation database. The permeability of the aquifer is determined from the data of the monitoring wells (Table D.2b) and the cover layer permeability is estimated based on the soil type description (Fitts, 2002).

The critical heave gradient is taken as 0.7, at which point sand boils start to occur according to experimental data (USACE, 1992). The exit point is assumed to be two meter behind the inner dyke toe resulting in an x_{exit} of 17 m. The remaining parameters λ_h, λ, ϕ_{exit} and i are determined below.

The damping factor (λ) can be estimated in simplified situations or determined with groundwater flow models, monitoring or expert judgement in more complex cases. The dampening factor is estimated with equation D.2 and equal to 0.7. The data from the monitoring wells show that the estimated dampening factor at the exit point is inline with the damping factors of the two surrounding measurement points for the river peak of 29-01-18.

$$\lambda = \frac{\lambda_h}{L_f + B + \lambda_h} exp^{(B/2 - x_{exit})/\lambda_h} = 0.69 \tag{D.2}$$

The water pressure in the aquifer drops between the two boundaries of the river water level (h) and the polder water level (h_p). The hydraulic pressure head (ϕ_{exit}) in the aquifer and the gradient (i) over the dyke can be expressed by the following equations:

$$\phi_{exit} = h_p + \lambda(h - h_p) = 9.27m \qquad i = (\phi_{exit} - h_p)/d = 0.67 \tag{D.3}$$

The measurements data is used to estimate the average horizontal conductivity of the aquifer with the failing head test (Eq. D.4) based on Darcy's law ((Fitts, 2002)). The results are depicted in Table D.2b and vary within the range of 2 to 8 m/d with an average of 5 m/d.

$$K_h = \frac{a}{A} \cdot \frac{L}{t_1 - t_0} \cdot \ln\left(\frac{\Delta h_0}{\Delta h_1}\right) = 1 \cdot \frac{70}{1/3} \cdot \ln\left(\frac{\Delta h_0}{\Delta h_1}\right) \ \ [m/d] \tag{D.4}$$

Table D.2: Determining properties with monitoring well measurements Tiel

(a) Dampening factor (λ) aquifer

Point	River	BIK	x_{exit}	BIT	AL
h [NAP m]	8.53	7.36	(7.21)	6.86	6.98
λ [-]	1.00	0.73	(0.69)	0.61	0.64
x [m]	0	35	52	105	125
Date: 29/01/18 08:00					
Polder level: NAP +4.20m)					

(b) Hydraulic conductivity aquifer (K_h)

Date	Water level BIK	BIT	Δh [m]	K_h [m/d]
29/01/18 0:00	7.364	6.843	0.520	
29/01/18 8:00	7.357	6.856	0.501	7.40
29/01/18 16:00	7.355	6.843	0.512	4.12
30/01/18 0:00	7.315	6.815	0.500	4.72
30/01/18 8:00	7.266	6.773	0.493	2.48
30/01/18 16:00	7.195	6.725	0.471	6.66
Length BIK-BIT	70 m		$K_{h,avg}$	5.08

The characteristic values (5 or 95-percentile) are derived from the stochastic variables with their distribution and variation coefficients, which are described by the schematisation WBI2017 manual (Deltares, 2017) and depicted in the table below. The last row of Table D.3 shows the resulting characteristic values that are used in the assessment.

Table D.3: Stochastic variables for semi-probabilistic assessment

Parameter	k_h	D	d	λ	L_f	h_p	$\gamma_{sat,bl}$	$i_{c,h}$	L	d_{70}	m_p
Distribution	lognormal	lognormal	lognormal	lognormal	lognormal	normal	normal	lognormal	lognormal	lognormal	normal
V	0.5	0.15	0.1	0.1	0.15			0.1	0.1	0.12	
σ						0.1	0.05				0.12
μ	5.79E-5	20	7.60	0.69	20	4.2	14	0.70	52	2.2E-4	1
ζ	0.47	0.15	0.10	0.10	0.15			0.10	0.10	0.12	
λ	-9.87	2.98	2.02	-0.37	2.98	-0.36	3.73	-8.43	3.95	-8.43	
Limit	95%	95%	5%	95%	5%	5%	5%	5%	5%	5%	5%
Value	1.13E-4	25.3	6.42	0.81	15.48	4.04	13.92	0.59	43.91	1.79E-4	0.8

The factor of safety for Uplift, Heave and Piping is calculated with equations 1.3, 1.4 and 1.5, respectively, and the results are depicted in Table D.4. The critical head difference for Uplift ($\phi_{c,u}$) and Piping (ΔH_c) is 2.51 m and 2.66 m, respectively. The required failure probability for section 43-6 is 1/17,375,000 per year (β_{req}= 5.3), which is larger than the combined failure probability of 1/570 per year ($\beta = 2.92$) and thus does not satisfy the requirements.

Table D.4: Results Piping assessment

Failure mechanism	Factor of safety semi-probabilistic	(deterministic)	Reliability index (-)	Probability (1/year)
Uplift	0.41	(0.59)	-1.69	0.95
Heave	0.62	(1.04)	-0.20	0.58
Piping	0.48	(1.08)	2.73	0.00031
Combined	$(P_f = P_u \cdot P_h \cdot P_p)$		2.92	1.7E-3

E

Calibrated groundwater flow model Tiel

This Appendix provides the groundwater flow calculations results for the case of Tiel and a summary of the model setup. The results present a situation with and without the preferred Pipingontspanner configuration and includes a safety assessment for Uplift and Heave to show the effectiveness of the Pipingontspanner. The setup of the groundwater flow model is fully described in Section 2.3 and Section 3.3. The schematisation of the Pipingontspanner model is depicted in ModelschemTiel (Figure E.1).

Geometry:

- The model domain is 200 x 100 metres.
- The geometry is uniform along the y-axis.
- The situation is divided into 3 layers: top layer, aquitard and aquifer.
- The top of the main and seepage dyke are NAP +12 m and NAP +2.3 m, respectively.
- The aquitard and aquifer are 7.6 and 20 m thick, respectively.

Layer properties:

- The layer properties are given in Table 5.4 and Figure E.1. The aquifer conductivity originates from the measurement data of Tiel and is determined in Table D.2b. The remaining parameters are deducted from the soil type with the guidelines for drainage measures (Niemeijer et al., 2017) and the groundwater pocketbook (Bot, 2011, Table 2.1 and 2.5).
- Layer 1 is divided into 2 parts (dyke and empty space), which are modelled using the conductivity properties of the soil.

Boundary conditions:

- The outer edge of the model is a no-flow boundary.
- The river boundary is river flood wave on top of the mean high water level (NAP +6.8 m).
- The polder boundary is a constant head of NAP +4.2 m at x = 372 m and is simulated using a head dependent flux module.
- The wells are located at the x= 42 m and situated in one line.

Well:

- The well is incorporated into the model using the hydraulic properties of a cell. The cell is given the equivalent vertical hydraulic conductivity of the well. The conductivity of the well ($K_{v,well}$) depends on the permeability of the aquifer and is determined with equation 2.7. The well conductivity for case Tiel is 480 m/d for a 1.25x1.25 m cell.

Calibration:

- The groundwater flow model is calibrated with the water level gradients from the monitoring well measurements (Table D.2a), which are based on a river peak in the data set (29-01-18). The calibrating parameters are the river bed resistance (C) and the conductivity parameters ($K_{h,v}$), which control how fast the water enters the aquifer and how the gradient develops over the dyke, respectively.
- The closest results are obtained with the river bed resistance equal to 2 days and the permeability of the dyke and aquitard equal to 1E-2 m/d. The aquifer conductivity remained unchanged at 5 m/d. A comparison between the model and measurement gradients (Figure 5.13) shows that only the surface gradient deviates from the measurements.

Safety assessment:

- The safety assessment is performed according to the semi-probabilistic approach in Section 3.1 with the calibration formula 3.1 and the factor of safety equations 3.2 and 3.3.
- The alternatives are calculated with calibrated groundwater flow model and the model parameters after which the safety assessment is performed with the characteristic values found in Table D.3.
- The 95% characteristic value for the dampening factor is included with a factor of 1.075 times the aquifer head in the assessment calculations. This value is determined with the ratio of the estimated assessment level divided by the calibrated water level (10.1/9.4 = 1.075).

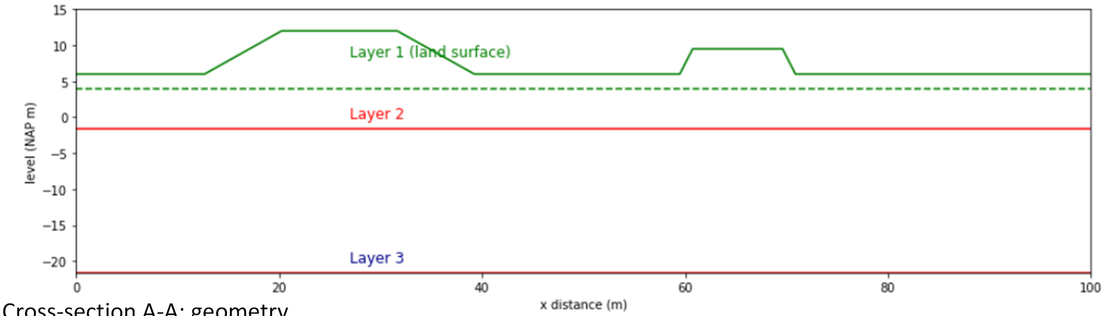
The calculation results for the groundwater flow model are depicted in Modelmap 0 and 1 for the base situation and the preferred alternative (L_w = 30 m, w_{sb} = 22.5 m), respectively. The Uplift and Heave assessment of the base situation is depicted in Modelmap 0 and is in accordance with the piping assessment (Table 5.3). The safety assessment for the preferred alternative is depicted in Figure E.4 and shows the improved safety factor for the Uplift and Heave mechanism. The failure probability of the preferred alternative is 5.5E-08/year, which satisfies the requirements.

The aquifer seepage discharge of the base situation and the preferred alternative is displayed in Figure E.4. The seepage discharge (q_x) is increased from 2.2 to 3.5 m^2/d compared with the base situation, which results in an additional 3.5 m^3 per meter dyke per flood wave event. This additional seepage is negligible compared to the volume that is imported in a base situation (35 m^3/100 days).

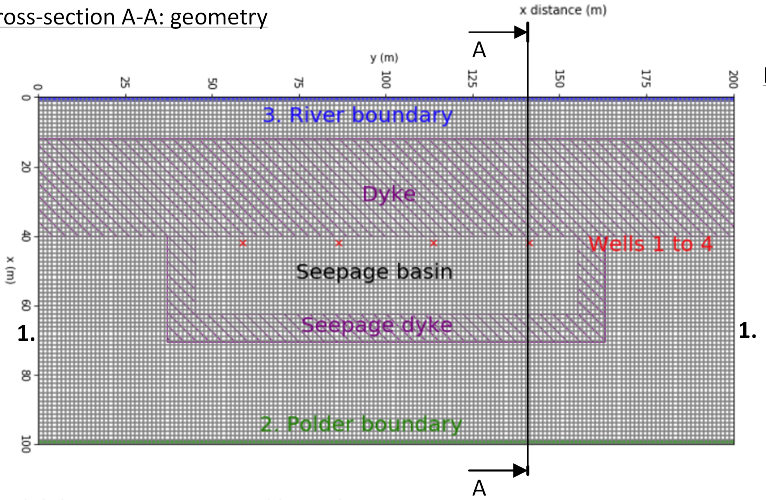
The following notes can be made for groundwater flow results of Modelmap 1:

- The highest phreatic level inside the seepage basin occurs at 20 days, which is far after the critical Uplift and Heave moment. The critical moment occurs when the design flood wave reaches the maximum water level (t= 9 days), at which the basin water level is 0.2 m. It appears that the water in the seepage basin is not the largest contributor to the increase of safety.
- The head reduction caused by the wells is visible in cross-section B-B and is characterised by a local drop in head around the well positions. The maximum water level in the basin is 0.8 m and thus only a small seepage dyke of 2.3 m is required to retain the water.
- The seepage discharge in the aquifer is increased slightly from 2.2 to 3.4 m/d compared with the base situation, but the additional seepage volume is limited.

Modelschem: Situation Tiel with POS (grid 1.25 x 1.25)



Cross-section A-A: geometry



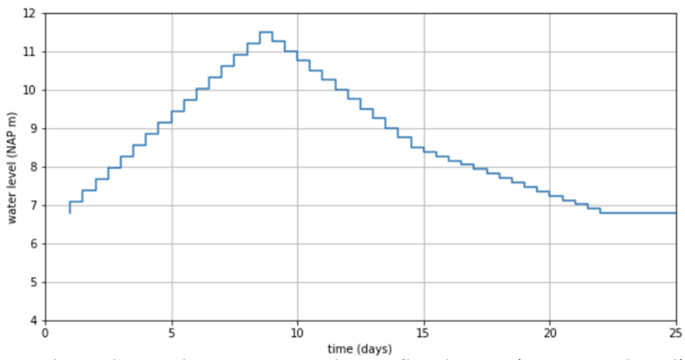
Model domain: top view and boundaries

Boundary conditions:

1. No-flow boundary
2. Head dependent constant boundary h = NAP + 4.2 m at x=L. L = 3λ = 372 m(Influence length)
3. Head dependent river boundary

Parametrisation layer model Tiel										
Property	Layer type	Top [m NAP]	Bottom [m NAP]	Thickness [m]	kh [m/d]	T [m^2/d]	kv [m/d]	C [d]	Ss [1/m]	Sy m/m
Layer 1 (part a)	dyke body	12	4	8	0.05	0.4	0.05	160	-	0.05
Layer 1 (part b)	empty space	12	4	8	86400	691200	86400	9.3E-05	-	0.99
Layer 2	cover layer	4	-1.7	5.7	0.05	0.285	0.05	152	0.001	0.05
Layer 3	aquifer	-1.7	-21.7	20	5	100	1.5	13.33	0.0001	0.2

Layer properties: overview table and cell assignment



River boundary: schematisation: design flood wave (River Waal, Tiel)

Parameter list:
- L_w= 27.5 m
- W_{sb}= 22.5 m
- K_{vwell} =449.6 m/d
- $H_{r,max}$ = NAP +11.5 m at t=9 d
- x_{well} = 42 m
- h_{sd} = 0.58 + 1.5= 2.08 m

Figure E.1: Schematisation summary

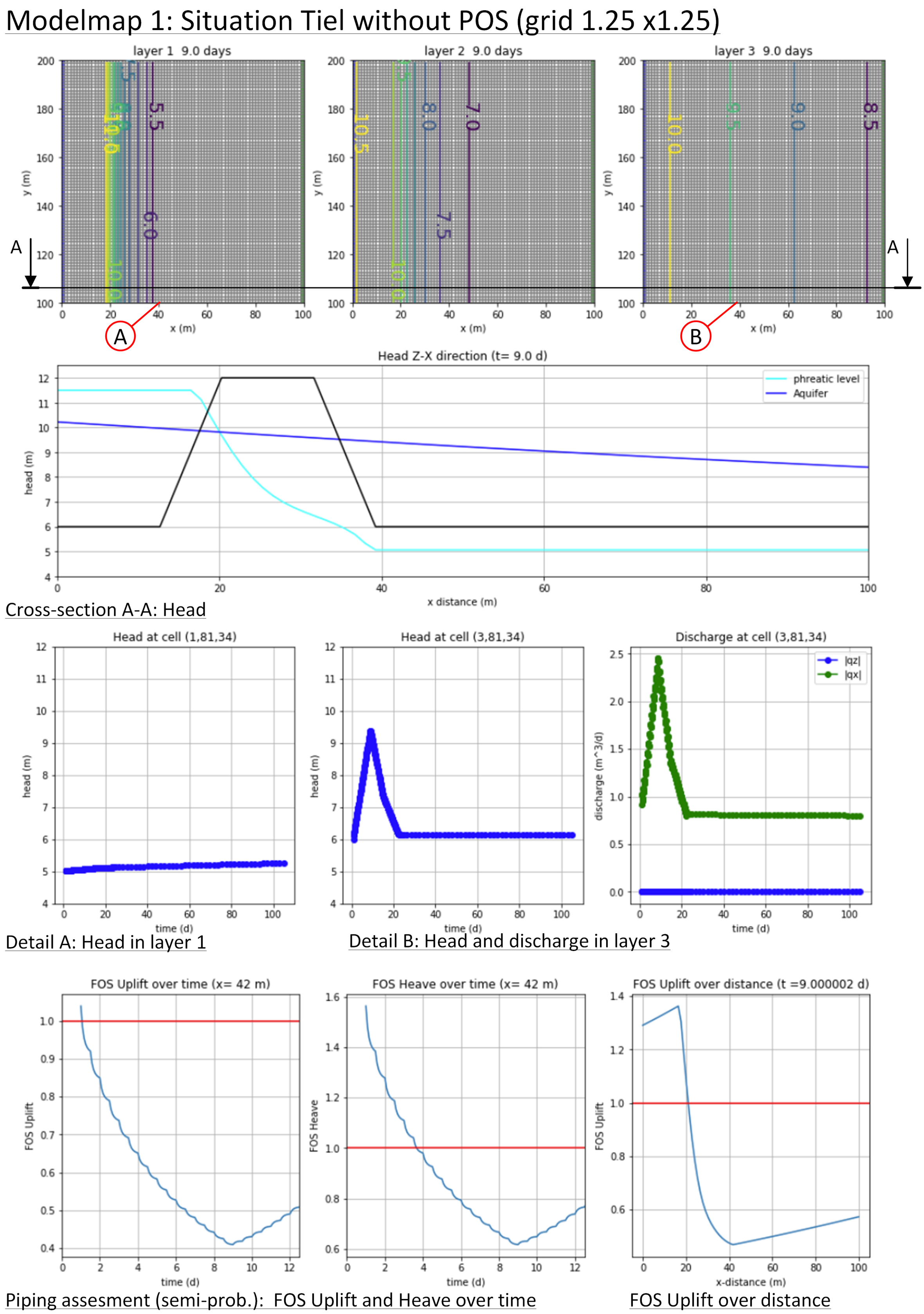


Figure E.2: Modelmap1: Modflow results Tiel without Pipingontspanner

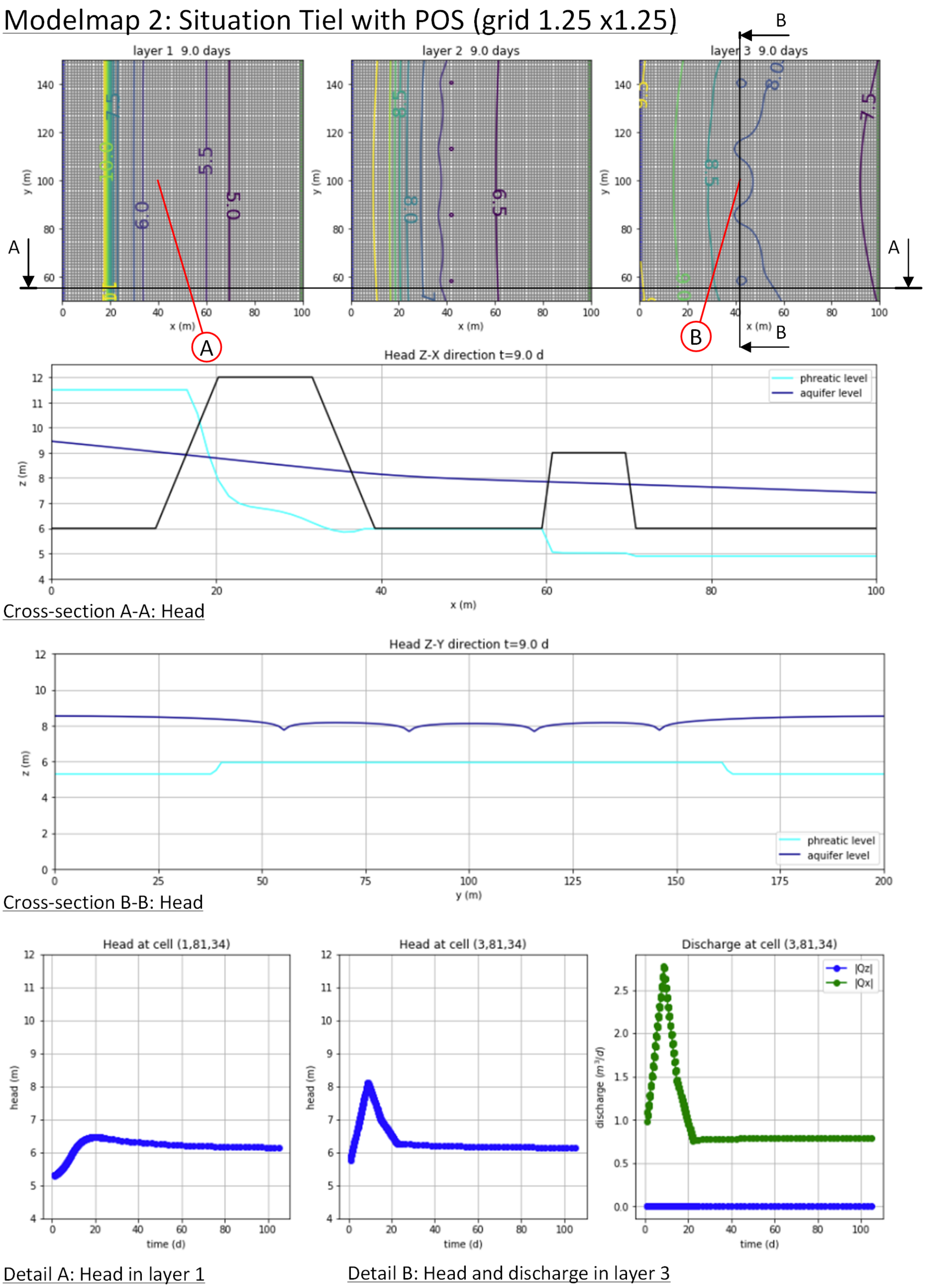
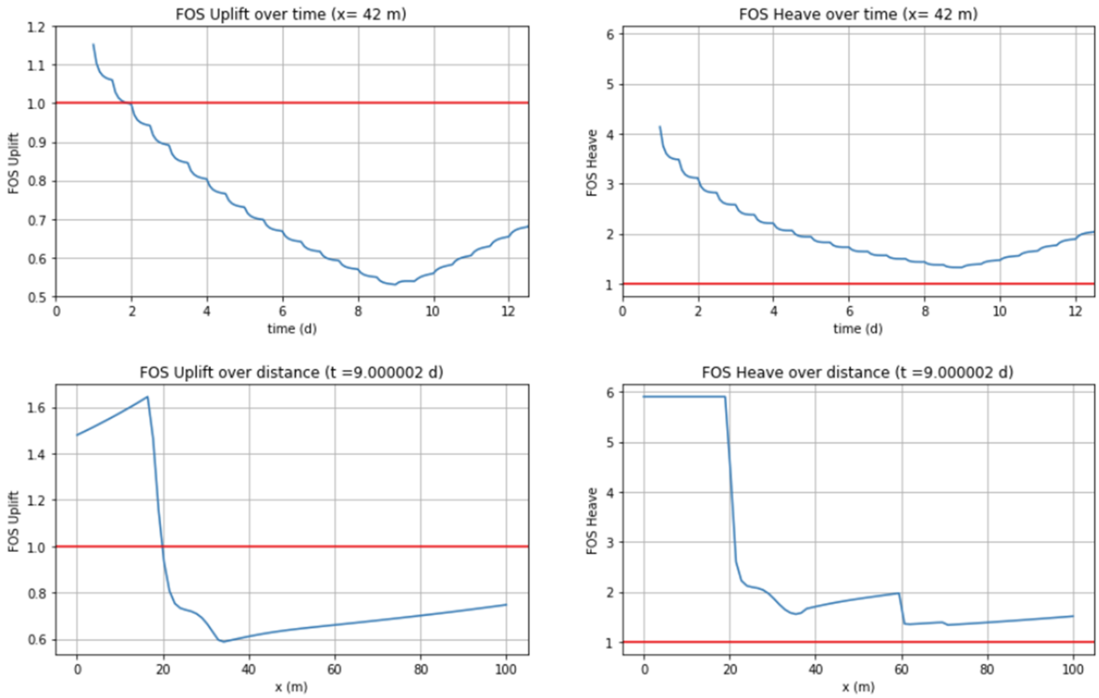
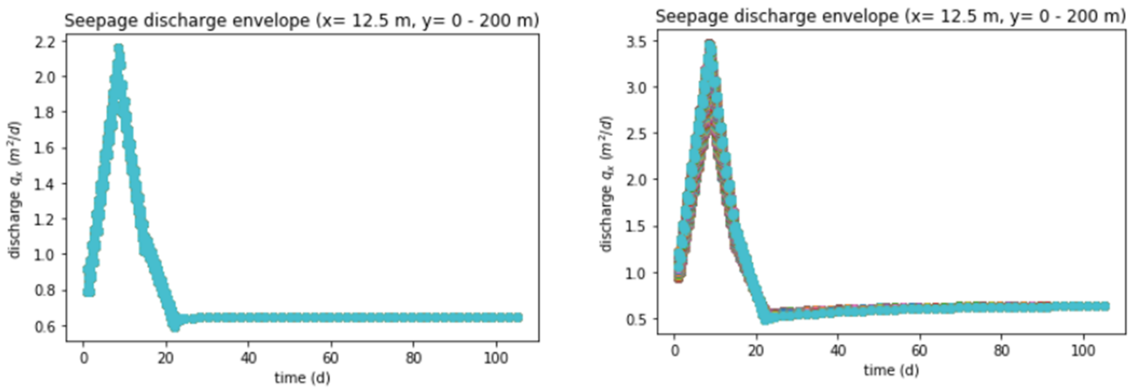


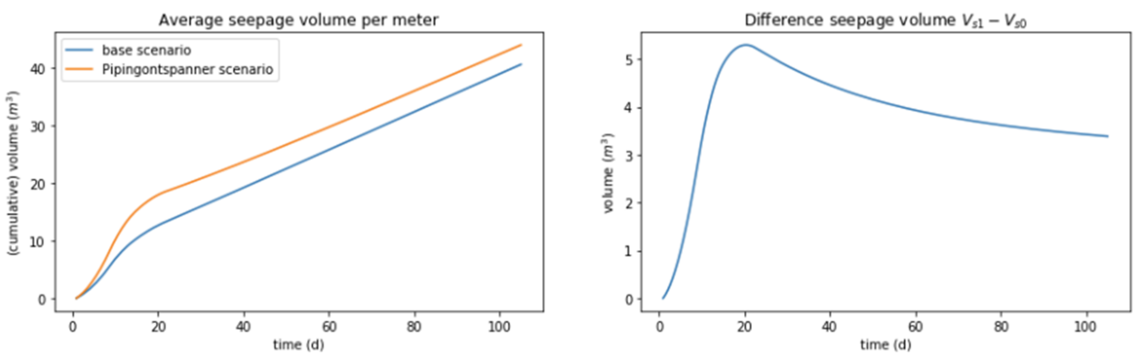
Figure E.3: Modelmap2a: Modflow results with Pipingontspanner (preferred alternative)



Uplift and Heave assessment over time and distance



Seepage discharge from the river (base (l) and Pipingontspanner situation (r))



Seepage volume from the river (base and Pipingontspanner situation)

Figure E.4: Modelmap2b: Modflow results with Pipingontspanner (preferred alternative)

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