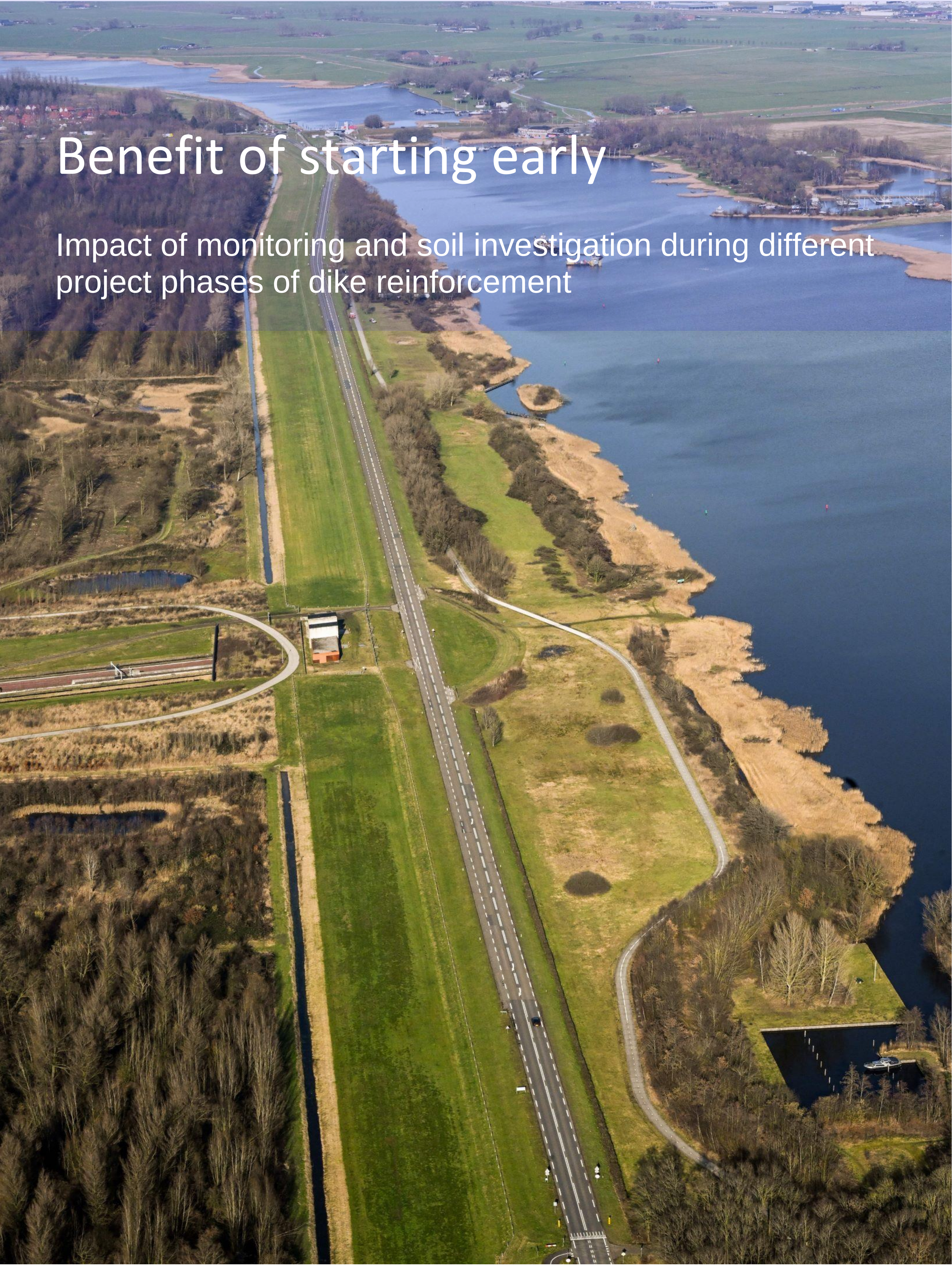

Benefit of starting early

Impact of monitoring and soil investigation during different project phases of dike reinforcement

TU Delft

FUGRO

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Impact of monitoring and soil investigation on different project phases of dike reinforcement

By

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In partial fulfilment of the requirements for the degree of

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Picture cover page: Room for the River at Drontermeerdijk
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Preface

This thesis is the result of a project proposal from Fugro to investigate ways to improve dike reinforcement. This work is part of my master Hydraulic Engineering at the Delft University of Technology, with Flood Risk as specialization. I am grateful for the opportunity given to me to contribute to the protection of Dutch inhabitants from flooding.

I have enjoyed my time working with Fugro in Utrecht. It was inspiring to work with colleagues who are motivated in their profession and interested in new developments in this field. I have enjoyed the walks around the business park and having lunch together. I want to thank my colleagues for their experience and shared knowledge, which have improved the quality of this research.

Furthermore I would like to thank my graduation committee, Matthijs Kok, Martin van der Meer, Robert Lanzafame and Phil Vardon, for their time and guidance. Matthijs for his optimism during the research and helpful suggestions on how to improve the thesis. Martin, for his insight on dike reinforcement projects and his commitment to help me improve my work. Robert, for the many ideas provided during the meetings which have helped shaped the results. And Phil for his help with the structure of the report and making sure the writing is coherent. All the help is much appreciated.

I also want to thank my family and friends for making my time as a student much more pleasant. I was (am) lucky to have my family support me during my time as a student. I want to thank my friends for the distraction from work and the pleasant memories we share. And finally I want to thank Vera van Meeuwen, for her love and endless patience with me.

Michael G. van der Lans
Delft, april 2020

Summary

To fulfil the new safety standards of the Netherlands against flooding in the future, a great deal of work needs to be performed to prepare the flood defences system. Around 50 kilometres of dike reinforcement will have to be realised annually, but currently only 25 kilometres per year are realised (Jorissen, 2018). For this reason the HWBP (Hoogwaterbeschermingsprogramma) is researching methods which can improve the efficiency of dike reinforcement projects.

This thesis was initiated to analyse what could be done in the transition period between registration of a dike reinforcement project at the HWBP programme and the start of the reconnaissance phase of the project, as it is potentially useful to utilize this time period. This has turned into an analysis of applying monitoring and soil investigation techniques in the different project phases. More specifically, the impact of implementing monitoring and soil investigation earlier was analysed, because information collected before or during the transition period can be used for the benefit of the dike reinforcement projects. The primary research question was therefore: *What will be the benefits to implement monitoring and soil investigation methods early in dike reinforcement projects?* The analysis was performed using a theoretical case based on the dike trajectory located near Amerongen.

The results of the analysis can be divided into two categories. First, the impact of using monitoring and soil investigation to reduce the project costs needed for dike reinforcement by identifying the strong sections of the dike trajectory. Secondly, consideration of the risk of weak sections in the dike trajectory and how monitoring and soil investigation can be used to reduce the calculated risk of dike failure. The duty of care in this thesis implies the legal responsibility to a standard of reasonable care and safety for the administrator of the dike.

To determine the effect on the project costs, a decision on the design was linked to every phase of the project. For example, in the initiation phase the project scope is defined and in the reconnaissance phase a preferred alternative is chosen. Possible measurement results of monitoring and soil investigation were determined using local data and compared to the representative cross-section of the dike trajectory. Based on this comparison, the expected project cost reduction for each project phase was determined. Furthermore by combining possible measurement results with the required measuring methods to verify the subsoil scenario, the estimated investment costs were calculated. The result is an estimation of the possible costs and benefits for monitoring and soil investigation.

A strategy was developed to cope with uncertainty in the subsoil for the different project phases and to serve as recommendations on how to assess whether enough information has been collected for the given project phase to make a decision on the design. The recommendations can be summarized as follows: scenarios of the subsoil must be determined using the available data which outline which sections are more resistant to the failure mechanism than was first expected. The probability of the scenario occurring and the margin of expected project reduction for the project phase are used to analyse whether the scenario can be eliminated or if more information must be collected to verify if the scenario represents reality. The effect of starting monitoring and soil investigation in different project phases is included.

For the second part of the research the effect of early soil investigation and monitoring on the duty of care for the dike trajectory was analysed. To analyse the safety of the dike trajectory the risk of dike failure due to piping is determined, for which risk is defined as the probability of failure times the consequences of failure. For this analysis, risk was calculated by multiplying the probability of failure due piping with the damage caused by dike failure for the design water level in euros.

Two aspects of the duty of care for a dike trajectory were considered in this analysis. The first aspect is the possibility of a section present in the dike with weaker parameters than first estimated. This was simulated using the presence of a sand lens under the dike in connection with the water in the river. The analysis showed that the probe interval of 100m which is commonly used can still cause a considerable risk for the given case, and demonstrated that the possibility of such a weak spot occurring requires a smaller probe interval.

The effect of monitoring and soil investigation on the calculated risk was considered. Usually the information used to test the dike trajectory during the dike inspection is not enough to design the dike reinforcement, and can show a very conservative view of the strength of the dike. If the dike trajectories are as vulnerable as calculated during the safety assessment, then the question is why no measures are taken to lower the calculated risk. Monitoring and soil investigation lead to a more accurate representation of the risk of dike failure with the potential benefit of a reduced calculated risk of dike failure for the trajectory. If the entire dike trajectory is more resistant to piping than first expected, the calculated risk is decreased. Another benefit in terms of calculated risk is the identification of weak spots in the dike trajectory and managing of risk by additional maintenance or inspections of these sections or by the application of emergency measures.

For the applied case dike trajectory at Amerongen the expected reduction in project costs is 19,4% of the initially estimated project costs if the measurements were performed or started in the initiation phase. In comparison, the expected project cost reduction for the common project structure is 13,3% for starting in the reconnaissance phase. The investment costs due to monitoring and soil investigation was 4,6% of the estimated project cost when starting early and 4,1% for the common project structure. In conclusion, by investing 0,5% of the estimated project costs on starting monitoring and soil investigation early in the dike reinforcement project a 6,1% reduction in the project costs is expected. It can therefore be concluded that investing in early monitoring and soil investigation is cost-effective.

This research is based on a theoretical case, so it is also recommended to verify the results with different realised projects. Unfortunately, inquiries directed at the HWBP determined that historical data was not readily available. However, already recommendations can be made for the project structure used by the HWBP. Facilitating investment into monitoring and additional in the initiation phase will lead to a reduction of expected project costs. Criteria can be added to the application process that data collection must already have been started before the project is applied to the HWBP programme to ensure the methods are implemented for relevant cases. For future analysis an early start of registering the data in new dike reinforcement projects will be beneficial. These results can be collected for evaluation at the end of the project.

Content

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1. Introduction

The management of water has deep roots in Dutch history. The water boards are the oldest democratic institutions in the Netherlands, of which some trace back to the Middle Ages. It forms the basis for the polder model, the policy for consensus-based economic and social decision making. The term found its origins from the fact that different societies living in the same polder were forced to cooperate as without agreement on shared responsibilities for maintenance of the levees all would suffer the consequences. And the tasks for water management of the water boards was even made part of the constitution in 1848.

There is good reason that water plays such a big role in Dutch society. Several branches of three mayor rivers in Europe, the Rhine, the Meuse and the Scheldt enter the sea through this country. Furthermore around 26 percent of the country is located below sea level. These two factors cause around two/thirds of the country to be vulnerable to flooding, which are coincidentally also one of the most densely populated areas on the planet.

To defend against the water, long systems of flood defences have been built to keep out the water. Natural defences such as sand dunes are maintained, while dikes and floodgates were built to keep out the seawater. Dikes were built along the rivers and channelled through their beds while belt systems of drainage ditches and pumping stations kept the hinterland dry. Now, a total of 3800 kilometres of primary flood defences are present in the Netherlands.

However, due to new calculation methods and insights in risk management a new approach has been implemented in 2017. Instead of evaluating the safety of a defence structure by its ability to withstand a certain design water level, the safety is now tested for the total probability of failure for the structure. This means 714 km of primary flood defences and 264 retaining structures have to be reinforced before the end of 2019 (Rijkswaterstaat, 2013).

1.1 Problem context

To fulfil the new safety standards and to guarantee the safety of the Netherlands for flooding in the future, a lot of work needs to be done on the flood defence system. Yearly around 50 km of dikes have to be reinforced per year to fulfil the requirements of the high water protection programme. In reality however, progress has been slower than expected, with only 25 km per year being realised (Jorissen, 2018). The goals of the Dutch Flood Prevention Programme are to find an approach which is smarter, faster and cheaper in redesigning and reinforcing the structures.

An advisory task force was appointed to find out which steps can be taken to realize the ambitions stated above (Rijkswaterstaat, 2013). One of the main possibilities to save time during the project which was suggested for before the start of the reconnaissance phase in the project. It was advised to start the analysis what the alternatives for the dike reinforcement are earlier. What happens now, is that when the water board finds out one of the dikes which they maintain is not safe, the dike goes on a waiting list for dike reinforcement. This waiting list is managed by Rijkswaterstaat, which provides about 50% of the funding for the project (Groenewoud, 2016). A combined budget of the water boards supply the remaining 40% of the funding.

The dike can spent about 10 to 15 years on this list before funding is received to start the project. What follows is that there is only a short time period left for the assessment and redesign of the dike. The total duration of the project is then around 6 years (Waterschap Rivierenland, 2019). And even then time is needed to complete soil investigation and/or monitoring, which means the available design period in which the new information can be applied is even shorter. This can lead to unnecessarily large and expensive dike reinforcements.

The idea behind this thesis is to make use of this preliminary phase of the dike reinforcement project to perform monitoring and soil investigation. If an estimate can be made on what the effect on the final design will be to have more measurements available earlier in the project, a better decision can be made on what information is needed during which project phases to come to the optimal dike design. Also the accuracy in the risk assessment used to determine the best method to provide the duty of care for the dike and its hinterland is increased earlier. Therefore the goal of this thesis is to set up an approach to estimate the uncertainties in the model and to determine in which project phase which information is needed to be most beneficial for the design of the dike. This can be done for different cost components, such as project cost, CO2 emission, land use etc. The design cost components will be defined in a different chapter.

1.2 Research objectives and questions

Now that the problem statement has been defined, the main goal of the report can be formulated. This goal is to analyse what the benefits will be for early monitoring and soil investigation during the project period of the dike reinforcement. The main research question becomes:

> ***What will be the benefits to implement monitoring and soil investigation methods early in dike reinforcement projects?***

Since the main question is too broad to investigate properly, the main question is divided into multiple sub-questions. These will serve as guidance through the research in what the result will be to be able to find a proper answer to the main question. The sub-questions are explained below

First of all, the type of benefits that are gained need to be explored. The benefits can be found in the reduction of project costs for the dike reinforcement, but can also for example be in annual risk reduction for the dike trajectory, or social or environmental impact. The type of results needed also determines the approach to the research, so what type of result is used to determine the benefit of early monitoring and soil investigation needs to be defined. So the sub-question becomes,

1. *What type of benefits can be found by implementing a monitoring plan and soil investigation of the dike trajectory?*

Based on the type of benefits that will be analysed, the scale of the benefits can be analysed for the different phases of the project period. The project period has been divided into 4 different phases, which are the initiation phase, the reconnaissance phase, the elaboration phase and the realisation phase. These phases all represents a different aspect of the design process which will have to be handled before the final design can be delivered. The effect will be analysed using a theoretical example case. Therefore the question becomes how large the benefit becomes for each stage.

2. *In the theoretical case, what is the benefit for each project phase of the project period?*

As part of determining whether early soil research will be a good investment, the comparison must be made between the investment costs for the implemented monitoring and soil investigation methods and the expected profit from implementing the methods. This will be done for all the project phases of the dike trajectory. So the question to answer is:

3. *How large are the benefits of investing in early monitoring and soil investigation compared to the required investment costs?*

The previous questions only focus on what the benefit for the design will be if beneficial results are found. However, it is not guaranteed that only positive results for the design will be found. This does not have to be negative for the project however, as it does provide better insight in the dike trajectory and the actual probability of failure for the dike. So for negative results, the question becomes:

4. *What is the effect for the dike reinforcement project when it is found the dike is more vulnerable than first anticipated?*

Finally it is also worthwhile to analyse what the effect of early monitoring and soil investigation can be on risk perspective for the dike. If the safety assessment of the dike trajectory indicates a highly unsafe dike and no emergency measures are implemented during the project period, the taken risk during the project is apparently high. Monitoring and soil investigation can be used to gain insight into the actual risk of dike failure, and possibly reduce the calculated risk of dike failure at the location. So the final sub-question is:

5. *What is the effect of early monitoring and soil investigation on the* calculated *risk of dike failure for the case?*

1.3 Methodology

This thesis focuses on what the possible benefits for the design process will be if certain information was collected earlier in the design process. The project phases are taken as indication of what is performed in what stage of the project. The analysis shows the possible the impact is in the project phases when data acquired by monitoring and soil research is performed earlier in the project.

To start, relevant information is collected from literature. The literature research provides insight in the relevant failure mechanisms, the design standards in the Netherlands and the uncertainty which is common in the design process. This information can then be applied to a model to be used in the research. The next part is therefore setting up the model based on a theoretical case and performing a safety assessment of the model. This is done for three failure mechanisms, which are overtopping, piping and macro instability. The analysis is continued with the dominant failure mechanism.

With the case defined and the dominant failure mechanism determined, the thesis splits in two different aspects of the benefit of monitoring early. These two aspects are the impact on the expected project costs and the impact on the risk related to the duty of care for the dike trajectory.

The first aspect focuses on design optimization and what the profit will be for design choices through the design process if more information is available at the time of the design decision. The other part focuses on the duty of care and how monitoring and soil investigation can be applied to more accurately assess the risk and identify the weak spots in the dike trajectory. The cost optimization focuses more on finding the strong sections of the dike where less dike reinforcement is needed, while the duty of care for the dike focuses more on finding the weak spots in the dike trajectory and how to assess the risk of dike failure for the dike. These two aspects will first be treated separately before the results will be combined and discussed.

As situations are needed which are different from the base model which was provided for the case, different scenarios were determined which had deviations from the given representative cross-section. Most scenarios have a different mean value for a relevant parameter for the failure mechanism. The difference in mean value was determined based on available data on the subsoil at the location of the dike. The difference compared to the base scenarios is described for each scenario.

Four characteristic design stages will be analysed to determine what the impact on reducing the uncertainty in the design process will be. These are the initiation phase, the reconnaissance phase, the elaboration phase and the realisation phase. The general approach for the analysis per design phase is described below.

For the initiation phase, the decision that reinforcement is needed will be analysed. This is because further research might lead to the conclusion that the reinforcement is not needed after all. The optimal level of testing can be a research topic, but is not evaluated in this thesis. The result of the dike inspection is assumed to be the representative cross-section. The effect of monitoring and soil investigation on the project scope will be evaluated for this phase.

The main focus for the reconnaissance phase is in the choice of preferred alternative. For this phase which alternative will be chosen if different data became available earlier in the project, and will this alternative be the same as without this data? A comparison will be made between project costs for the different scenarios and the base scenario.

Next the elaboration phase will be discussed. Here the specific dimensions of the preferred alternative are calculated. For this phase it is therefore interesting to see how different measurements impact the required dike reinforcement dimensions and therefore the expected project cost. This is done by comparing the required dimensions of the dike reinforcement measures based on different measurements which were performed.

Finally the impact of reducing the uncertainty in the realisation phase will be analysed. For this analysis it can no longer be determined what the impact will be of having the information earlier, as with the realisation of the dike reinforcement the design stage of the project is over and construction begins. But it is possible to analyse the impact uncertainty possibly has on the final design for the given scenarios. This is done by comparing the final cost for the measures for larger uncertainty in the statistical parameters.

Next to the benefit in project costs, the benefit of monitoring and soil investigation for the duty of care is also analysed. The duty of care in this thesis implies the legal responsibility to a standard of reasonable care for the administrator of the dike. The aspect of duty of care for the dike trajectory can be divided in two different approaches to the benefit of performing monitoring and soil research. The first aspect is that the risk is larger than expected because the parameters are weaker than expected. The benefit of monitoring and soil research then becomes a better understanding of the dike trajectory and being able to more accurately assess the risk which is taken by leaving the dike as it is during the design process and whether emergency measures are needed.

Monitoring and soil research can be also used to reduce the yearly risk that is taken for the duration the dike. During the period for which the dike trajectory is rejected to the time construction has started, you take a risk by leaving a dike you know is not safe enough for the period of the design process. If beneficial aspects are expected for the dike trajectory, you can lower the taken risk by determining if the failure probability is lower than was first expected.

The result for the benefit of reducing the uncertainty for the project costs and the benefit for the duty of care will be explained in the discussion. Also the limitations of the result of the research will be defined. After the discussion the conclusions can be drawn and recommendation for further research can be defined.

2. Theoretical background

2.1 Dutch policy

2.1.1 General approach flood risk management

The common definition of risk is a combination of the probability of failure times the related consequences of failure. The probability of failure is the probability the load exceeds the resistance in an element. For a system such as a dike ring a probabilistic analysis needs to be performed to assess the failure probability for various failure mechanisms in all elements of the flood defence system to determine the total failure probability. The consequences can be expressed in for example monetary terms or in fatalities, but also consequences such as societal disruption and environmental damages need to be taken into account. This quantification makes it possible to make a comparison between the benefits of risk reduction and the investment costs required.

The base for Dutch flood risk management originates from the disaster year of 1953, in which mayor flooding occurred in the Netherlands. The result was inundation of 2000 square kilometres and 1836 casualties (Kok, 2016). The disaster led to the instalment of the Delta Commission. Their goal was to re-evaluate the flood defences to make sure such a disaster will not happen again. The new risk management policy will be described below.

2.1.2 Old safety standards in the Netherlands

After 1953, new safety regulations were implemented for the flood defences in the Netherlands. The safety standards for flood defence systems was based on a hydraulic load consisting of water levels and wave height with a determined probability of exceedance (Jonkman, 2018). The focus was mainly on the failure mechanisms of overflow and overtopping. For the other failure mechanisms it was determined that the probability of failure should be lower than 10% for the design water level.

The probability of exceedance value was determined based on a risk-based optimization. For this calculation the cost of dike reinforcement was compared to the reduction in flood risk the reinforcement will cause. The principle of this design strategy is shown in Figure 1 (Schweckendiek, 2014). The core idea is to determine the measure for which the total cost of both the investment and the risk was minimal.

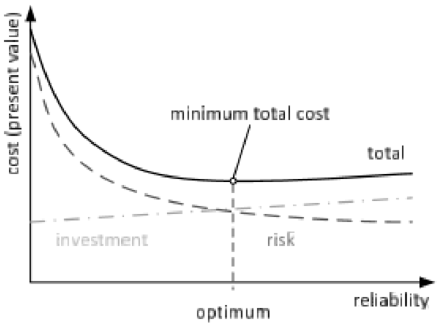


Figure 1 economic optimum for investment in flood reduction

This analysis was performed for every dike ring. Based on the results, the exceedance probability of the design water level was determined. This value varied from very low values of 1/10.000 per year for the densely populated western part of the country to 1/1.250 per year for lower risk areas.

2.1.3 Current safety standards in the Netherlands

Since the start of 2017, different safety standards apply to the flood defences of the Netherlands. These safety standards no longer apply a single design value for the hydraulic loads, but are expressed as a maximum failure probability. Furthermore, the probability of exceedance is no longer determined for the entire dike compartment, but for separate dike trajectories. This is because the consequences of failure might be different for different dike trajectories, which has to be taken into account in the risk analysis. The safety standards for the dike trajectories are determined based on 4 criteria (Kok, 2016).

1. Individual risk
2. Group risk
3. Economic risk
4. Presence of important infrastructure

Individual risk
The individual risk is defined as the probability of dying due to drowning for an individual living in a certain area. The individual risk contains the probability of flooding, the conditional flood characteristics such as flood water height and flow velocity and the probability of dying due to a flooding event (Vrijling, 2011). The maximum individual risk is determined based on the willingness to accept the exposure to this risk in society. For the Netherlands the maximum accepted individual risk for flooding is 10^{-5} per year.

Group risk
Beside the individual risk, the total numbers of fatalities caused by a flood also need to be considered. Considering what the impact will be on society, the acceptable risk is much smaller for large amounts of fatalities than the acceptable risk for smaller amount of fatalities. Typically a FN-curve is used to determine the acceptable risk. In this figure the exceedance probabilities of different numbers of fatalities is plotted on a double logarithmic line. The FN-curve should not cross the criterion line (Schweckendiek, 2014).

Economic risk
Separate from the acceptance criteria of loss of life, the economic point of view of balancing the investment costs into the flood defences and the corresponding reduction of flood risk needs to be analysed. An example of this analysis was already shown in Figure 1. This is typically done using a cost-benefit analysis. In case the optimal failure probability for the cost-benefit analysis are lower than the individual risk, the individual risk is decisive.

Presence of important infrastructure
If important infrastructure is present in the hinterland of the dike trajectory, stricter safety requirements than the requirements above are to be applied. This is for example the case if there is a nuclear power plant present in the hinterland.

All these factors have been taken into account for the safety standards for the dike trajectories. The resulting safety standards are shown in Figure 7.

2.2 Failure mechanisms

There are many possible failure mechanisms which have to be taken into account when assessing the safety of a dike section. Figure 2 gives a summary of failure mechanisms which have to be considered in the design of a dike (TAW, 1998). Due to the limited size of this research 3 failure mechanisms have been selected. These are erosion of the inner slope through overflow or overtopping, sliding of the inner slope or macro instability and piping. These were assumed to be the most likely to cause dike failure for the given case described further on in the thesis.

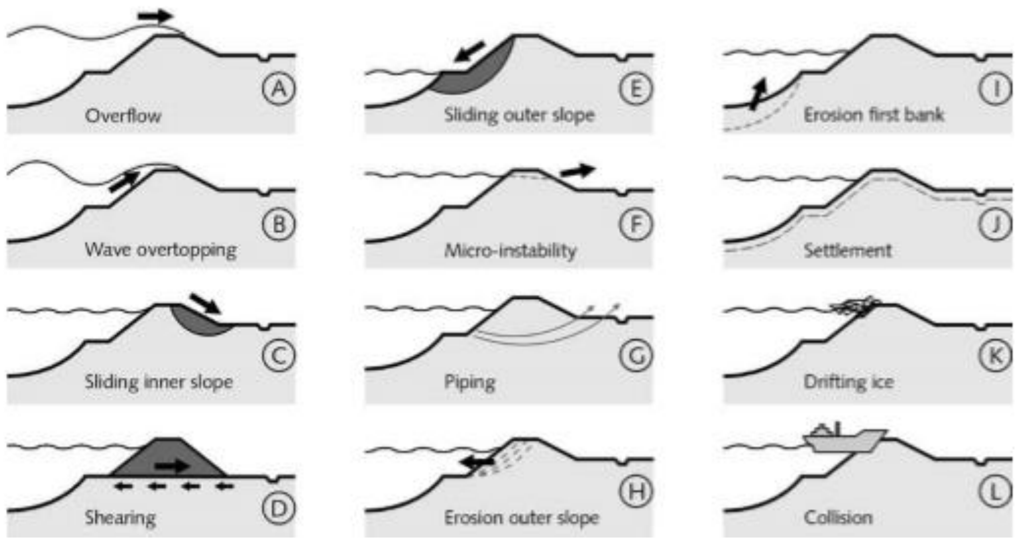


Figure 2 possible failure mechanisms dike

Distinction is made between collapse and failure of the dike. Failure of the dike means the dike loses its ability to perform one or more of the dike's supposed functions. The collapse of the dike entails loss of cohesion or large changes in the geometry of the dike. But there is a difference. The dike can partially collapse but still retain its water retaining function. It is also possible for the dike to overflow with large enough water quantities for the scenario to be considered that failure occurred without the collapse of the dike. Therefore the collapse and failure of the dike need to be treated separately.

2.2.1 Overtopping

One of the main failure mechanisms of a dike is the failure due to overtopping. Waves run-up on the water side of the slope, flow over the crest of the dike and can cause damage to the cover layer on the inner slope of the dike and on the crest. If the overtopping flow is large enough, the flow will start to erode the cover layer. Once this erosion reaches the core material the process starts to weaken the dike structure and dike failure will start to occur.

To determine the amount of overtopping over the dike, two factors need to be determined. First of all the wave height needs to be calculated, and secondly the wave run-up needs to be determined. The wave height is calculated using the Sverdrup-Munk-Brettschneider method. This method uses the wind speed at about 10 meters above the ground and the fetch to determine the significant wave height (Schiereck, 2016).

Subsequently the wave run-up can be calculated. This can be done using the van der Meer equations (Jonkman, 2018). There are three reduction factors which need to be taken into account for this equation. These are the reduction factors for the berm, the obliqueness of the waves and for the roughness of the slope.

Ultimate limit state requirements have been determined for the overtopping volume over the dike, depending on the strength of the cover layer (Molenaar, 2016). For a grass-covered embankment of clay, the maximum mean discharge is between 1-10 l/s/m.

2.2.2 Piping

Backward erosion piping is a failure mechanism relevant for dikes which are located above an aquifer which are covered by a cohesive impermeable layer (van Beek, 2015). Sand particles from the aquifer are transported to the surface, which creates pipes in the subsoil. If these pipes do not reach an equilibrium length, they can keep growing and can eventually cause dike failure. However, for this to happen the pipe needs the layer above the aquifer to be cohesive enough to support the pipe to prevent collapse from above. Piping consists of three sub-mechanisms: uplift, heave and piping (see Figure 3)

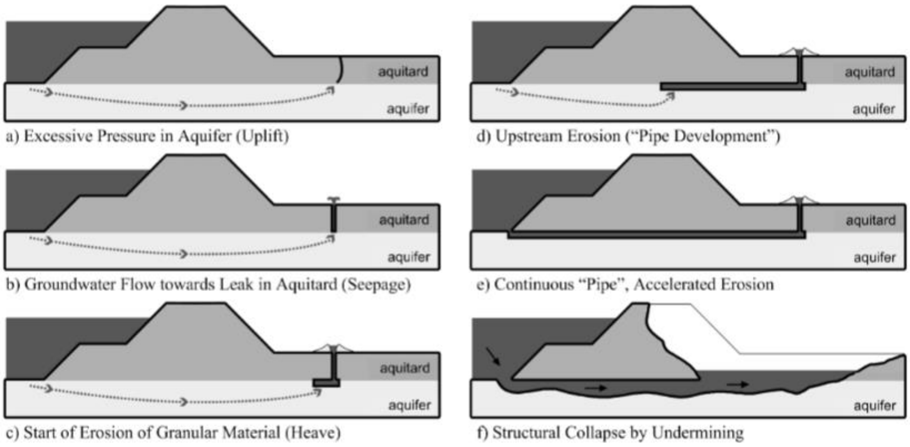


Figure 3 all phases of piping (Jonkman, 2018)

For the pipe to form, the sand particles from the aquifer need to be able to be transported to the surface. If the aquifer is covered by a blanket layer, this blanket layer needs to be ruptured so the sediment has an open exit. This can occur if the upward pressure from the hydraulic head difference exceeds the weight of the blanket layer. The upward pressure can cause cracks in the blanket layer and allow water to flow through.

If the gradient at the crack in the blanket layer exceeds the critical gradient of the sand particles in the aquifer, the aquifer will start to erode. As long as the hydraulic gradient is high enough and no new equilibrium is reached, this erosion will continue. This process is linear until about 40% of the total seepage path, after which it will accelerate until the pipe reaches the outer side (Knops, 2018). When the outer side is reached the resistance of the soil particle disappears and the water can flow freely through the pipe. This increases the flow velocity in the pipe and therefore increases the erosion in the aquifer. As the size of the pipe increases, the stability of the dike above decreases and can fracture or subside and eventually leads to failure.

2.2.3 Macro-instability

When referring to instability or macro-instability in this research, the failure mechanism consists of instability of the inner slope of the dike. The instability of the inner slope is caused by loss of stability of a soil mass along a certain slip plane. There are two driving moments which determine the factor of safety which determine the factor of safety for the dike for macro-instability. These are the driving moment and the resisting moment. The driving moment is caused by the own weight of the soil on the left side of the centre point of the slip plane, which is the half of the slip plane in the direction of the water. The resisting moment is cause by the weight of the soil on the right side of the slip plane facing the hinterland and the shear stress along the slip plane.

In most of the cases, slope instability is only a trigger mechanism for progressive failure of the dike. This is because for the slope instability to be the cause of dike failure the entire dike crest needs to be lowered to such a point that the water can flow over the crest. But typically a part of the crest will remain intact (‘t Hart, 2016). So in this case dike failure has technically not occurred yet. But the weakening of the dike can lead to progressive failure due to other failure mechanisms. The most important progressive failure mechanisms are

- Damage due to second slope instability
- Damage due to micro instability
- Damage due to wave overtopping

Only when the progressive failure leads to lowering the crest level to around or below the water level and a breach is formed through which the water can flow is the definition of failure reached. The remaining resistance of the dike between the trigger failure and the total failure due to progressive failure is called the residual strength of the dike. The residual strength of a dike after slope instability depends on the width of the crest that remains after slope failure, and therefore depends on the critical slip surface.

Multiple Limit Equilibrium Methods (LEM) exist to analyse the slope stability in a dike. These methods mostly vary on what shape is used for the analysed slip surfaces and the shear stress distributions which are used along the slip surface (Zwanenburg, 2013. LEM calculations Finite Element Models can also be used to model slope stability, but these models will not be considered for this research.

2.4 Nature of uncertainty

One of the major problems defined in this research is the uncertainties in the design that occurs when modelling a hydraulic structure. This can be due to limited knowledge, approximation, and heterogeneity in the system, etc. So to analyse how to reduce the uncertainty in how the structure functions it is important to know what types of uncertainties there are.

Uncertainties can be divided into two categories. First there is the inherent uncertainty, which comes from the randomness displayed in the known samples. This represents the variability in nature, that even with a very large sample size you cannot exactly predict what for example the maximum water level in the river Rhine will be next year. This type of uncertainty cannot be reduced.

Secondly there is the epistemic uncertainty. This type of uncertainty stems from the lack of sufficient data or knowledge of all the parameters and phenomena and their effects on the physical system. For example, it might be uncertain which type of distribution the parameter is, or what the standard deviation is. Acquiring more data may reduce the epistemic uncertainty in the design.

Inherent and epistemic can be further divided into more types of uncertainties. These types of uncertainties will be discussed in the chapters below.

2.4.1 Model uncertainty

With model uncertainty the uncertainty coming from the imperfection of the model itself is described. The uncertainty from the input of the model is discussed in statistical uncertainty. Since this analysis involves multiple failure mechanisms, different models are used for the analysed failure mechanisms. The model uncertainty can typically be described as shown in Equation 1, in which m is the model safety factor and f(x) is the result from the model. The mean value for m is 1, as this implies there is no model uncertainty, and the coefficient of variation determines the effect of the model uncertainty.

$$FoS = m * f(x) \quad [1]$$

2.4.2 Statistical uncertainty

The statistical uncertainty concerns the input variables that are put into the model. The sources from this uncertainty are imperfect information, transformation errors, uncertainty due to limited information, spatial variability and the random character of nature (Schweckendiek, 2014). In this chapter the characteristics of this uncertainty will be analysed.

Distribution uncertainty

To describe the geotechnical and hydrological random variables, typically characteristic values are used to model the parameters. The characteristics of the dike and subsoil profile are assumed to be epistemic, as the uncertainty is in the lack of knowledge, instead of in actual heterogeneity of the parameters. This makes this uncertainty interesting to analyse for the design of a hydraulic structure, as it is possible to reduce the uncertainty by collecting more data. With these assumptions, all uncertainty modelled in random variables for the geotechnical and geo-hydrological aspects of the model can be classified as statistical uncertainty. This uncertainty is included in the parameter distributions.

Spatial uncertainty

As stated above in distribution uncertainty, typically characteristic values are used as input in the model. As such, the spatial variability is treated implicitly instead of explicit. As a consequence the length effect is also treated implicitly in the design. In theory, such a characteristic value can be found for every ground parameter, but the designer needs to be careful that the picked value or interpretation suits the chosen model or performance function. While for soil stability representative values can be derived from spatial averages, for groundwater flow one has to account for the upscaling of permeability and anisotropy, and for piping the weakest path problem needs to be incorporated in the model (Kanning, 2012).

2.4.3 Schematisation uncertainty

The schematisation uncertainty characterizes how much the model reflects the reality for the dike section. A large contributor to this uncertainty of the resistance of flood defences is caused by the heterogeneity in ground conditions (Vrijling, 2011). This is because the geotechnical input variables are typically modelled using continuous probability functions or continuous random fields (Schweckendiek, 2010). Often the assumption is made that the properties of the model refer to homogeneous soil layers. The parameters for these distributions have to be extrapolated from statistical analysis of discrete observations or simply by expert judgement. The schematisation uncertainty can be divided into three categories. There is the heterogeneity in the subsoil, geological or man-made anomalies and the geo-hydrological conditions.

Heterogeneity subsoil and anisotropy

The heterogeneity of the soil is a large source of uncertainty in the model. This is caused by the high variability in the stratification on the limited availability of direct measurements. Usually the determination of the stratification scenarios require interpretation of the soil profiles and expert judgement. There are commonly multiple scenarios which match the available boring profiles. In engineering this can be accounted for by using conservative assumptions, or explicitly by generating scenarios and assigning probabilities to them.

The definition of heterogeneity describes the variation of physical properties between 2 or more layers in the subsoil. This implies that a heterogeneous layer consists of at least 2 or more layers. A layer with constant characteristics over the width, height and length is called a homogeneous layer. In comparison, anisotropy describes the difference of characteristics of the soil depending on the direction which is analysed. This for example can be a difference between the vertical and horizontal permeability in a soil layer. If the vertical and horizontal permeability are equal than this is called an isotropic layer. The different processes are visualized in Figure 4.

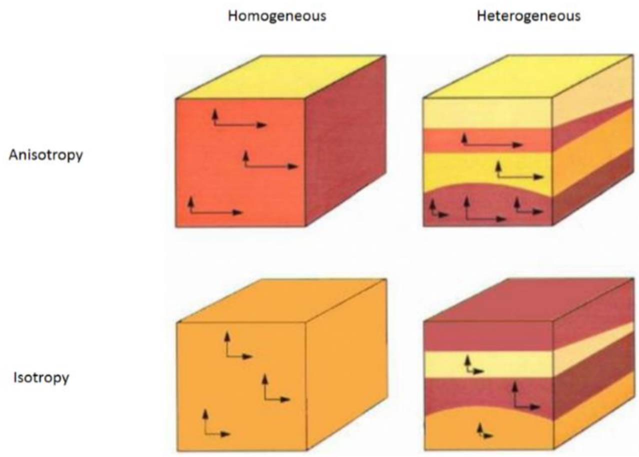


Figure 4: The difference between Homogeneous and Heterogeneous and the difference between Anisotropic and Isotropic soil (Ikelle & Amundsen, 2005)

Both anisotropy and heterogeneousness are commonly observed in the subsoil (Ikelle & Amundsen, 2005). Additionally, another characteristic of heterogeneous soils is that the upper layers of the heterogeneous soil are less permeable than the layers below. This means the lower layers have a draining effect on the upper layers, increasing the critical head difference for which piping occurs.

Geological or man-made anomalies

Two types of anomalies can occur in the soil: Geological and man-made anomalies. Examples of geological anomalies in the subsoil are old riverbeds. Such a local sand layer may lead to high pore pressures under the blanket layer, which may lead to piping. Secondly man-made anomalies can be present in the dike sections. An example is pipelines passing through the dike, or an old clay dike present inside the dike itself. These factors can also affect the factor of safety of the dike section.

Geo-hydrological conditions

The schematisation of the geo-hydrological conditions in and around the dike can also affect the likelihood of a failure mechanism occurring. Especially for piping the seepage length of the dike in a cross section depends heavily on where the entry point of the river to the aquifer is. If a foreshore with a cover layer with low hydraulic conductivity is present, this depends on the hydraulic resistance against the water from entering the aquifer.

2.5 monitoring and soil investigation methods

In this thesis, multiple monitoring and soil investigation methods will be mentioned. Therefore it is beneficial to briefly describe which methods are commonly used for monitoring and soil investigation. The methods applied to determine the corresponding parameters is specified. This will be explained in this chapter.

2.5.1 Determination d_{70}

The d_{70} is measured using soil investigation in the subsoil of the dike trajectory. There are two types of soil investigation. These are geophysical methods and point investigations. Geophysical techniques analyse the composition of the soil (ENW, 2012). Point investigations are used to determine the soil characteristics at that point. Three categories can be discerned in point investigations:

- Field measurements (cone penetration tests, drilling soil samples)
- Laboratory research
- Time series tests (piezometers, deformation measuring techniques)

For the determination of the d_{70} the focus will be on the first two categories. The time series tests will be discussed in the determination of the seepage length. The methods used for this thesis will be described below.

Cone penetration tests

The most common application for the cone penetration test is to determine the soil composition (thickness of each layer and the classification of the soil). During the test the resistance that a cone-shaped element experiences as it pushes through the soil is recorded. This happens at a standardised speed. The measured resistance can be used to distinguish between sand layers and peat or clay layers. The benefit of this method is that it is relatively cheap and can be applied in almost all types of soil found in the Netherlands (ENW, 2012).

Drilling soil samples

In combination with determining the soil composition, drilling also collects soil samples which can be used for laboratory tests of the soil. Furthermore the holes which were drilled can be used to place piezometers to measure the water pressure in the different soil layers. There are two different types of drilling techniques which are applied in the Netherlands. First of all there is drilling by hand. This technique consists of heavy labour, and can only be performed until 5m below ground level. Secondly mechanical drilling can be applied. The two methods which are commonly applied for mechanically drilling are Ackermann drilling and Begemann drilling. Both techniques are suited for collecting samples for laboratory tests on the soil. The Begemann drilling technique has an added benefit that the technique provides an accurate visualisation of the structure of the soil layers.

Laboratory research

Laboratory research on taken soil samples of the dike trajectory is performed to determine the geotechnical characteristics of the soil layers. The following tests are typically performed for the safety assessment and design of dikes:

- Laboratory classification is performed to identify the soil layers. Usually performed as verification of the field measurements
- Determining the volumetric weight and water content. Provides an indication of the consistency of the material. Can also help identify the classification of the soils.
- Determining the undrained shear strength. This can also be analysed in-situ. The results of the undrained shear strength can only be applied to cohesive materials
- Calculating the strength parameters. Most commonly used method to calculate the strength parameters is the triaxial test. This test is relatively costly. The triaxial test can be applied in different soil types in drained and undrained conditions. The results can be used for stability analysis of the dike.
- Determining compression parameters of the soil in the laboratory. This is only useful for cohesive materials for which the load is increased. Used to analyse the settlement of the soil.
- Analysing the grain size distribution of the soil. From the grain size distribution the hydraulic conductivity can be estimated using correlation methods. The grain size distribution is important to determine the risk of dike failure due to piping for the dike trajectory.
- Determining the hydraulic conductivity of the soil. The constant head method is applied to permeable soils. The falling head method is applied to soils with a low hydraulic conductivity such as peat or clay. However, the hydraulic conductivity in the soil can be different from the result of the laboratory, as cracks and other structural aspects of the soil influence the hydraulic conductivity of the soil layers.

2.5.2 Determination hydraulic conductivity

Figure 5 provides an overview of the available methods to measure the hydraulic conductivity. The most commonly used methods to determine the hydraulic conductivity are the correlation methods and hydraulic methods (Stoop, 2018). Hydraulic methods can be divided in methods performed in the laboratory and field methods performed in-situ. For the field methods a distinction can be made between quick and mobile small scale tests and large scale tests which are more expensive and time-consuming. However, large scale field methods provide a more reliable representative value for the hydraulic conductivity. For this thesis, the HPT boring test and correlation methods were used as methods to determine the hydraulic conductivity. These will be briefly explained in this section.

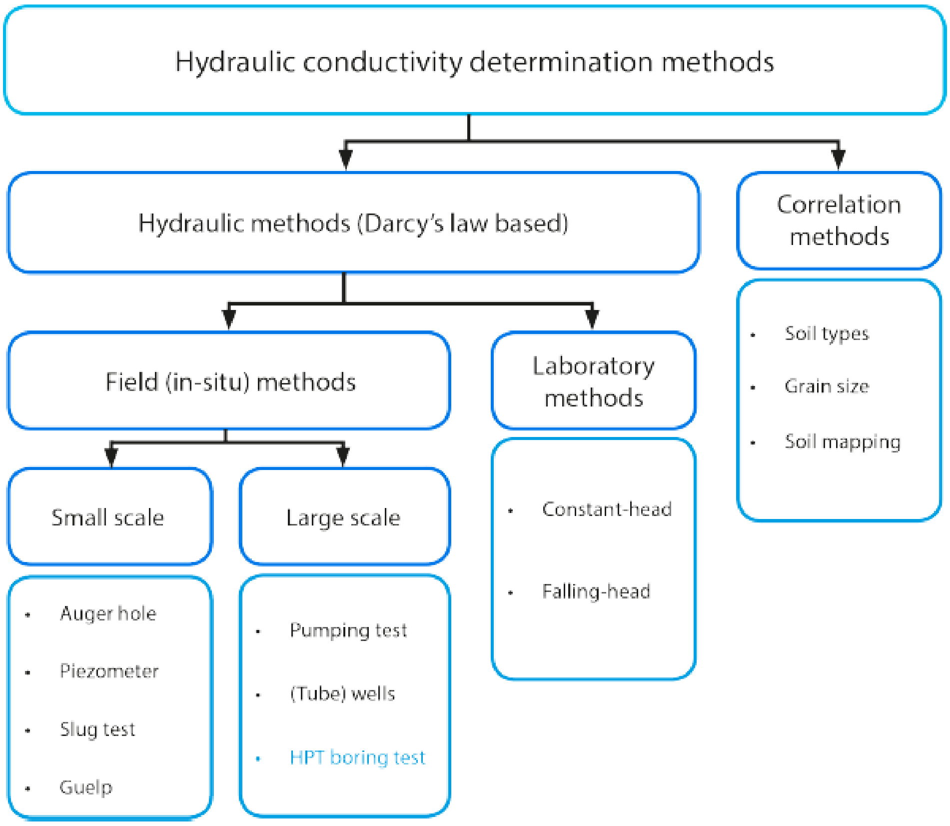


Figure 5 schematised overview of the methods available to determine the hydraulic conductivity (Ritzema, 1994)

First of all there is the correlation method for the determination of the hydraulic conductivity. These methods determine the hydraulic conductivity based on different soil properties, such as soil type or grain size. The advantage of these method is that the hydraulic conductivity can be easily determined. The disadvantage is that these methods are not very accurate as they only provide the rough relations between the two soil properties. The hydraulic conductivity can vary with a factor of 10 based on which correlation method is applied (Berbee, 2017).

2.5.3 Determination seepage length

The seepage lengths of a dike consists of three sections. These are the effective foreshore, the width of the dike and the effective length of the hinterland. For the case used in this thesis, the width of the dike and the effective length of the hinterland are known but the effect of the foreshore is not. This will be elaborated in chapter 3. In this section it is explained how the length of the effective foreshore can be determined by monitoring the hydraulic head in the aquifer using piezometers.

The stability of dikes is primarily dependent on pore pressure and hydraulic head in and under the dike during the normative load (ENW, 2012). Piezometers are useful to determine the geo-hydrological characteristics of the soil. They can be used to perform the following measurements:

- Manual piezometer recording. The recording is performed at a regular interval, for example once every week. The measurement series can be used to verify the geo-hydrological model and extrapolate the measurement series to the normative load on the dike. This method is not suited for dynamic or cyclical conditions.
- Electronic piezometer recording. Electronic piezometer recordings can be used for high frequency (continuous) measurements of the hydraulic head in the soil layer. These

measurements can for example be used to determine the hydraulic response in a sand layer in the dike on a high wave.

- Field measurements to determine the geo-hydrological parameters of the soil. This category include pump and well tests to determine the hydraulic conductivity and the storage capacity of an aquifer or the hydraulic resistance of a layer with a low hydraulic conductivity.

If there is a foreshore present at the dike which is covered by a layer with a low hydraulic conductivity, this layer can provide hydraulic resistance against the water to enter the aquifer. This can even occur when the cover layer is interrupted by ditches or other obstacles. The cover layer providing hydraulic resistance against the water entering the aquifer, the hydraulic head in the aquifer becomes lower than the water level in the river. This effect can be measured using piezometers

Common practice for choosing the entry point is using the outer toe of the dike (Roode, 2019). The entry point is typically defined as the point where the hydraulic head in the aquifer matches the water level in the river. This is the point where the water can enter the aquifer unhindered.

By including the hydraulic resistance of the cover layer it is possible to determine a fictional entry point based on the relation between the hydraulic conductivity and the hydraulic resistance of the cover layer, increasing the seepage length. This is shown in Figure 6. Furthermore monitoring using piezometers also are also beneficial for more accurately assessing the damping of the hydraulic head in the aquifer towards the hinterland.

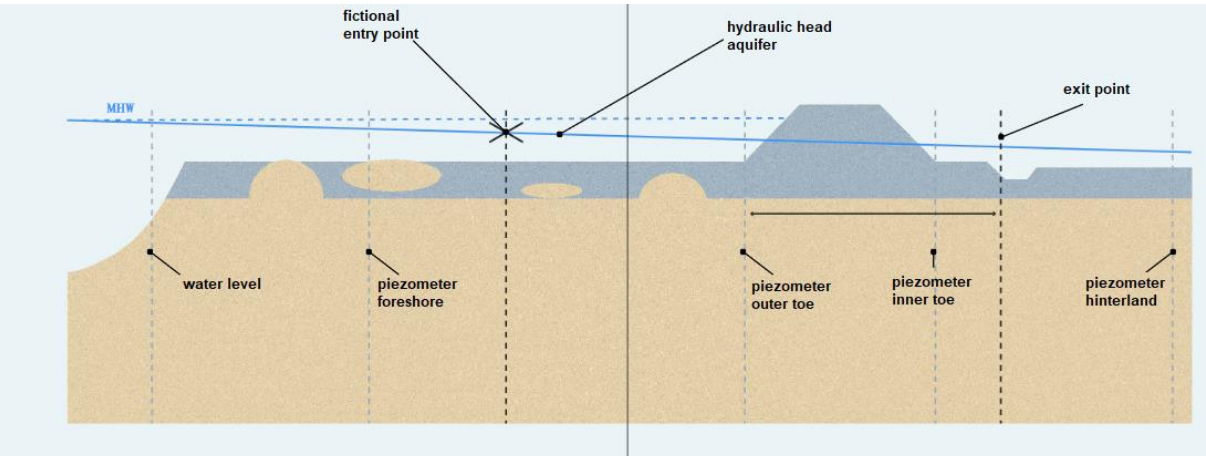


Figure 6 schematic representation of a dike with a foreshore and cover layer (Roode, 2019)

There are some limitations for monitoring the hydraulic head of the aquifer using piezometers which are mentioned below:

- Collecting measurement series of the hydraulic head takes time. If the results are to be included in design or the safety assessment, measuring should start well before the design is made
- Normative circumstances for the design are typically not recorded during the measuring period. The measurements are usually interpolated using geo-hydrological models to come to the normative loads. It needs to be verified in which period the most useful measurements can be recorded.
- To use geo-hydrological models the geo-hydrological parameters of the layers should be known. This means soil investigation must be performed before monitoring takes place. The measurement series from the piezometers can be used to verify the parameters in the model.

3. Case description

As was mentioned in the methodology, a theoretical case study will be used to determine the effect of performing monitoring and soil investigation in different project phases of the dike reinforcement project. The case functions as example of the different steps to be taken for a dike reinforcement project, and how monitoring and soil investigation contribute in each of these steps. In this chapter the case is introduced and a safety assessment is performed. Furthermore the project structure applied for this thesis is explained. And finally, the scenarios used throughout this thesis is described.

3.1 Case introduction

The dike trajectory used for the analysis is section 44-01 of dike ring 44, located near Amerongen. The length of this trajectory is approximately 1800m. The representative cross-section for the dike trajectory has been provided by Fugro (Figure 8). The representative cross-section is based on the results of the safety assessment, from which the dike trajectory was found unsafe due to piping. Soil investigation has already been performed for the safety assessment, such as a literature study and expert judgement to determine the d_{70}. This provides the base scenario for collecting information on the geotechnical and hydrological characteristics of the dike trajectory.

The case is based on a dike trajectory located along the Nederrijn in the Netherlands. The Nederrijn is a branch of the river Rhine, which flows through the middle of the Netherlands from the confluence at the town of Angeren of the Old Rhine and the Pannerden Canal. It continues past Wijk bij Duurstede as the River Lek. As the river is a branch of the Rhine, the water levels of the river are dependent on the discharge of the Rhine.

For the subsoil of the area, the Nederrijn is located in the River lands, which is part of the fluviatile clay parent material group (Bakker, 2006). Parent material is the description of the physical and chemical characteristics of the upper sediment layer. The lower subsoil is dominated by coarse sand deposits originating from the Pleistocene and is regarded as the Kreftenheye formation. Locally deposits of peat and loam can also be present.

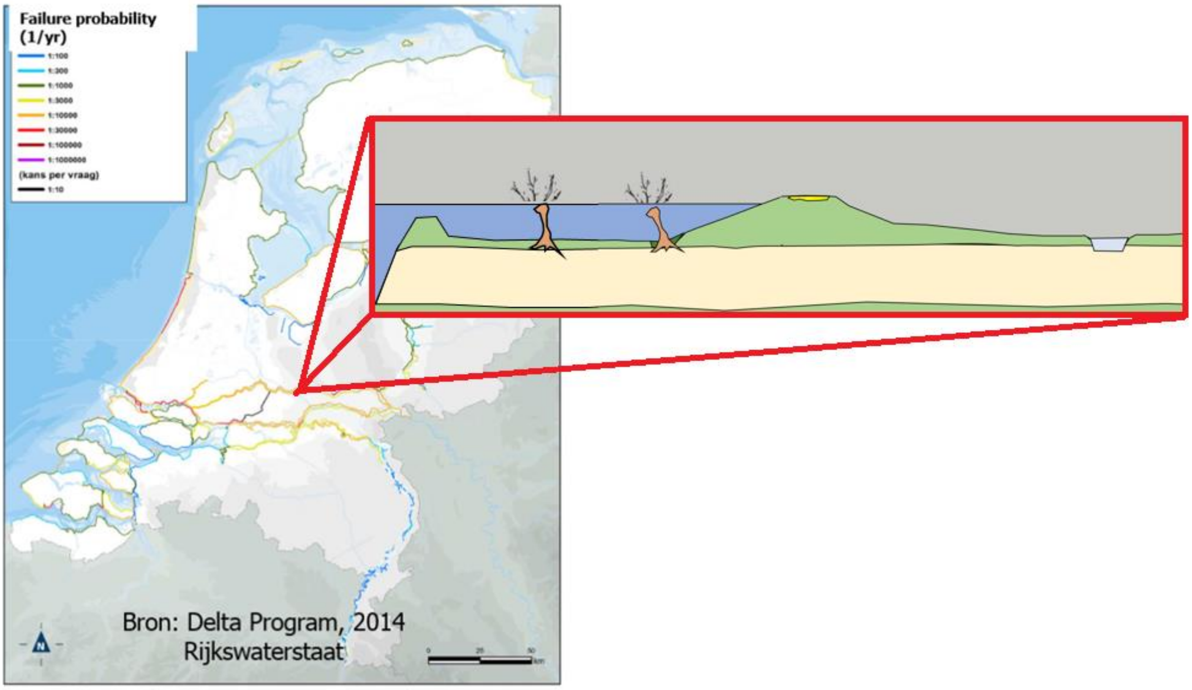


Figure 7 location of the case

The subsoil for the River lands is determined by the Echteld and Naaldwijk formations. However, the Naaldwijk formation is more of a marine formation found closer to the coast. At the case location the Echteld formation is expected in the subsoil. This formation consists of sandy deposits as well as sandy to heavy clays (Bakker, 2006).

Base scenario

Minor soil investigation has already been implemented before the start of the project. The grain size distribution was determined using archived data and geological knowledge of the area. The mean d_{70} of the upper layer of the aquifer was determined using the lithology classes. The mean value of the range of grain sizes found in the smallest lithology class found in upper layers of the aquifer in the dike trajectory was used as the d_{70} in the representative cross-section. To determine the hydraulic conductivity a correlation method with the grain size was applied.

At the location of the dike trajectory, the dike is separated from the river by a swath of land functioning as foreshore. When the water rises, the foreshore gets flooded. The effect of a longer seepage length has on the hydraulic head in the aquifer at the toe of the dike depends on the geometry and characteristics of the foreshore. Since no data was available on the foreshore, as a conservative estimate the effect of the foreshore on the seepage length was neglected for the representative cross-section.

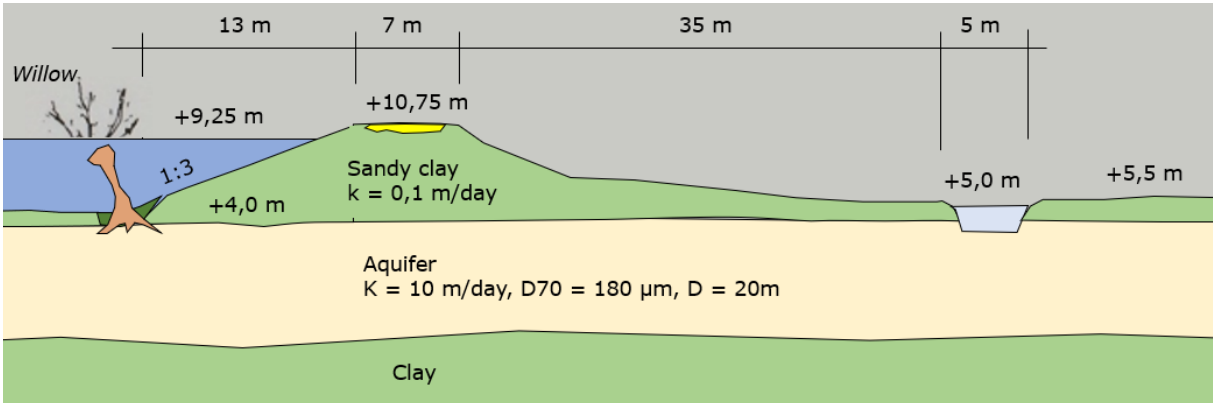


Figure 8 Representative cross-section

As is shown in Figure 9, the presence of this foreshore can significantly impact the seepage length of the dike. 40m of foreshore is present in front of the dike, but the foreshore contains willows whose roots are possibly in contact with the aquifer. This means water can enter the aquifer at these points. It is therefore unknown what the entry point is for the water flow. The length of the effective foreshore are based on the different schematisations for the cross-section of the dike. The foreshore is 40m long, with trees possibly in contact with the water-carrying layer at 20m from the dike and at the toe of the outer slope of the dike. These are assumed to be the possible entry points for the pipe.

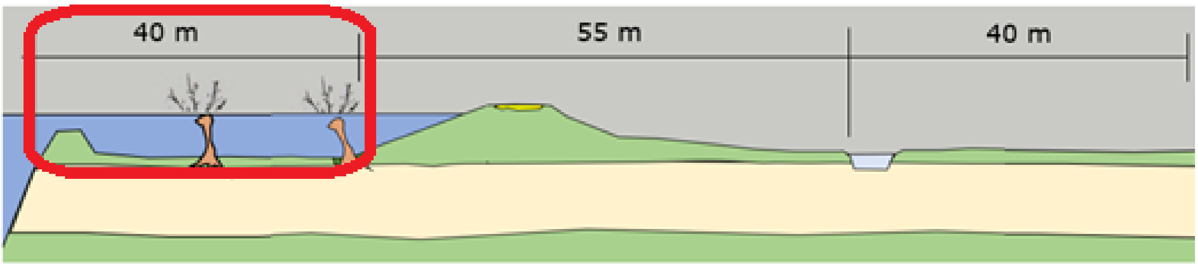


Figure 9 representation of the foreshore

To calculate the probability of failure due to piping, Sellmeijer's equation was used. In order to perform this calculation a few input parameters are required. These parameters are given in Table 1.

Symbol	Parameter	Mean value	unit
H_p	Hinterland phreatic level	5	[m + NAP]
d	Thickness hinterland blanket	0,5	[m]
L_{eff}	Length (effective) foreshore	0	[m]
B	Width dike	55	[m]
λ_h	Seepage length hinterland	37,4	[m]
D'	Aquifer thickness	20	[m]
γ_{sat}	Saturated volumetric weight blanket	16	[kN/m3]
γ_w	Saturated volumetric weight water	10	[kN/m3]
γ_s	Volumetric weight sand grains	26,5	[kN/m3]
I_{ch}	Critical heave gradient	0,7	[-]
Θ	Bedding angle	37	[deg]
v	Kinematic viscosity	$1.33*10^{-6}$	[m2/s]
η	Constant of White	0,3	[-]
g	Gravitational constant	9,81	[m2/s]
d_{70}	70%-fractile of grain size distribution	180	[µm]
d_{70m}	Reference value for d_{70}	208	[µm]
K_{zo}	Hydraulic conductivity aquifer	$1.0*10^{-4}$	[m/s]
K_h	Hydraulic conductivity aquitard	$1.2*10^{-5}$	[m/s]
M_u	Model factor uplift	1	[-]
M_p	Model factor piping	1	[-]
H_{dec}	Decimation height	0,5	m
H	MHW	9,25	m

Table 1 input parameters piping case dike trajectory (retrieved from the case supplied by Fugro)

The seepage length of the dike can be determined by combining the effective length of the foreshore, the width of the dike and the seepage length of the hinterland (Equation 2)

$$L = L_{eff} + B + \lambda_h \quad [2]$$

The design water level for the dike trajectory is shown in Figure 8, and is equal to +9,25m in reference to the N.A.P. Using the design water level the annual maximum water level can be determined. This design water level corresponds to the water level with a probability of exceedance of 1/1250. The annual maximum water level is described with a Gumbel distribution given in Equation 3.

$$F(h) = e^{-e^{-\alpha(h-u)}} \quad [3]$$

The design water level is linked to protection levels in terms of exceedance probabilities which are specified for the dike trajectory (Ministerie van Milieu en Waterstaat, 2017). Using the decimation height, the design water level and the corresponding annual probability of exceedance the parameters for the Gumbel distribution of the annual maximum water level can be determined. The decimation height is the water level difference that reduces the exceedance probability by a factor of 10. For the dike trajectory at Amerongen the decimation heigh is equal to 0,5m. To determine the Gumbel parameters, Equation 4 and 5 were used.

$$F_{exc} = 1 - e^{-e^{-\alpha(MHW-u)}} \quad [4]$$

$$\frac{F_{exc}}{10} = 1 - e^{-e^{-\alpha(MHW+h_{dec}-u)}} \quad [5]$$

Equation 4 and 5 can be rewritten so the parameters of the distribution become as shown in Equation 6 and 7. Implementing the annual probability of exceedance, the corresponding MHW and the decimation height results in the parameters of the gumbel distribution given in Table 1 results in the gumbel parameters shown in Table 2. The resulting probability density function is shown in Figure 10.

$$\alpha = \frac{z(F_{exc}) - z\left(\frac{F_{exc}}{10}\right)}{h_{dec}} \quad [6]$$

$$u = MHW + \frac{1}{\alpha} * z(F_{exc}) \quad [7]$$

For which

$$z(F) = \ln(-\ln(1-F)) \quad [8]$$

Parameters Gumbel distribution	**Value parameters**
α	4,6
u	7,7

Table 2 parameters Gumbel distribution annual water level maxima

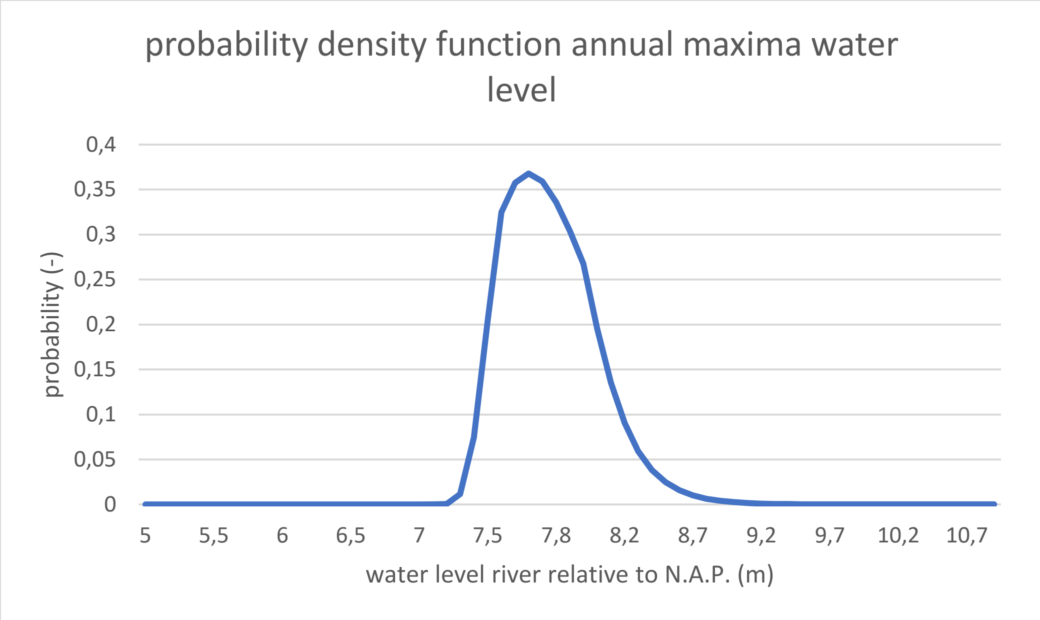


Figure 10 probability function annual maxima water level river

It was mentioned that piping is the dominant failure mechanism for the case dike trajectory. This was verified using fragility curves of three failure mechanisms. The fragility curves and their corresponding calculations have been included in Appendix A and B. To determine which failure mechanism is dominant, the failure probability is compared at the design water level at 9,25m above N.A.P. (Table 3). It was found that piping is the dominant failure mechanism for this dike trajectory. The benefit of early monitoring and soil investigation will be analysed for this failure mechanism.

Water level river relative to N.A.P. (m)	**Failure probability piping (-)**	**Failure probability overtopping (-)**	**Failure probability macro-instability (-)**
9,25	$4{,}0*10^{-4}$	$2{,}9*10^{-7}$	$2{,}4*10^{-6}$

Table 3 failure probability for different failure mechanisms given the water level

Dike reinforcement base scenario

To determine what dike reinforcement is needed for the base scenario, first the required annual failure probability of the dike trajectory due to piping needs to be determined, which is done in Table 4. The safety standard at the case dike trajectory set by law specifies two boundaries for the failure probability, which are the lower boundary of 1/10.000 and the signal value of 1/30.000. The required probability of failure is chosen based on the life cycle process of a dike shown in Figure 11, which leads to a required annual failure probability of 1/40.000.

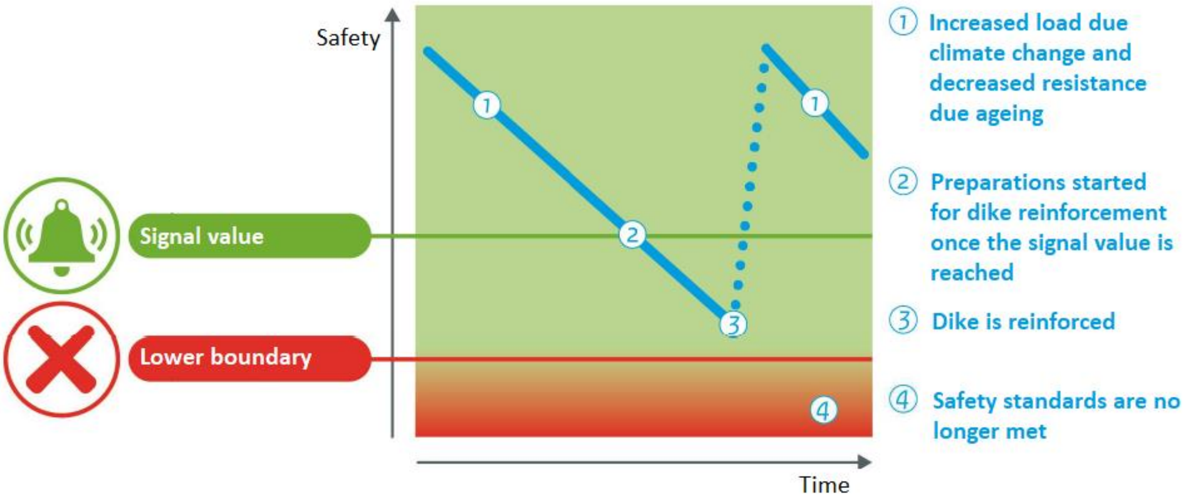


Figure 11 Safety over the life cycle of the dike (adapted from Waterschap Limburg, 2017)

The values for the parameters for the length of the dike trajectory sensitive to piping and the length of a typical independent section due to piping are the standard values for piping at the case location. The fraction of dike trajectory sensitive was also verified in literature that the standard value is applicable (Förster, 2012). These parameters lead to the required annual probability due piping shown in Table 4.

Total length dike trajectory	**L**	**1800**
Contribution piping to failure probability	ω_j	0,24
Fraction of the dike trajectory sensitive to piping	a	0,4
Length of a typical independent section for failure due to piping	b	300
Length effect factor	N	3,4
Required annual failure probability for the dike trajectory	P_{req}	$2{,}5*10^{-5}$
Required annual failure probability for the dike trajectory due to piping	$P_{req,p}$	$1{,}8*10^{-6}$

Table 4 required annual failure probability due to piping

Based on the soil and hydrological parameters for the base scenario the cheapest necessary dike reinforcement was determined. For the reinforcement method the cheapest method between a piping berm and a sheet pile wall was chosen (see Appendix B). The result is a 19m sheet pile wall implemented in the toe of the dike, which is illustrated in Figure 12. The estimated project costs for a sheet pile wall were calculated in Appendix D, which is approximately €401,- per m^2 of sheet pile wall. With an 1800m long dike trajectory the estimated project costs for the dike trajectory is approximately
13,7 M€.

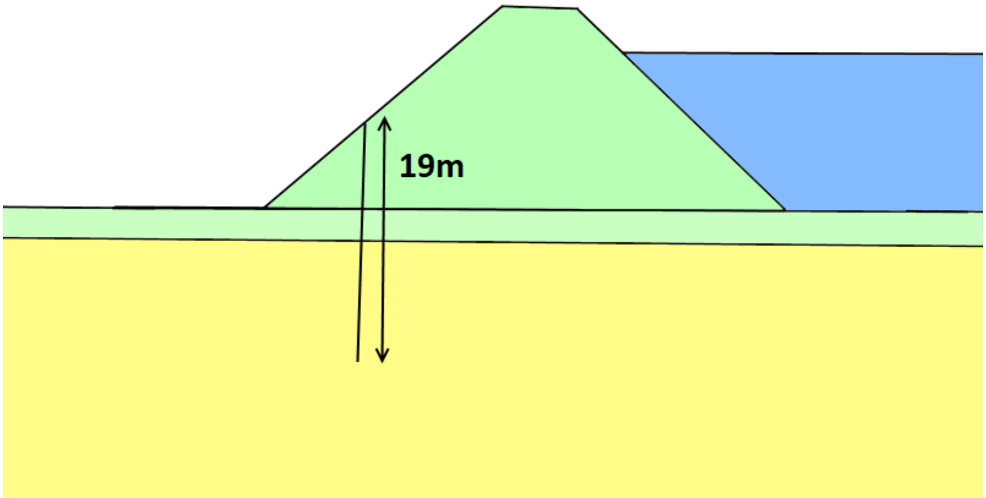


Figure 12 sheet pile wall implemented in the base scenario

Sensitivity analysis
A sensitivity analysis was performed for the parameters relevant to piping. For the sensitivity analysis it was analysed how much the coefficient of variation for certain parameters had an impact on the final probability of failure. This was done by performing a simple sensitivity analysis using the one at a time principle, for which the coefficient of variation was varied for each variable between 0 and 1. The result is shown in Figure 13. For the probability of failure A FORM-analysis was performed in Prob2B. The used equations are included in Appendix B.

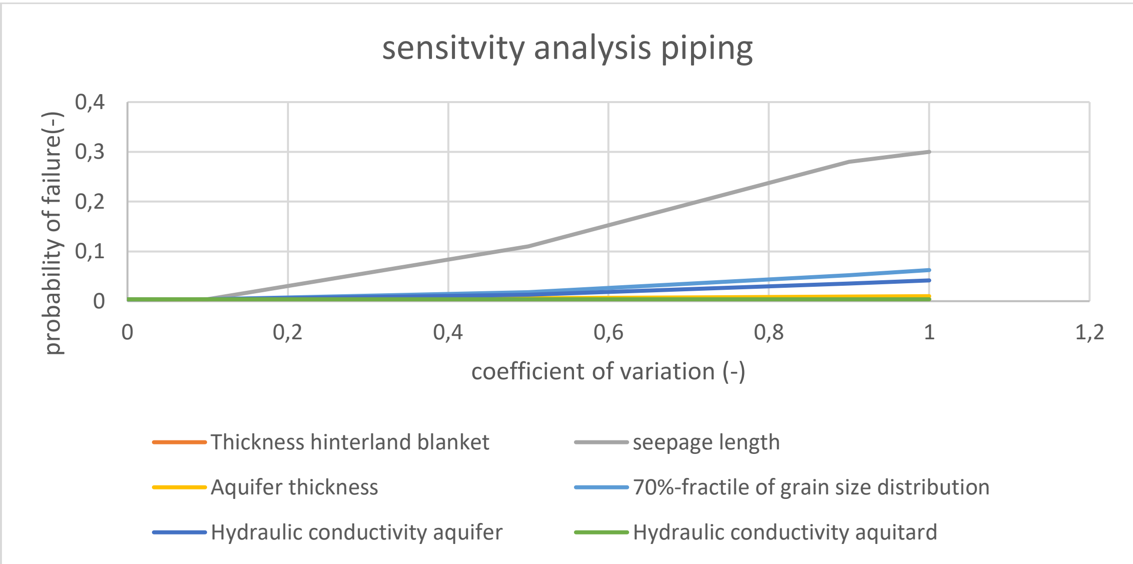


Figure 13 sensitivity analysis piping

What can be seen in the figure is that the most influential parameter on piping for the dike trajectory is the seepage length, with the hydraulic conductivity of the aquifer and the d_{70} of the aquifer as second and third. These three parameters will be used for the analysis of the benefit of monitoring and soil investigation in the project phases.

3.2 project structure dike reinforcement project

Since the research focuses on what the benefit will be to collect information in earlier phases of the project, this chapter will define 4 specific project phases typically used for a dike reinforcement project. The used phasing of the project is based on the project definition of the HWBP (Bernardini, 2017). In the project definition of the HWBP, four project phases are defined. These are the initiation phase, the reconnaissance phase, the plan elaboration phase and the realisation phase. Decisions made for a specific project phase can influence the phases that follow, but should not limit the design during the following phases (Figure 14).

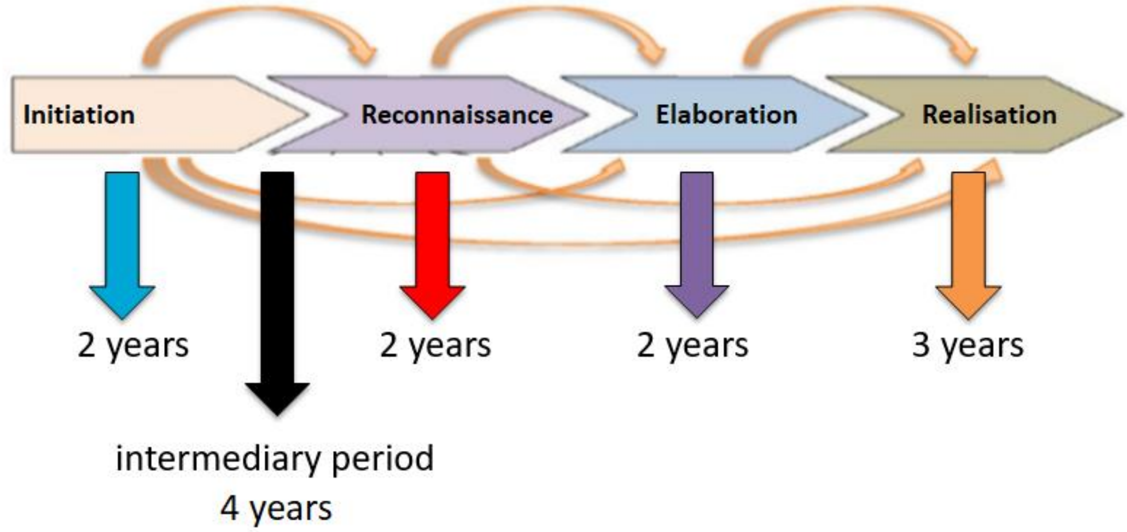


Figure 14 average values for project phases of a dike reinforcement project

Application of this type of phasing is flexible, a project does not have to explicitly complete every phase. Sometimes the phases can be combined if this is beneficial to the project. However, each phase is finished with an administrative resolution and each phase starts with a start resolution. Agreements are made on the scope, risks, planning, progress and finances which become more detailed as the project progresses.

Safety assessment

The project starts with the decision that dike reinforcement is needed. This is usually based on the results of an assessment of the flood defences whether they are still up to the current standards. This inspection takes place every 12 years for the primary flood defences in the Netherlands (Waterwet, 2018), in which is tested whether the dike can withstand the determined hydraulic loads. If the dike trajectory fails the assessment whether the dike meets the safety standard, there are two possible outcomes. If the dike trajectory might turn out to be sufficiently strong if a more detailed analysis is performed, then the dike trajectory will be analysed in more detail. If it is likely the dike will still not meet the safety standard even if a more detailed analysis is performed, no further analysis takes place and it is decided the dike needs to be reinforced. The initiation phase ends with the application of the reinforcement project to the HWBP programme.

Initiation phase
If the results of the third test round for the failed inspection of the dike trajectory based on the safety requirements are deemed insufficient to assess the safety of the dike, it is possible for the regional water authority to include a pre-reconnaissance phase as well (Bernardini, 2017). This phase is used to gain a better perspective on the safety of the dike in consideration of the relevant failure mechanisms. Subsidy for pre-determined activities can be requested from the HWBP.

Reconnaissance phase
The HWBP determines which dike is reinforced first by ranking the dike projects based on importance. Once the HWBP makes the decision that the dike project will be included in their programme, an inventarisation of possible alternatives for the dike reinforcement is made. This contains the possible solutions which can be used to make sure the dike trajectory meets the current standards. From the possible alternatives a preferred alternative is chosen. This decision is based on criteria such as safety, sustainability, costs and public support, as well as practicality and maintenance.

Intermediary period
The intermediary period is the waiting period between the application of the dike to the HWBP programme and when funding is received to start the reconnaissance phase. The length of the period is based on ranking determined by the probability of failure of the structure and the consequences of failure for the structure. Dikes which protect economically valuable hinterland and which are more likely to fail will be reinforced quicker than dikes for which failure will have little effect. Therefore the time period between the initiation of the project and the start of the reconnaissance phase can vary significantly.

Elaboration phase
Once the preferred alternative is chosen, the plan is elaborated upon to the level of detail needed for the application of the main licenses. Depending on the contract strategy, the contract preparation for the realisation phase is typically started in the elaboration phase. One of the end products of this phase is the project plan.

Realisation phase
Finally the realisation phase starts with preparing for the execution and the actual execution of the reinforcement measure. When the reinforcement is finished the dike meets the determined safety norm. After the realisation phase the regional water authority takes over responsibility of the dike. This transfer has been discussed with the regional water authority during the reconnaissance phase.

The length of each phase is different for every project. But for this thesis, average length of the project phases has been estimated based on the definitive programme of the HWBP of 2019-2024 and 2018-2023. There is however large variation in the length of the intermediary period. All dike and hydraulic structures are ranked based on the probability of failure of the structure and what the consequences of failure can be for the structure. This ranking determines the length of the intermediary period. Dikes which protect economically valuable hinterland and which are more likely to fail will be reinforced much quicker than dikes for which failure will have little effect. This can vary between instantly starting the reconnaissance phase of the project after the application to the programme is finished, to waiting for 5 to 10 years. In this case an intermediary period of 4 years is used.

3.3 Description of the scenarios used for monitoring and soil investigation

In this chapter the scenarios used throughout this thesis are explained. The scenarios are generated by combining three categories which together form the scenario for the expected result of the monitoring and soil investigation on the design. The scenario is then used to calculate the expected benefit in project costs for applying the monitoring and soil investigation. The three categories are:

1. Soil characteristics of the dike trajectory.
2. Monitoring and soil investigation methods.
3. Project phase the measures are applied.

Soil characteristics

For the soil characteristics, three types of schematisations of the dike trajectory have been determined. The schematisations used are shown in Table 5, and are elaborated below.

For the first type of schematisation there are 1 to 3 sections in the dike trajectory. In each section the most conservative value found in the section for the relevant parameter are used. For example if moderately coarse sand is the smallest grain size lithology class found in the section of the dike trajectory, the d_{70} based on this classification is used for the section of the dike.

For the second type of schematisation of the soil characteristics, the soil characteristics are the same for the entire trajectory, but for which one parameter is different than the soil characteristics described in the base scenario in chapter 3.2.

Finally for the third type of schematisation, the soil characteristics which result in the highest expected value is described. The product of the probability of occurrence combined with the expected reduction in project costs result in the highest value for this scenario.

Monitoring and soil investigation methods

The second subcategory for the scenarios are the monitoring and soil investigation methods. For this thesis the soil investigation results in the d_{70} in the upper layer of the aquifer, the hydraulic conductivity methods measure the hydraulic conductivity K in the aquifer and the monitoring of the hydraulic head measures the seepage length L under the dike. In which level of detail the characteristics of the dike trajectory are known depends on the amount of soil research which is performed.

Project phase the measures are applied

When the measurements are performed determines in which project phases the information can be used. Data collected by soil investigation performed in the reconnaissance phase for example can be used in the reconnaissance phase and the elaboration phase. As such the project phase in which the measures are applied influences the estimated project cost reduction.

Each scenario is numbered using the combination of the three subcategories. At the start of the project, some information on the soil characteristics of the dike trajectory has already been collected, referred to as the base scenario. This scenario is 1-A-0. However, more detailed soil investigation and monitoring will likely provide a different result. The project phase in which the measures were applied is numbered 0, as the measures were applied before the project started.

As example scenario 2-E-III is used. The soil characteristics of the dike trajectory are defined in two sections, two HPT-borings are applied on top of the already applied methods of the base scenario and the HPT-borings are applied in the elaboration phase (year 8).

Soil characteristics dike trajectory	Sections (-)	d_{70} (µm)	K (m/day)	L (m)
1. **Base scenario**	1	180	10	55
2. **Two sections**	2	255	5	55
		180	10	75
3. **Three sections**	3	180	5	95
		255	10	75
		255	5	55
4. **Small change d_{70}**	1	255	10	55
5. **Large change d_{70}**	1	360	10	55
6. **Small change K**	1	180	5	55
7. **Large change K**	1	180	2,5	55
8. **Small change L**	1	180	10	75
9. **Large change L**	1	180	10	95
10. **Scenario highest expected benefit**	2	255	10	55
		180	10	75

Table 5 soil characteristics of the dike trajectory

The soil characteristics of the dike trajectory are described in Table 5, and can be divided into three types. The soil characteristics numbered 1 to 3 are the conditions for which different parameters can be found along the dike trajectory. For example, a coarser sand can be found in the upper layer of the aquifer for one part of the dike trajectory than the sand in another part. The soil characteristics in the dike trajectory numbered 4 to 9 describe scenarios for which different soil parameters are present than described in the base scenario from the safety assessment described in chapter 3.1. And the soil characteristic with the number 10 is the scenario with the highest expected benefit from the scenarios (See Appendix F).

Monitoring and soil investigation methods	K	d_{70}	L_{eff}
A. **base scenario** Archive data and expert judgement d_{70} correlation method hydraulic conductivity	X	X	
B. Soil investigation probe interval of 100m.		X	
C. Soil investigation probe interval of 50m		X	
D. One HPT boring measurement.	X		
E. Two HPT boring measurements.	X		
F. Three HPT boring measurements.	X		
G. A monitoring plan is implemented.			X
H. Soil investigation probe interval of 100m. Two HPT boring measurements.	X	X	
I. Soil investigation probe interval of 100m. Three HPT boring measurements.	X	X	
J. Soil investigation probe interval of 100m. A monitoring plan is implemented.		X	X
K. Three HPT boring measurements. A monitoring plan is implemented.			
L. Three HPT boring measurements. A monitoring plan is implemented.	X		X
M. Soil investigation a probe interval of 100m.	X	X	X

	Two HPT boring measurements. A monitoring plan is implemented.			
N.	Soil investigation a probe interval of 100m. Three HPT boring measurements. A monitoring plan is implemented.	X	X	X

Table 6 monitoring and soil investigation methods applied in the dike reinforcement project

The second subcategory of project conditions are the monitoring and soil investigation methods implemented during the dike reinforcement project (Table 6). These measures are additional to the soil investigation which already took place for the safety assessment. These were described in chapter 3.1.

Soil investigation entails both using probes and taking soil samples to be analysed in the laboratory. The interval at which these methods are performed is mentioned as the probe interval. The hydraulic conductivity of the aquifer is determined using HPT boring measurements. A monitoring plan using piezometers is implemented to determine whether the foreshore influences the seepage length of the dike. 4 piezometers are placed over the cross-section of the dike at an interval of 100m.

Time and phase of implementation	
I.	Monitoring started/soil investigation performed in year 0. (initiation phase)
II.	Monitoring started/soil investigation performed in year 6. (reconnaissance phase)
III.	Monitoring started/soil investigation performed in year 8. (elaboration phase)

Table 7 possible implementation strategies

Finally there is the project phase in which the methods from Table 6 are implemented (Table 7). A distinction must be made between implementation of additional soil investigation and monitoring. Soil investigation can be completed within a year (ENW, 2012). Data on the soil composition and characteristics can always be found, independent from conditions at the dike trajectory. Therefore when soil investigation is implemented is treated as a point in time, as visualized in Figure 15. This means the data is available for subsequent project phases. The arrows indicate when the soil investigation is performed and when the data from the measurements is used.

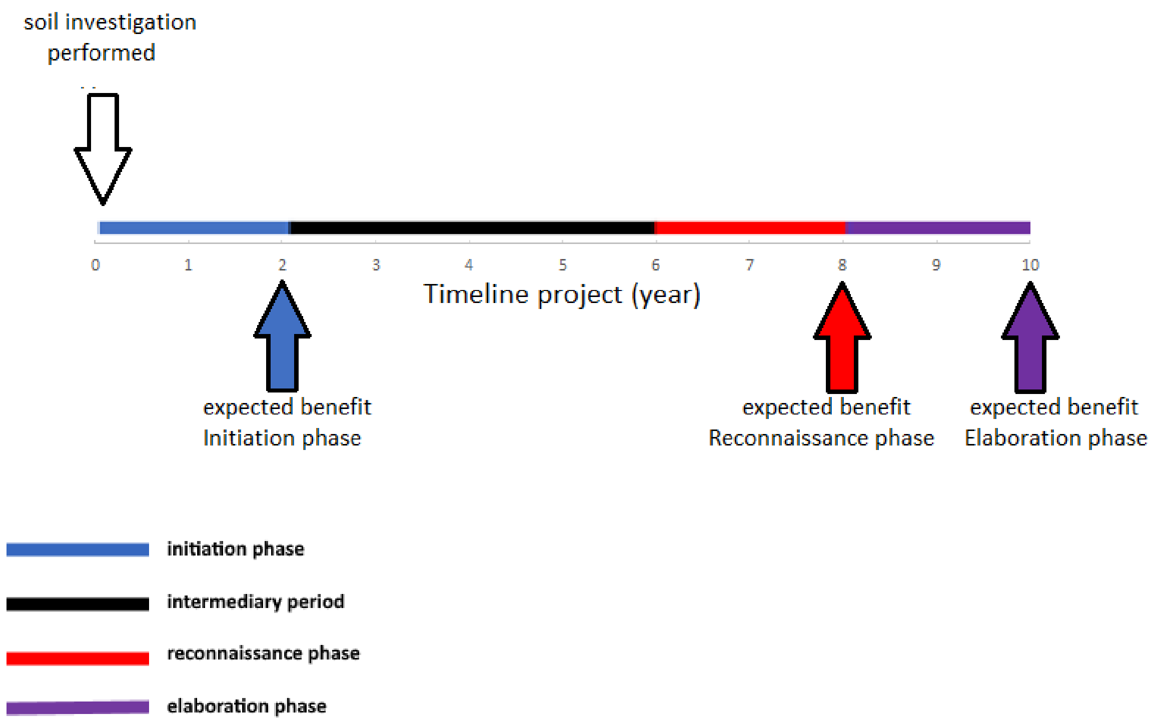


Figure 15 effect of additional soil investigation on decisions in the project phases

Implementing a monitoring plan works differently than conducting soil investigation. Measurements for monitoring are taken periodically over time, and require certain conditions to be met. For example, a water level in the river high enough to determine the fictional entry point is needed for the measurements to be relevant. The moment of implementation is therefore treated as the start of the measuring period. A visualization of the measuring period in a dike reinforcement project is shown in Figure 16. The bar on the x-axis indicates the monitoring period during the project. The probability of having relevant data for the project decision is then dependent on the length of this measurement period. If relevant data from the measurements is acquired during the measurement period, then the data can be used at the points in time indicated by the arrows.

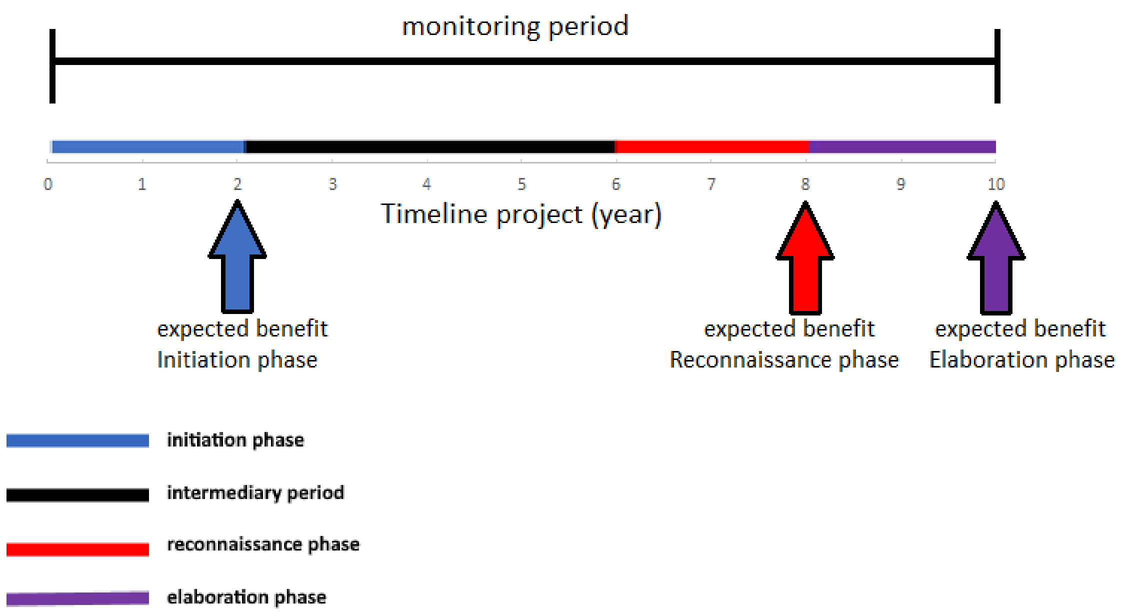


Figure 16 effect of the monitoring plan on the project decisions

4. Monitoring and soil investigation for project costs

In this chapter, the following topics are covered:

- Effect monitoring and soil investigation in different project phases
- Difference results monitoring and soil investigation
- Application of previous results to different scenarios

The result of monitoring and soil investigation are assumed to be different parameters for the soil parameter for a section of or the entire dike trajectory. The different values from the parameters stem from available data on the dike trajectory provided by Fugro and Dinoloket. The d_{70} of the upper sand layer in the aquifer were derived from the lithology classes of these layers, for which the soil composition is shown in Appendix G. The hydraulic conductivity was determined based on data from REGIS II and a HPT-boring provided by Fugro shown in Appendix H. The length of the effective foreshore was determined based on the locations of willows as entry points in the foreshore. These were estimated using satellite images from Google maps.

First the effect of monitoring and soil investigation is analysed for each of the defined project phases. The next step is to analyse the combined effect of monitoring and soil investigation for the entire dike reinforcement project. This is applied to different scenarios of what the soil characteristics are and what the applied monitoring and soil investigation uncovers about these characteristics (See Chapter 3). Finally a strategy is devised to deal with these scenarios.

4.1 Effect on the initiation phase

The level of detail in which the dike inspection takes places in the Netherlands is dependent on the state of the dike. If the basic inspection shows that the dike will fail and the inspector rules that further inspection will not prove that the dike is safe, the dike will be rejected based on the simple inspection (Hordijk, 2010). This can be detrimental for the dike design, as further inspection can provide insight into how the dike is unsafe and what measure is needed to increase the safety to an acceptable standard.

During the initiation phase, the project scope is determined before it is applied to the HWBP programme. Additional information to the data collected for the dike inspection on the composition of the subsoil can be beneficial for defining the project scope. When using the representative cross-section for the dike trajectory, it seems the entire dike trajectory needs to be reinforced. But further insight on the soil composition for the entire dike trajectory might show that not the entire dike trajectory is weak to piping, and that some sections might not need reinforcement at all. This principle is shown in Figure 17.

A total seepage length of 95m must be present to meet the safety standards for piping of this dike trajectory described in chapter 3 if all the other soil parameters are as described in the base scenario. If the fictional entry point into the aquifer is at the beginning of the foreshore such that the length of the entire foreshore can be included in the seepage length than this criteria described above can be met. The benefit in the initiation phase is in determining the length of the dike trajectory which will not need any dike reinforcement. This reduces the scope of the dike reinforcement project and the project costs for reinforcing the dike. Using the estimated project costs of constructing a seepage berm per m^2 of €256,12 (Appendix G), this leads to an estimated cost reduction of around €7427,-/m for the seepage berm alternative.

Figure 17: defining the project scope for the application to the HWBP programme

Length dike section for which L=95m (m)	costs seepage berm (M€)	costs sheet pile wall (M€)
0	7.4	7.2
180	6.7	6.9
360	5.9	6.7
540	5.2	6.4
720	4.5	6.1
900	3.7	5.8
1080	3.0	5.5
1260	2.2	5.3
1440	1.5	5.0
1620	0.7	4.7
1800	0	4.4

Table 8 difference project cost for seepage length =95m

The results in Table 8 are only one example on how a longer seepage length found by monitoring leads to a reduced project scope. A different combination of results from the soil investigation and monitoring may also lead to a reduction in the project scope. But to determine if for the dike trajectory a longer seepage length can lead to a smaller project scope, monitoring of the hydraulic head in the aquifer needs to be implemented (Tonneijck, 2018). This measuring method takes time to collect relevant data, so it is preferable to start monitoring early in the project to have a longer measuring period. This will be elaborated on in chapter 4.5.

Although the effect of the level of detail of the measurements performed during the safety inspection on the project costs is an interesting topic, it is not the main focus of this research. The best approach to what kind of inspection of the dike trajectory is most beneficial is a topic large enough to cover its own research. So to keep the thesis within the scope the topic is not analysed any further.

4.2 Effect on the reconnaissance phase

Once the scope of the project has been determined, the preferred alternative needs to be selected. Two different methods of dike reinforcement are used for this thesis. Using more alternatives will overcomplicate the analysis, while two alternatives are enough to serve as example for the comparison. The only factor for the choice of preferred alternative is the project costs. However this will rarely be the only factor for determining the preferred alternative (TAW, 1998). The two used stability measures for piping are constructing a stability berm on the inner slope of the dike and constructing a sheet pile wall in the inner slope of the dike.

The benefit in project costs for the choice of alternatives lies in the different situations for which the methods are applicable. Stability berms are generally cheaper than sheet pile walls for shorter seepage lengths as soil has a lower price and is easier to place in comparison to a sheet pile wall. But if the required length of the berm becomes too long to fulfil the required seepage length to meet the safety requirements, the sheet pile wall becomes a more favourable alternative for the criteria of project costs. Also when there is only limited space behind the dike available the sheet pile wall is better applicable because the piping berm requires more space in the hinterland.

Given the representative cross-section for the dike trajectory from the safety inspection of the dike, the sheet pile wall is the cheaper alternative. However the representative cross-section is derived from conservative assumptions, and taking the local conditions over the length of the dike points to less severe reinforcement needed for sections of the dike trajectory. This chapter focuses on the benefit in project costs for choosing the cheaper alternative given the more detailed representation of the parameters over the dike trajectory instead of simply using the representative cross-section.

In this chapter only the difference in cost between the alternative dike reinforcement methods are treated, as the reconnaissance phase is used to determine which dike reinforcement method will be used. The estimated project cost for specific dimensions of the dike reinforcement methods is elaborated on in Chapter 4.3.

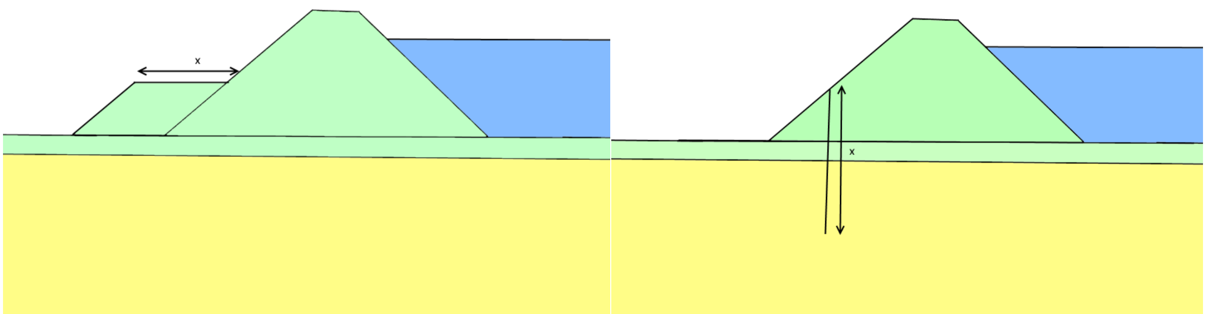


Figure 18 Choice of preferred alternative for the reconnaissance phase

4.2.1 Effect preferred alternative for grain size distribution d_{70}

The first parameter of the subsoil that is considered for the choice of preferred alternative is the grain size distribution of the aquifer directly below the top layer. The used d_{70} for the representative cross section is 180 µm, determined from the mean value of the lithology class for fine sand in the safety assessment. The soil layers present in the dike trajectory can be found in Appendix H. Using the most unfavourable situation as representative scenario may seem as the best approach in terms of safety. But this may lead to a conservative design which is unnecessarily expensive. A more cost effective design method is to take the spatial variability of the grain size distribution into account and design the dike trajectory accordingly.

For the analysis of the reconnaissance phase, the dike trajectory was divided into two sections. The values for d_{70} are based on the found lithology classes for the upper soil layer in the aquifer. The project costs were estimated for the two dike reinforcement methods used in this thesis. The cost calculations per method can be found in Appendix G. The costs were determined based on estimated values for costs per m^2 of the implemented dike reinforcement method. The difference in costs between the piping berm and sheet pile wall were compared for different lengths of the dike sections. The project costs for the piping berm were subtracted from the project costs for constructing a sheet pile wall in the inner slope of the dike.

For measuring the d_{70} in the aquifer, soil investigation using probes and taking soil samples is commonly applied. Typically a probe interval for primary flood defences is 100m (ENW, 2012). This is assumed to be sufficient to determine the d_{70} in the aquifer along the dike trajectory. In this chapter only the difference in project costs is estimated given that the d_{70} value occurs over a certain length of the dike trajectory, not the likelihood of this actually occurring over that length of the dike. This will be treated further on in the chapter. The found scenarios for the value of d_{70} are based on soil composition retrieved from Dinoloket.

Figure 19 shows the difference in project costs when a piping berm is used versus when a sheet pile wall is applied. The total project cost for the sheet pile wall are subtracted with the total project costs for the piping berm. The length on the x-axis indicates the length of the section for which the upper layer of the aquifer consists of medium sand using the lithology classifications (Williams, 2006). The mean value for lithology classes of the upper layer of the aquifer which were in the dike trajectory were used as d_{70}, which are is 255 µm and 360 µm.

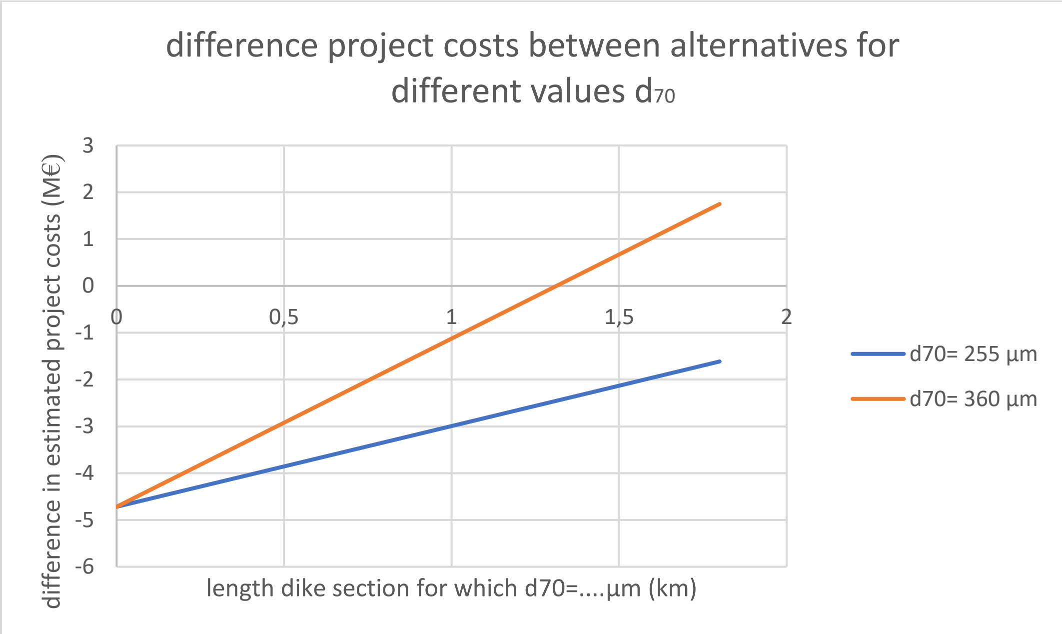


Figure 19 difference in project costs between the alternatives for d_{70}

A negative value for the difference in project costs when the length for the dike section for which d_{70} is 255 µm indicates that the sheet pile wall is the cheaper solution given the lengths of the two sections for that d_{70}. When solely a different value d_{70} is measured, the sheet pile wall remains the preferred alternative for most of the time. Only when there is an approximately 1400m section for which the d_{70} is 360µm is the piping berm a cheaper solution. For d_{70}=255µm the cheaper alternative is still a sheet pile wall. This means that if measurements reveal that d_{70}=255µm this will not change much for the choice of preferred alternative

When using the coarser sand with a d_{70} of 360 µm which was found in drill samples along the dike trajectory the piping berm is the cheaper alternative for that section. The increased grain size for d_{70} has a larger impact on the required length for the piping berm than for the sheet pile wall, as the difference in project costs between the two measures is larger when compared with the line for d_{70}=255µm in Figure 19.

4.2.2 Effect preferred alternative for hydraulic conductivity of the aquifer

The same calculation was performed for the hydraulic conductivity of the aquifer. In Figure 20 the difference in project costs between the alternative dike reinforcement methods is shown if for part of the dike trajectory the horizontal hydraulic conductivity is equal to 5m/day and 2,5m/day. The values for the hydraulic conductivity are based on soil data from Dinoloket. The highest value of the provided range of K in the measurement is used as conservative value of the measurement. The soil profiles are provided in Appendix G. The difference between the project costs for the different dike reinforcement methods is much larger for K than that found for the d_{70}. Even the smaller change of 5m/day is enough for the piping berm to become the cheaper alternative This has to do with the fact that when using the method of Lane for the sheet pile wall a different hydraulic conductivity does not impact the length of the sheet pile wall (see chapter 4.3.2.).

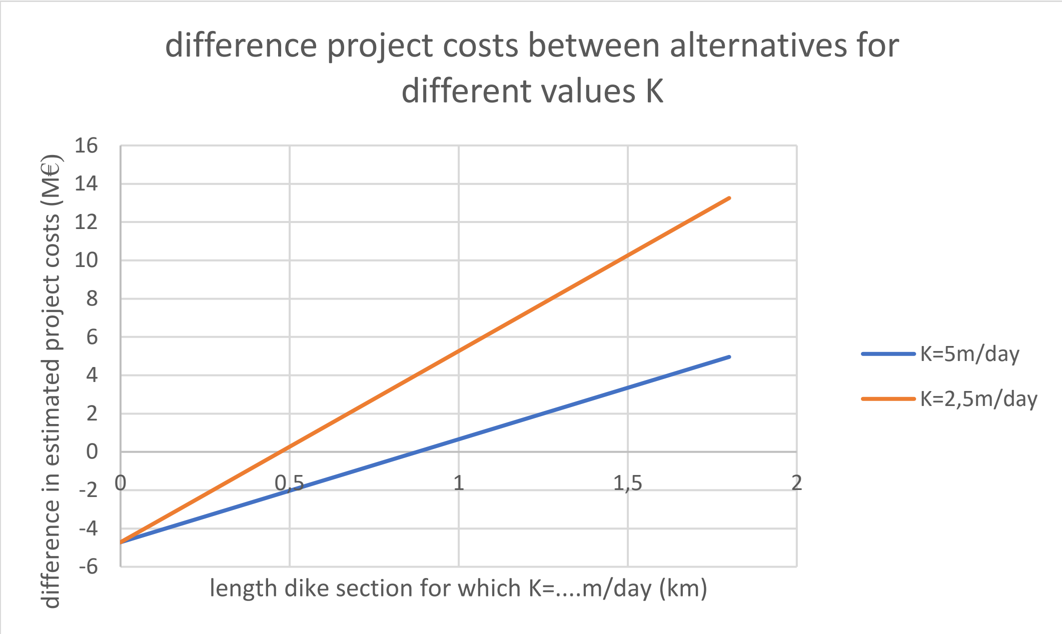


Figure 20 difference in project costs between the altneratives for K

The difference in estimated project costs between the alternatives becomes even larger for K=2,5m/day. Since the piping berm is the cheaper alternative for shorter seepage lengths, the difference then becomes larger when the seepage length becomes smaller. This reduces the project costs for the dike trajectory when the cheaper alternative is applied at the dike section for which the determined hydraulic conductivity is lower.

4.2.3 Effect preferred alternative for seepage length dike

Finally for the different scenarios of the impact of the increase in seepage length on the difference in project costs, the entry point in the foreshore was taken as the dominant factor. The entry points were determined based on tree locations and whether or not these connected to the water carrying layer. The distance between these entry points and the dike are shown in the case description. The distances of these entry points from the dike are 20 and 40 meter.

For monitoring the hydraulic head in the aquifer over the cross-section it must be taken into account that relevant data is not immediately acquired and takes time. This will be discussed in chapter 4.5.1. For now, only the reduction in project costs given that the relevant data is collected is analysed.

The difference between the piping berm and the sheet pile wall for a section of the dike trajectory for which an increased seepage length of 20 meter can be accounted for is shown in Figure 21. The range of difference in project costs is larger than the effect by larger grain size distributions, but smaller than the effect by a smaller hydraulic conductivity. This is likely because the increase in horizontal seepage length can be included in Lane's method, whereas a change in the hydraulic conductivity cannot. The sensitivity analysis performed for the case description found the probability of failure was more sensitive to the seepage length than to the hydraulic conductivity and the d_{70}.

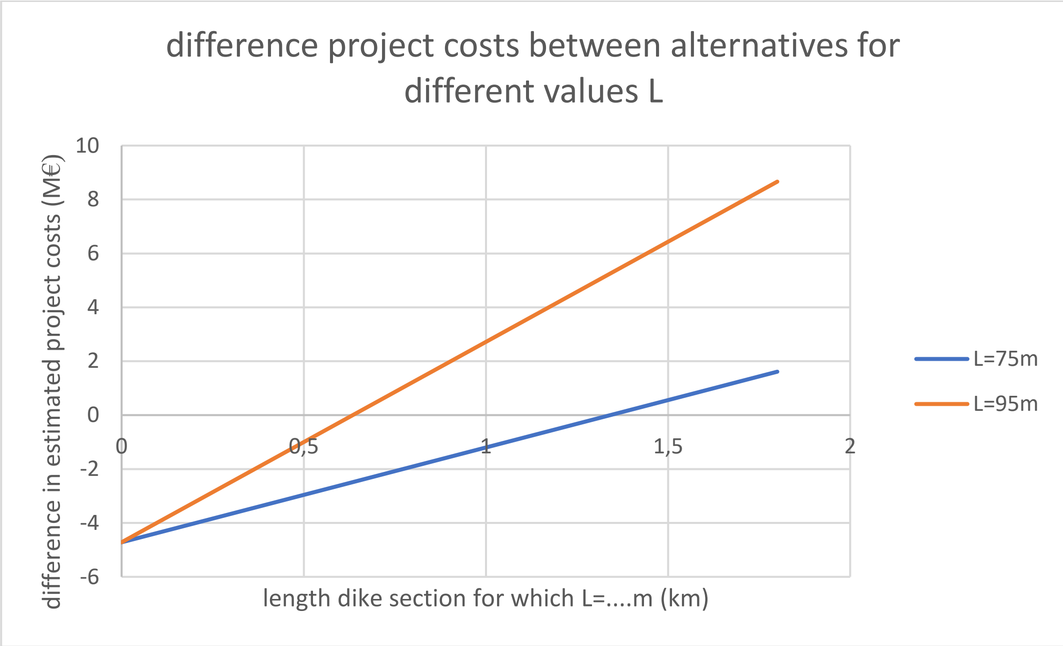


Figure 21 difference in project costs between the alternatives For L

The possible difference in total project costs is significant, however gathering data on the effective foreshore length is more difficult. In contrast with determining K and the d_{70} which are measured using a point investigation, the measurement of the effect of the foreshore on the dike safety is dependent on time. The soil can be tested independent of time and an applicable result can be found. But for the impact of the foreshore on hydraulic resistance the water pressure in the aquifer needs to be measured (Tonneijck, 2018). To obtain a useful result a long measurement period is needed, preferably containing a high water situation. This means that even if there is an effective foreshore present on the riverside of the dike, the usefulness of the monitoring data depends on the fact whether they include a high water situation. The exploration of this effect will be described later in this report.

4.3 Effect on the elaboration phase

In the elaboration phase, the design is specified to the point that applications for the licenses can be made. For this research, the task performed in this project phase is determining the exact dimensions of the preferred alternative are to meet the safety requirements. This is visualised in Figure 22. If no monitoring and soil investigation has been performed in the previous project phases, the preferred alternative based the information of the base scenario is used. The preferred alternative for the representative cross-section of the base scenario is implementing a sheet pile wall.

The benefit of monitoring and soil investigation in terms of project costs is found in the possible reduction in size of the required dike reinforcements. If for example a longer seepage length than first assumed is found and the sheet pile wall can be 2m shorter for the entire dike trajectory, than that can save a considerable amount of construction costs for the project. Positive outcome for the results of monitoring and soil investigation can be in the form of less conservative design values for the entire dike trajectory, or stronger parameters found in sections of the dike for which less reinforcement is needed than the rest of the trajectory. The effect of negative results of monitoring and soil investigation in the form of weak spots in the dike trajectory is treated in chapter 5.1.

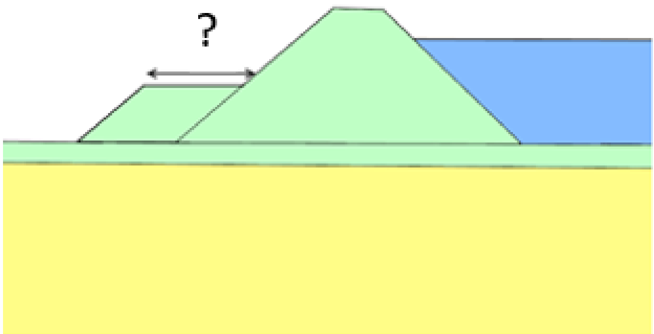


Figure 22 example of a different schematization than the representative cross-section

4.3.1 Effect due to grain size distribution d_{70}

As was explained for the reconnaissance phase, the difference in project costs was determined for the presence of different scenarios for the grain size distribution for a section of a certain length in the dike trajectory. For the elaboration phase the size of the needed dike reinforcement needs to be determined, specifically how much the size of the dike reinforcement can be reduced once additional soil investigation or monitoring take place. Part of the research is what the impact will be when monitoring and soil investigation starts in different project phases. Therefore the reduction of design size needs to be determined regardless of the results for the choice of alternative. This means that the project costs for different partitions of the sections is determined for both of the alternatives.

The effect of a more detailed design based on the presence of a section with larger grain dimensions in the aquifer is shown in Figure 23 and Figure 24. The results are calculated in using the same method as the previous chapter, only these results show the total costs while the results from chapter 4.2. show the difference in costs between the alternatives. The actual cost reduction depends on the local situation and the schematisation of the dike trajectory.

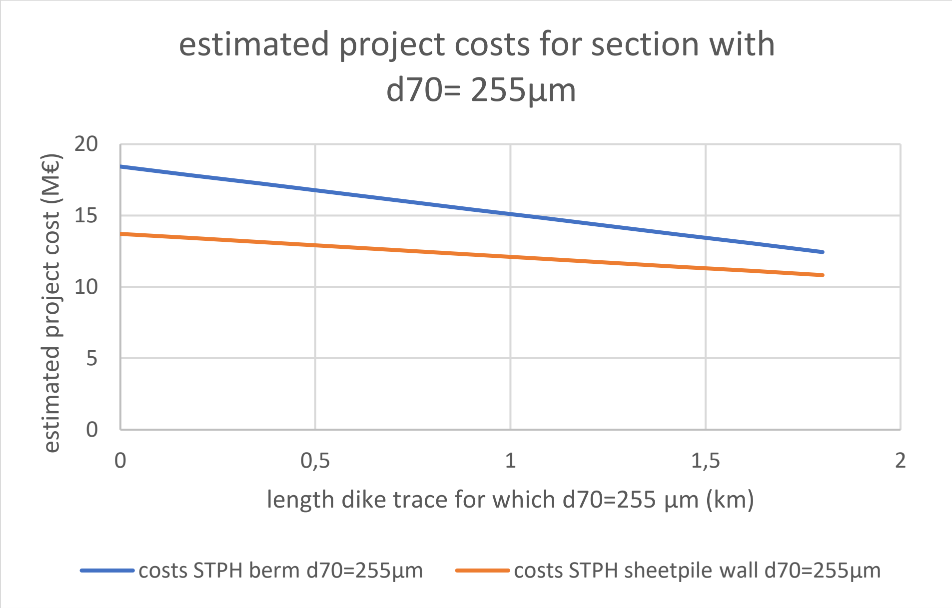


Figure 23 project costs for section of dike trajectory with d_{70} of 255µm

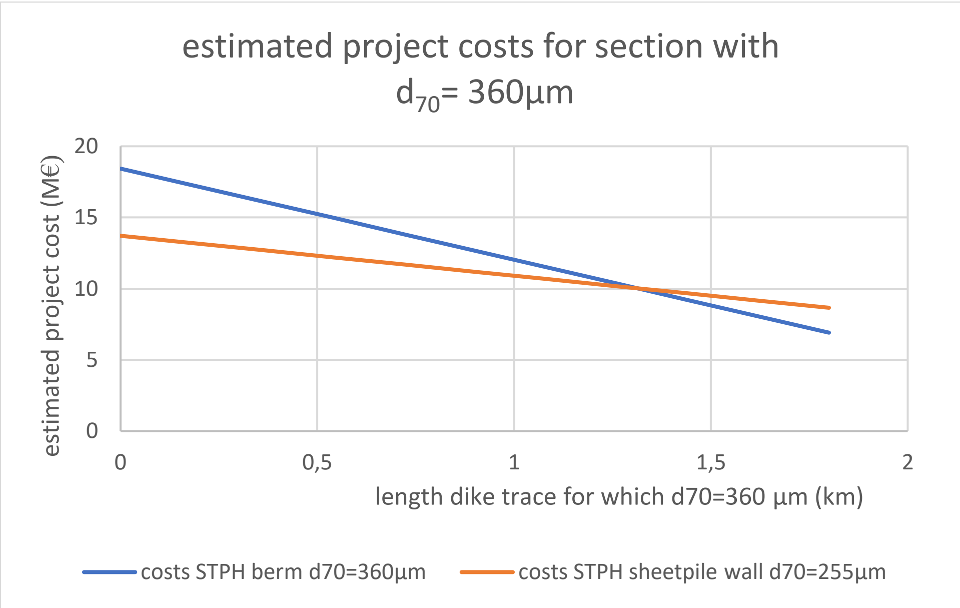


Figure 24 project costs for section of dike trajectory with d_{70} of 360µm

4.3.2 Effect due to hydraulic conductivity of the aquifer

The effect of the hydraulic conductivity of the aquifer on the project costs needs to be determined. Three differences in trends can be seen in the obtained results visualised in Figure 25 and Figure 26. First of all the expected reduction in project costs for the dike reinforcement method of a piping berm is larger for the horizontal hydraulic conductivity than for the grain size characteristics. The result is more comparable to the effect found due to the increase in seepage length, which will be treated in the next chapter. The relatively larger impact for the hydraulic conductivity than for the grain size distribution is probably due to the fact that the relative change for the hydraulic conductivity is much larger than the d_{70}, as in the sensitivity analysis it was found that the factor of safety was more influenced by a change in the d_{70} than a change for the K.

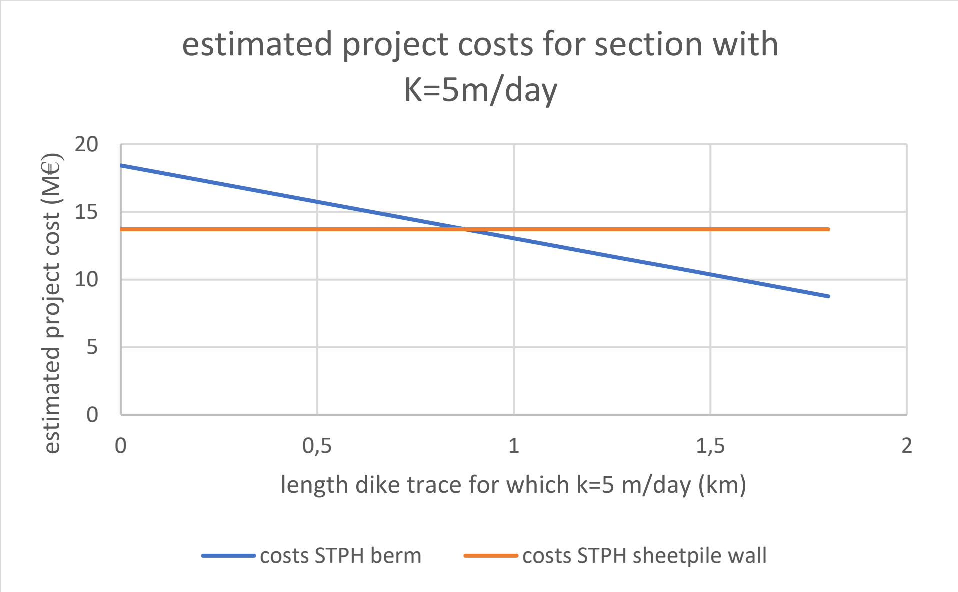


Figure 25 project costs for section of dike trajectory with K_h=5m/day

Secondly, the effect of the hydraulic conductivity on the project costs of the sheet pile wall is non-existent in the method of Lane. The method of Lane is an empirical formula using a creep factor based on the grain size distribution. The hydraulic conductivity is dependent on the shape and size of the grain particles (Stoop, 2018), but since Lane's method was used this correlation had no effect. This means that for this case the project costs for the sheet pile wall remain constant even though technically there is probably less reinforcement needed. There are correlation methods between the grain size distribution and the hydraulic conductivity which can be used. However, these are only established as rough relations between the grain size distribution and the hydraulic conductivity, which may lead to an inaccurate assessment. What's more, the result from these correlation methods can vary by a factor of 10 (Berbee, 2018). Therefore these methods were not included in the cost calculations.

Excluding the effect the hydraulic conductivity by Lane's method has interesting effects for the benefit in monitoring and soil investigation in different project phases. If no hydraulic conductivity method is performed during the reconnaissance phase so the sheet pile wall is chosen as preferred alternative, later investigation into the horizontal hydraulic conductivity will have no benefit on itself as a more accurate result will not alter the design in this phase only. Even if the dike trajectory is safer than was first expected.

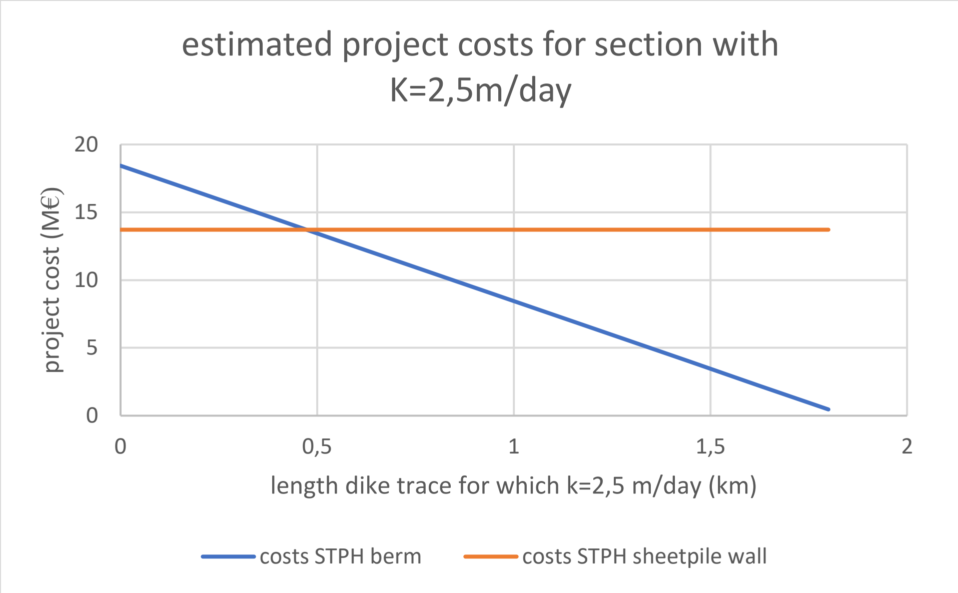


Figure 26 project costs for section of dike trajectory with Kh=2,5m/day

4.3.3 Effect due to effective foreshore length L_{eff}

Finally the effect of the effective foreshore length on the project costs for dike reinforcement of the dike trajectory needs to be determined. The project costs were determined for different partitions of the dike trajectory between sections were the foreshore could not be included and sections with an increase in seepage length of 20m or 40m.

The results for a seepage length of 75m are comparable To those for K=5m/day for the piping berm, while the length of the required sheet pile wall is also decreased for the longer seepage length For a seepage length of 95 a dike reinforcement is not needed anymore, so the project costs in Figure 28 can possibly be reduced to 0. In contrast with the results for the hydraulic conductivity, the project costs of the sheet pile wall due the change in horizontal seepage is included in Lane's method. But the effect on project costs for the sheet pile wall method is still lower than the effect for the piping berm. Even when according to Sellmeijer's equation no extra reinforcement is needed, Lane's method still results in a needed dike reinforcement. Lane's empirical method is in this case the more conservative approach.

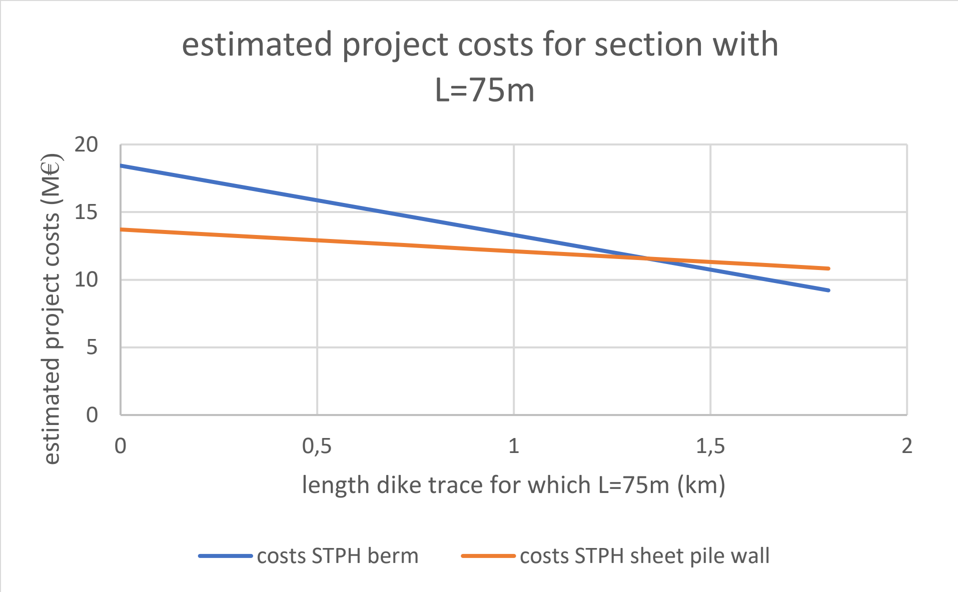


Figure 27 project costs for section of dike trajectory with 20m effective foreshore

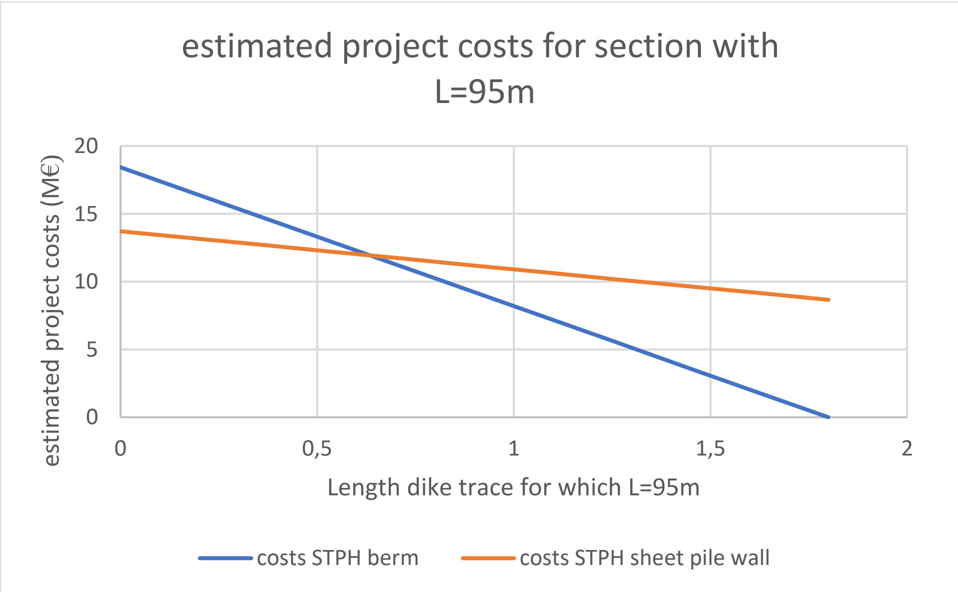


Figure 28 project costs for section of dike trajectory with 40m of effective foreshore

As mentioned previously, the effect of the increase in seepage length by the accounting the effective foreshore on the project costs is dependent on the length of time for the measuring period and therefore the probability of measuring the hydraulic response as well. The calculated project costs in Figure 27 and Figure 28 show the results if the required response is measured. So the probability of measuring during a water level which is high enough to determine the hydraulic response under the dike still needs to be included.

4.4 Effect on the realisation phase

The main approach of this research was to investigate what will be the effect on the project costs if information from the design phase was available earlier in the project. For the design phase this has no effect, since all this data is assumed to be available in this phase However, this does not mean the uncertainty cannot be reduced. For this project phase the effect of larger variation in the parameters is compared to the increase in project costs that comes with the variation. Prob-2B was used for the analysis.

4.4.1 Effect of coefficient of variation (c.o.v.) on the required berm length

It is possibly beneficial for the project in terms of costs to reduce the statistical uncertainty in the parameters in the used model. This can be done by collecting more data by performing more measurements, if little data is available. In this chapter, the benefit of reducing the statistical uncertainty in the model is analysed by varying the coefficient of variation of two of the analysed parameters, and to determine what this does with the required piping berm length for the dike reinforcement. Since the distribution of the effective foreshore is unknown, the choice of distribution is compared between the used lognormal distribution in the analysis and a uniform distribution. The coefficient of variation for the equivalent mean value of the analysed parameters is equal to 0,1.

The results for a uniform distribution of the length of the effective foreshore are shown in Table 9. What is interesting is that the higher the range for the effective foreshore becomes, the closer the equivalent berm length comes to the lower bound of the uniform distribution. This could be caused by that as the values for the uniform distribution increases the higher the α-value becomes and the more the value for the effective foreshore influences the failure probability. As the FORM calculation determines the most likely combination of parameters leads to failure, if the influence of the foreshore on the probability of failure increases the most likely value of failure for the effective foreshore will be low.

To compare the effect of different distributions on the failure probability of the dike, the equivalent mean value was determined. The equivalent mean value is the value of the parameter the mean needs to have to get the same failure probability as the newly specified distribution. The used c.o.v. for these parameters is 0,1. For example, for a uniformly distributed effective foreshore the equivalent mean value is 5m. This means the failure probability for a uniformly distributed effective foreshore between 0-10 is the same for if it has a normal distribution of 5m as mean and 0,1 as c.o.v. if all the other parameters are constant.

Uniform distribution effective foreshore (m)	Required berm length (m)	Equivalent mean value (m)	cost per m dike trajectory (€*10^3)
0	40	0	10,2
0-10	35	5	8,9
10-20	25	15	6,4
20-30	13	27	3,3
30-40	0	40	0

Table 9 effect of statistical uncertainty of the effective foreshore on the berm length

The effect of larger uncertainty in the value of the grain size distribution is also analysed, for which the results shown in Table 10. The impact of a coefficient of variation seems large, as the required berm length to meet the safety standard is more than doubled for a c.o.v. of 0,5. What is also interesting is that the required berm length becomes larger with increasing steps. A larger uncertainty in the grain size distribution can therefore lead to a considerable increase in the cost per m dike reinforcement.

But uncertainty in the grain size distribution may be unavoidable in dike design. The spatial variability of the grain size distribution in the subsoil is large, combined with a small correlation length (Kanning, 2014). It was also found that a characteristic value within the confidence interval of 95% might not be sufficient to compensate for the spatial variability of the d_{70}. Assuming a larger c.o.v. for the d_{70} might then be more expensive, but possibly also the safer decision.

c.o.v. d_{70}	Required berm length (m)	Equivalent mean value (µm)	cost per m dike trajectory (€*10^3)
0,1	40	180	10,2
0,2	43	170	11,0
0,3	52	160	13,3
0,4	62	140	15,9
0,5	75	130	19,2

Table 10 effect of statistical uncertainty of the d_{70} on the berm length

The same effect between the c.o.v. of K and the required berm length occurs as with the d_{70}. This effect is demonstrated in Table 11. Since the hydraulic conductivity is also correlated with the grain size of the particles (Stoop, 2018), the hydraulic conductivity also varies over space. But the hydraulic conductivity is also dependent on other factors, such as heterogeneity of the aquifer.

c.o.v. K	Required berm length (m)	Equivalent mean value (m/day)	cost per m dike trajectory (€*10^3)
0,1	40	10	10,2
0,2	43	10,5	11,0
0,3	49	11	12,5
0,4	53	12	13,5
0,5	63	13	16,1

Table 11 effect of statistical uncertainty of the permeability on the berm length

4.5 Difference effect monitoring and soil investigation on project costs

4.5.1 Difference results monitoring and soil investigation

Before the benefit in project costs can be determined, the difference between soil investigation and monitoring must be described. For the determination of the grain size distribution and the hydraulic conductivity of the aquifer, a point investigation can performed in the form of drilling a probe, taking soil samples or performing an HPT boring test. These methods analyse the soil composition regardless of time, which means that there will always be a resulting measurement of such a measuring method.

In contrast, when monitoring the hydraulic head in the aquifer, a high water level is needed to measure the response of the head in the aquifer (BZ, 2018). This means placing the piezometers into the aquifer will not immediately result in relevant data. Instead, the chance of recording relevant data becomes larger the longer the piezometers are into place.

The difference between these two measuring method categories is also reflected in the estimated project cost reduction. When performing a point investigation such as soil investigation or HPT boring test, the data is available once the measurement is finished and analysed. This means the benefit in estimated project cost reduction for the project cost for soil investigation is only dependent on the result of the measurements and in which project phase the soil investigation is performed. When the soil investigation is performed affects the estimated benefit because it determines in which project phases the result of the measurement can be used.

To illustrate the effect of the project phases on the value of data, the effect of observing different soil parameters in the described project phases is elaborated. To do so the soil characteristics of scenario 3 are used, which are shown in Table 12. The length of each section is 600m. The effect of the monitoring period on the expected cost is not yet included, which will be explained in chapter 4.5.2.

Soil characteristics dike trajectory	**sections**	**d_{70} (µm)**	**K (m/day)**	**L (m)**
Three sections	1	180	5	95
	2	255	10	75
	3	255	5	55

Table 12 used parameters for an example of the effect seepage length in different project phases

The estimated project costs for both reinforcement methods used were calculated for all three of the sections of the example (Table 13) using the semi-characteristic analysis methods described in Appendix B and the estimated project costs per m^2 determined in Appendix E.

section	Required length piping berm	Estimated project costs piping berm (M€)	Required length sheet pile wall	Estimated project costs sheet pile wall (M€)
1	0	0	12	2,9
2	7	1,1	12	2,9
3	8	1,3	15	3,6

Table 13 estimated project costs per reinforcment measure

Three project decisions have been defined in Chapter 3.2 related to the design. These are defining the project scope, choosing the preferred alternative and determining the exact dimensions of the reinforcement measures.

These project decisions are performed with the information available in the corresponding project phase. Data available in prequel project phases can also be used in the subsequent phases. If the results from the measurements are available in the initiation phase, it can be used for all three project phases. When available in the reconnaissance phase, the project scope is defined based on the base scenario. This means even when not necessary, the entire dike trajectory is included in the scope. A small dike reinforcement is included for sections which did not require reinforcement, in the case a sheet pile wall required when using Lane's method for the given soil characteristics.

When the results are only available in the elaboration phase, a preferred alternative is already chosen based on the base scenario and only the specific dimensions of the reinforcement can be determined based on the new information, i.e. the length of the sheet pile wall. When the preferred alternative is chosen based on the base scenario, the cheapest solution is a sheet pile wall. The exact calculations are elaborated in Appendix E. When this principle is applied to the soil characteristics in Table 10 using the estimated project costs Table 11, than the estimated project costs can be determined (Table 14).

Phase the different soil characteristics are observed	Initiation phase	Reconnaissance phase	Elaboration phase
Estimated project costs section 1 (M€)	0	2,9	2,9
Estimated project costs section 2 (M€)	1,1	1,1	2,9
Estimated project costs section 3 (M€)	1,3	1,3	3,6
Total estimated project costs (M€)	2,3	5,2	9,4
Estimated project costs reduction (M€)	11,4	8,5	4,3

Table 14 estimated project costs for the parameters given in Table 12

To illustrate the effect when the results are available during the project, two examples are compared. One example for which the results are available in the initiation phase and one where the results are available in the reconnaissance phase. The first example shows the estimated project cost reduction given that the soil characteristics from Table 12 were known in the initiation phase (Figure 29). This means the data acquired from the soil investigation can be incorporated in all the decisions on design, such as the preferred alternative of dike reinforcement and the exact dimensions of the dike reinforcement. The figure shows what the estimated reduction in project costs is for knowing the soil characteristics for each individual project phase, i.e. the estimated project cost reduction for the reconnaissance phase is the estimated reduction achieved by choosing the cheaper alternative of dike reinforcement.

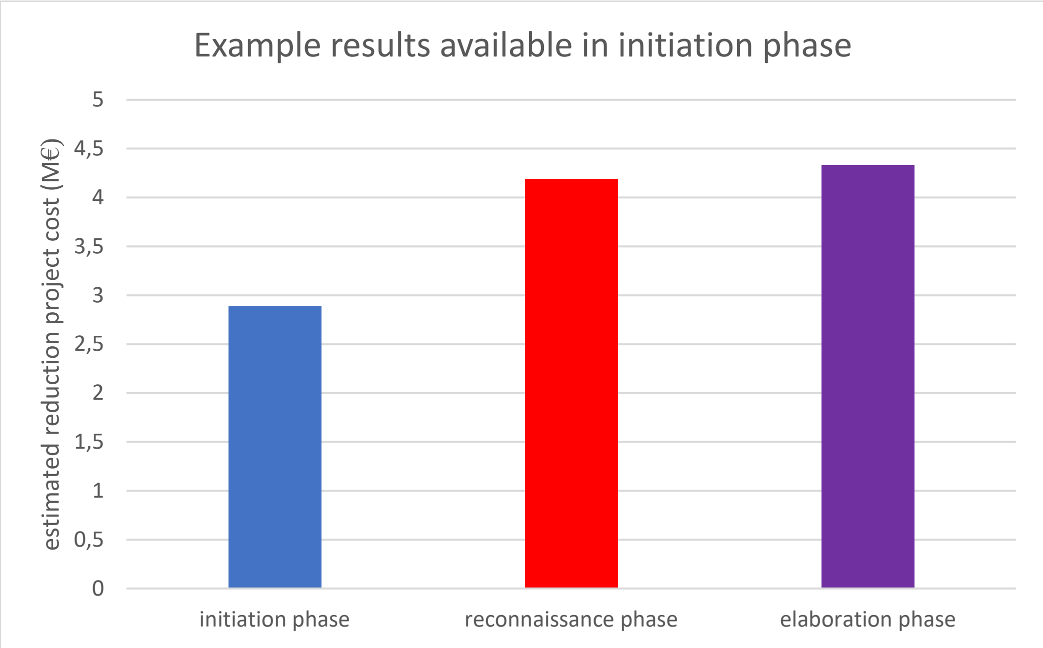


Figure 29 example results for soil investigation in the initiation phase

The second example shows the estimated project cost reduction given the soil characteristics from Table 12 were known later, namely in the reconnaissance phase. This influences the total estimated reduction in project costs. As the data cannot be used the initiation phase, which means a project decision has been made based on insufficient data for the subsoil. Instead, conservative assumptions were applied which lead to a more expensive design. Therefore the value of the data in the project was reduced. This is illustrated in Figure 30. If the information has no added value for the project phase however, it is also possible no reduction in the estimated project costs in the phase is found even if soil investigation took place beforehand. For example, if the measurement data does not lead to a project scope reduction, the value of the data is not increased for starting in the initiation phase.

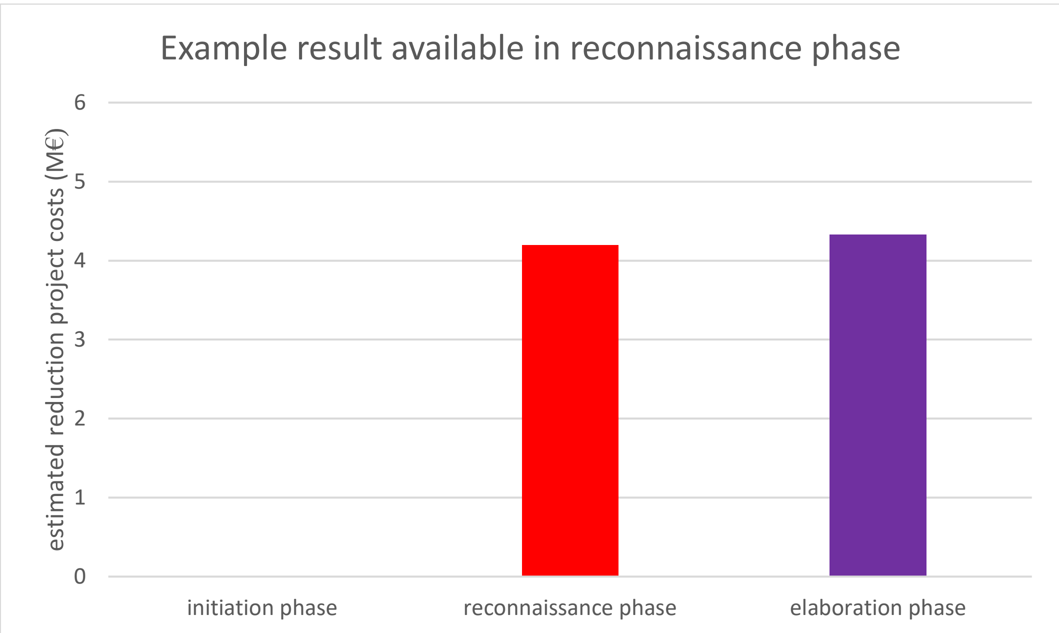


Figure 30 example results for soil investigation in the reconnaissance phase

Due to the effect described above of the availability of data during the project phases on the estimated project cost reduction the value of data is decreased as the data is measured later in the project. But the total value of the data is immediately obtained, as the collected data does not change over time. This value is collected during the project decisions when the data is used.

The value of the measurements for monitoring is obtained differently. Once the piezometers are placed, relevant data is not immediately available. This process takes time. Since the probability of measuring relevant data increases as the measuring period is prolonged, the estimated project cost reduction is then increased. The influence of the measuring period is compounded on the effect in which project phase monitoring was started, as the value of the data is still depended on in which project decisions the data is used. This means both the impact of the measurements on the project decisions as the probability of measuring relevant data influence the value of the information. But instead of immediately receiving the value of the measurements such as for soil investigation, the value of monitoring data actually increases as the monitoring period progresses. This is shown in Figure 31.

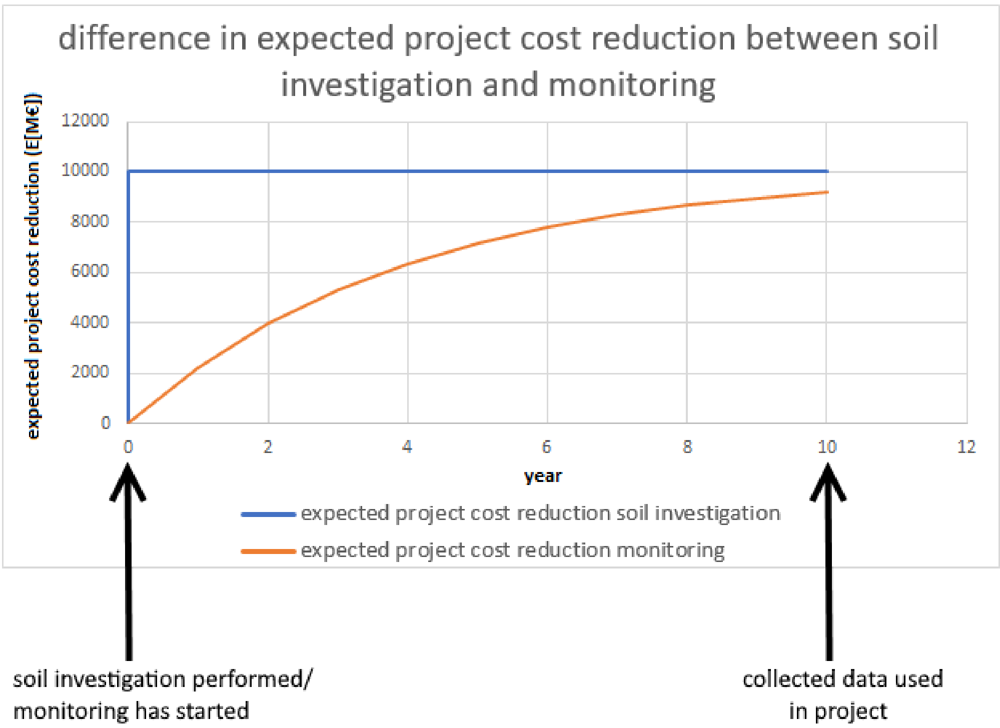


Figure 31 difference between point investigation and monitoring

In Figure 31 the estimated project cost reduction from the result of both monitoring and soil investigation is €10.000,-. For the soil investigation the value is immediately attained as the information is available as the soil investigation is performed. Monitoring happens continually for those 10 years. At the start of monitoring, no data has been collected. As more time passes, the probability of exceeding the threshold value increases (see chapter 4.5.2), and so does the expected project cost reduction. The expected project cost reduction is the estimated project cost reduction multiplied with the probability of exceedance of the threshold water level calculated with Equation 9. If monitoring continues for long enough the expected project cost reduction will eventually reach approximately E[€10.000,-].

4.5.2. Effect of monitoring on project costs

The results determined in the previous sections are based on the assumption that if soil investigation or monitoring takes place, the difference between the representative cross-section and the actual circumstances at the dike trajectory are always found. This is not necessarily the case, as there is inherent uncertainty in measurements that needs to be taken into account. Measurements such as soil investigations or in-situ hydraulic conductivity determination methods such as HPT-borings are point measures. Any results from these tests only provide insight in the parameters on that specific point, and any other parameters over the length of the trajectory need to be inferred. Based on the correlation length the spatial variability can be very large for short correlation length and smaller for long correlation length.

When measuring the entry point by monitoring the hydraulic head in the aquifer the uncertainty over time also becomes a factor. A monitoring system can be in place, but then the results still depend whether for example high water levels high enough for the system to record the response of the hydraulic gradient occur.

This uncertainty has to be taken into account to be able to more accurately predict the expected profit in reduced project costs. For the measurements of permeability and grain size distribution, this means incorporating the spatial variability. As for monitoring, the occurrence of high water over time also needs to be incorporated. The interesting part for this research is determining when in the project is most effective to start monitoring, as the longer the measuring period the higher the chance of recording high water levels.

In order to calculate a probability of exceedance a value needs to be determined which the water level must exceed. For this research it is assumed this value is a threshold value which must be exceeded during monitoring to record the relevant data for the design. To determine the probability of a water level exceeding a certain value during a given period of time, the relation of the annual maxima water levels is assumed to be independent. The probability of a high water event exceeding a certain value during the time period can then be calculated using Equation 9. The effect of the duration of the measuring period on the probability of exceeding different water levels is illustrated in Figure 32.

$$p(X > x) = 1 - (1 - \frac{1}{T})^n \quad [9]$$

n=number of years
T=return period

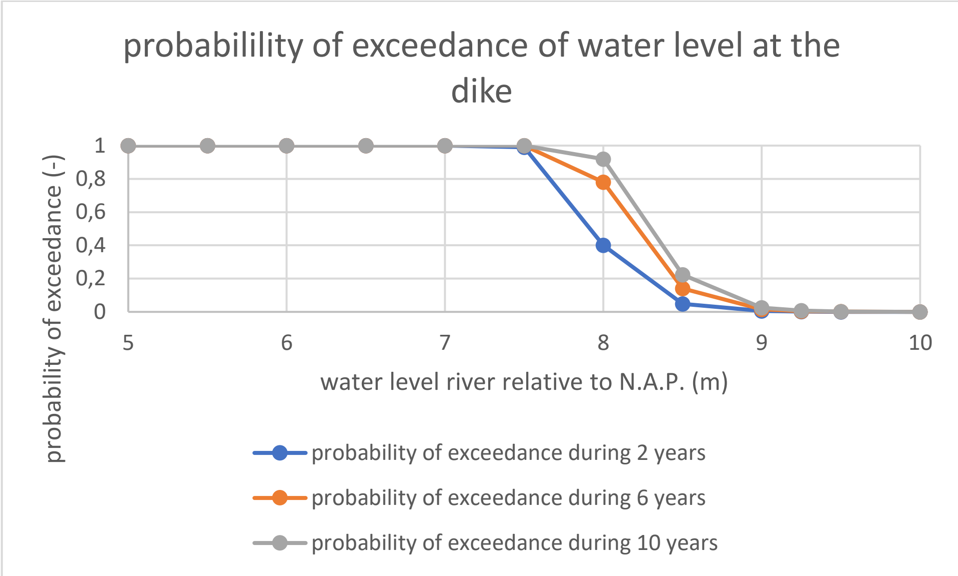


Figure 32 probability of exceedance of different water levels given the period of time over which is measured

Hydraulic head measurements in the aquifer during high water are very useful to determine the effect of the foreshore on the hydraulic response in the aquifer (Tonneijck, 2018). Aspects such as the heterogeneity of the cover layer, entry points and local differences due to holes and ditches can be incorporated in a bulk characteristic for the dike section. But to determine what the benefit of monitoring will be in terms of reduction in project costs, a criteria is needed of what water level in the river is needed for relevant information on of the hydraulic response in the aquifer to prove the foreshore influences the seepage length.

For this reason an assumption was made that a certain minimum water level or higher needed to occur during the monitoring period to be able to measure the fictional entry point and so the length of the effective foreshore using the measured water head in the aquifer over the cross-section of the dike. This water level is defined as the threshold value. The measurements have to be performed at multiple locations to verify the effect along the whole dike trajectory. In this case an interval between cross-sections over which the sensors are placed is 100m.

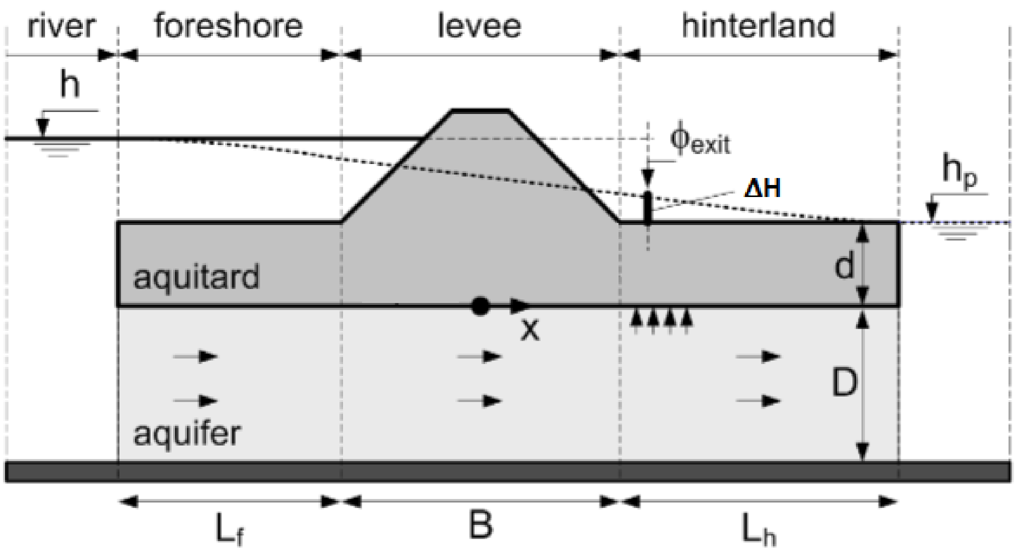


Figure 33 head difference at the toe of the dike (adapted from Jonkman, 2018)

To determine a suitable threshold value for the case, different threshold values were compared (Figure 34). The head difference between the head in the aquifer and the phreatic level in the hinterland (Figure 33) was plotted for different seepage lengths. First the return period must be high enough that such a water level in the river has a significant chance of occurring during the project. This eliminates the design water level of 9,25m +N.A.P. Furthermore it was verified there is a sufficiently high head present in the aquifer at the toe of the dike which can be distinguished from daily conditions.

$$\Delta H = \phi_{exit} - H_p \quad [10]$$

ΔH= head difference
Φ_{exit}=head in the aquifer at the toe of the dike
H_p= hinterland phreatic level

A threshold value of 8m above N.A.P. was chosen. The reasons are that for this threshold value that even for the maximum seepage length of this case of 95m the head difference is larger than 1m, which is assumed to be sufficiently high to be distinguished from daily conditions. The head difference is defined as the difference between the phreatic line in the hinterland and the head at the toe of the dike (Equation 10). Furthermore the head difference is larger over the seepage length than for example for a water level of 7m above N.A.P. In comparison, the difference between a seepage length of 55m and 95m is approximately 0,5m at a threshold value of 8m +N.A.P. and only 0,3m for 7m +N.A.P. Since the difference in head difference over the seepage lengths is larger for 8m +N.A.P., it is easier to determine what the seepage length is.

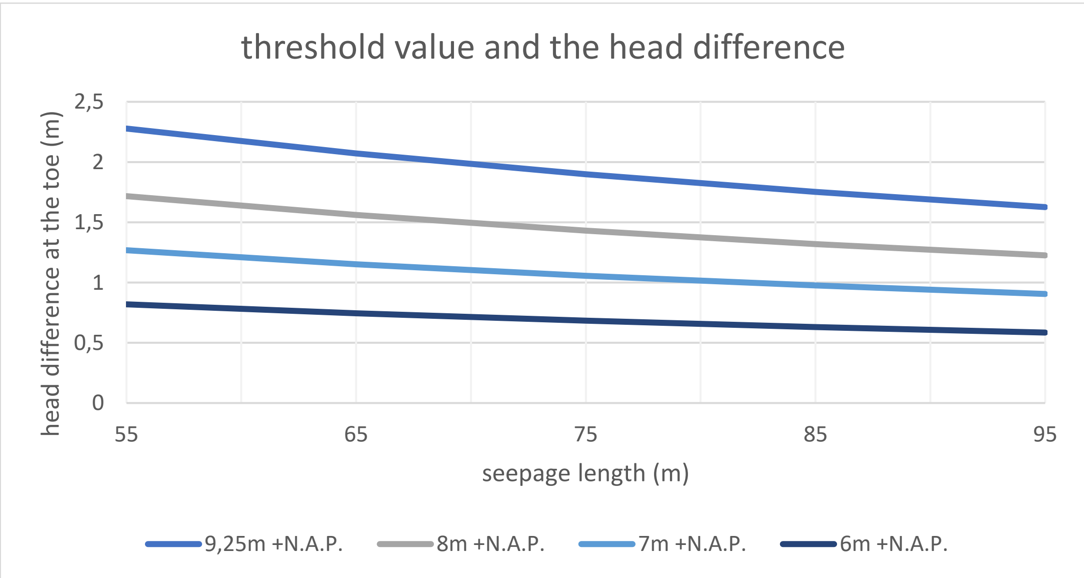


Figure 34 head difference between head in the aquifer and the phreatic line for different threshold values.

In the end, the threshold value was chosen due to the if the difference between the head in the aquifer under normal conditions and the head in the aquifer when the water level in the river exceeds the threshold value is large enough, and if the difference in head measurements across the aquifer were deemed large enough. This choice affects the expected benefit in project cost reduction in monitoring, as it determines the probability of exceedance used as the probability to obtain the required data to verify the impact of the foreshore. A lower threshold value leads to a higher expected benefit, as the probability of exceedance is increased. However the expected benefit of a longer monitoring period will be lower, as a shorter measuring period is required to achieve the same probability of exceedance during the monitoring period for a lower threshold value. The opposite is true for a higher threshold value, for which the expected benefit will be lower but the benefit of a longer measuring period will be higher.

From now on the expected benefit is used instead of estimated benefit. The estimated benefit is expressed in euros only, but for the expected results probability is also included in the benefit. Therefore the benefit becomes an expected value. For example, if the estimated benefit is 1 M€ and the probability of gaining this benefit is 0,5, then the expected benefit becomes E[0,5 M€].

For Figure 35 scenario 2-G-… is used. In short, the soil characteristics for scenario 2-G-… consist of an effective foreshore of at least 20m for a length of about 900m (50% of the length of the dike trajectory) and no effective foreshore being found for the other 900m. The threshold value is 8m above N.A.P, which is the water level in the river that needs to be exceeded for relevant data to determine the seepage length to be recorded.

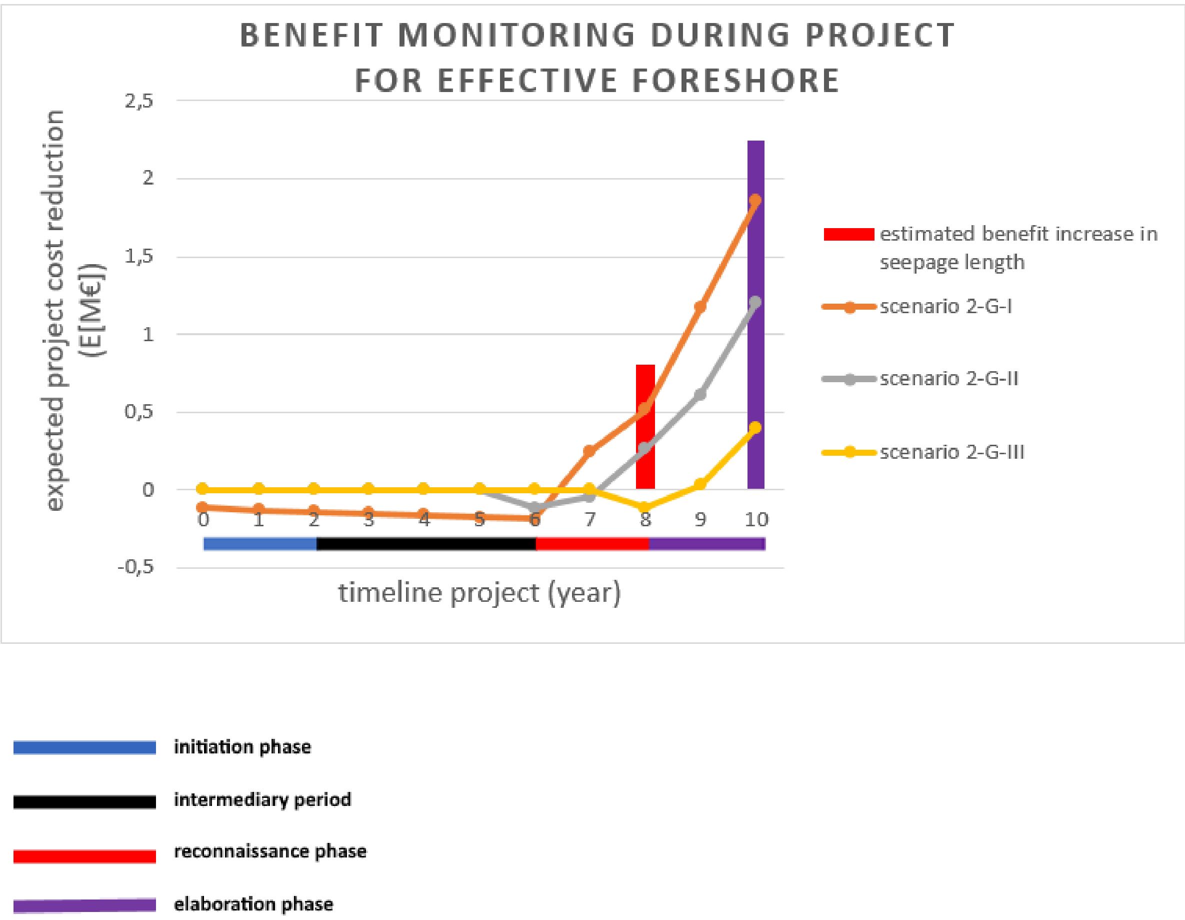


Figure 35 monitoring of the effective foreshore for a signal value of 8m N.A.P.

The estimated project cost reduction for each phase depends on what project decision the data from the measurements can be used for. If for example scenario 2-G-II is used, the monitoring plan was implemented in year 6 the probability of the water level at the case location exceeding 8m above N.A.P. in a period of 2 years is used to determine the benefit of knowing there is a longer seepage length than first anticipated for the choice of the preferred alternative in year 8. And a measuring period of 4 years is available for to determinate the exact dimensions of the preferred dike reinforcement alternative in year 10.

On the x-axis of Figure 35, a timeline of the project is represented. This timeline runs from the initiation of the project until the start of the realisation phase. The project period for this case is 10 years. The phases in a dike reinforcement project is described in Chapter 3.2. The estimated benefit for each project phase has a different colour assigned to bar. Blue is the reconnaissance phase, red is the elaboration phase and purple is the elaboration phase. Since there is no expected benefit for this scenario in the initiation phase, there is no blue bar in Figure 35.

The expected cost reduction are calculated by interpolating the estimated project cost reduction over its project phase, and then multiplying the estimated project cost reduction with the probability of exceeding the threshold value during the monitoring period. The estimated investment cost is also subtracted from the expected project cost reduction. This is done separately for each project phase in which monitoring can start. To illustrate, the described calculations are shown for performing monitoring in the initiation phase for the soil characteristics of scenario 2 (Table 15). The complete calculations for the expected project cost reduction due to monitoring can be found in Appendix D.

Year	**0**	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
Estimated project cost reduction (M€)	-	-	-	-	-	-	-	-	0,8	-	2,2
Interpolated project cost reduction(M€)	0	0	0	0	0	0	0	0,4	0,8	1,5	2,2
Probability of exceedance water level during monitoring period (-)	0	0,22	0,40	0,53	0,63	0,72	0,78	0,83	0,87	0,90	0,92
Expected project cost reduction minus investment costs(E[M€])	0	0	0	0	0	0	0	0,33	0,70	1,37	2,07
Estimated investment cost monitoring (M€)	0,11	0,13	0,14	0,15	0,16	0,17	0,19	0,2	0,21	0,22	0,23
Expected project cost reduction (E[M€])	-0,11	-0,13	-0,14	-0,15	-0,16	-0,17	-0,19	0,13	0,49	1,15	1,84

Table 15 expected project cost reduction due to monitoring for scenario 2-G-I

The total expected project costs are approximately €13,7 million. This results in an expected project cost reduction of approximately 15% when monitoring is started in the initiation phase. Two trends can be seen in Figure 35 which influence the expected project costs reduction. There is the value the data has in their respective project phases, and the increase in the probability of exceeding the threshold value while monitoring as the monitoring period becomes larger. The value of the data is collected at the end of each project phase. These are at the end of the initiation phase at year 2, the end of the reconnaissance phase at year 8 and the end of the elaboration phase at year 10 (see chapter 3.3). The relation of the estimated value of the data between these points is assumed linear.

Besides the value of the data there is the increase in probability of monitoring during the right conditions to determine the fictional entry point. As the period in which monitors increases, the probability of monitoring during the right conditions increases. For Figure 35 it is assumed these conditions occur when the water level in the river is 8m +N.A.P. or higher. As the probability of exceeding the threshold value increases, the expected project cost reduction is also increased.

If a seepage length of 75m is measured in the dike trajectory and the other parameters are as described in the base scenario, there is no scope reduction. This increase in seepage length is less than the required seepage length. A scenario in which part of the dike the seepage length is larger than the required seepage length and no dike reinforcement is needed was analysed in Figure 36. For these results scenario 3-G-I, II and III were used. The soil characteristics for this scenario are defined as 3 characteristic cross-sections for equally large sections of the dike trajectory, one with L=55m, one for which L=75m and one section for which L=95m. This shows that implementing a monitoring plan at year 0 for these possible results leads to a scope reduction for the project. It is assumed that if the monitoring plan is implemented during or after this point in year 2 this benefit is lost.

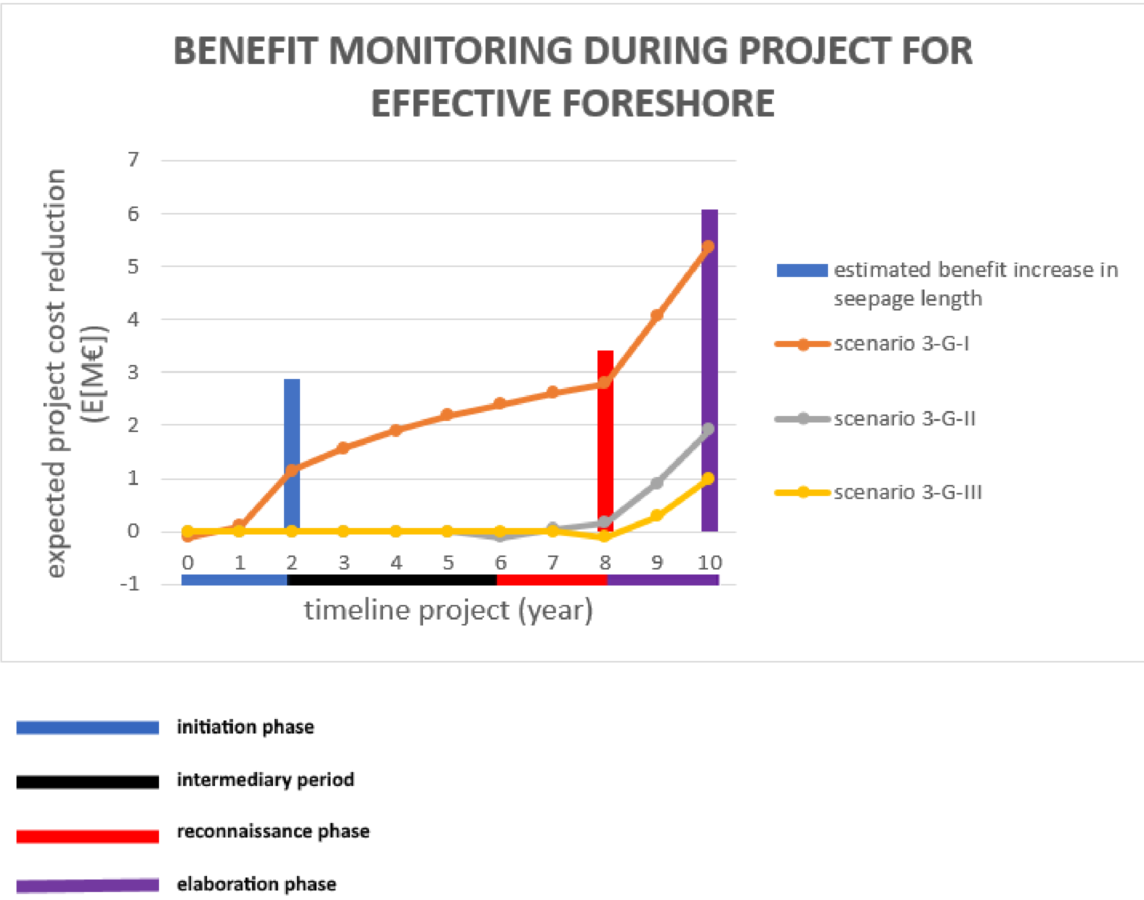


Figure 36 monitoring the effective foreshore for a signal value of 8m N.A.P. with a section for which Leff=40m

The results in Figure 36 show a different trend line for the expected project cost reduction when started in the initiation phase (scenario 3-G-I). This is because for this scenario there is an estimated benefit for performing monitoring in the initiation phase, as the project scope and therefore the required dike reinforcement can be reduced. This means more expected project cost reduction can be found in the earlier phase of the project. This is under the assumption that once the project scope has been set, it can no longer be changed. Furthermore, the expected project cost reduction in Figure 36 is also higher in general than found in Figure 35. This is in part by the reduced project scope found in Figure 36, and in part due to the longer seepage length in scenario 3 which reduces the required dike reinforcement.

What should be considered that there is not one threshold under which the values are unusable. Even measurements of the hydraulic head over the cross-section during lower water levels can be useful to modelling piping under the dike. This means the usefulness of the measurements will increase as the water level measured increases, instead of that all potential value is collected as the threshold is crossed. So the expected benefit will be higher if lower water levels in the river also prove useful. But this will have to be verified using actual monitoring data that was collected.

4.6 project cost reduction over the project phases

After the analysis of the effect of monitoring and soil investigation for each project phase has been performed, the effect on the project as a whole can be determined. In particular the effect of performing soil investigation and to start monitoring in different project phases is analysed. The impact on the project costs of when measurements takes place is important as it gives an indication on when in the project which measurement needs to be taken.

Two possible outcomes of the measurements are taken into account. First outcome is that sections of the dike trajectory contain stronger parameters. Second outcome is that the entire dike trajectory has stronger parameters than specified in the base scenario.

The result of the safety assessment before the start of the project is typically a representative cross-section of the dike trajectory. The first outcome of a representative cross-section of the dike trajectory is too conservative for the sections with stronger parameters. A more accurate representation leads to reduced project costs. This is because the representative cross-section for the dike trajectory is constructed from the most conservative parameters found in the entire dike trajectory. It is highly likely not all of the dike trajectory is as vulnerable to piping as is assumed in the representative cross-section. A better design approach is to specifically identify the weak spots in the dike and reinforce the dike at these spots. This principle is shown in Figure 37 (Tonneijck, 2018).

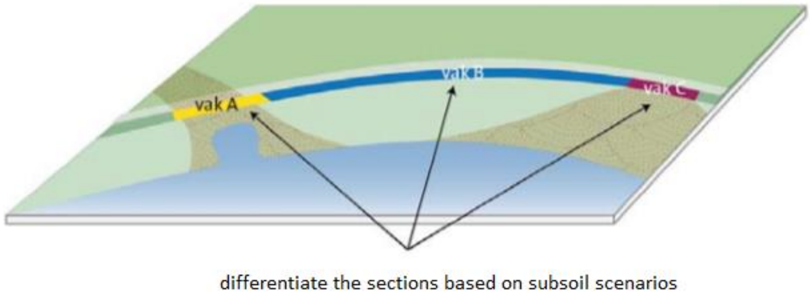


Figure 37: simplified example of schematising the dike sections based on the subsoil (Tonneijck, 2018)

The second outcome means the measurements used for the representative cross-section were inaccurate. This implies the mean used parameter needs to be adjusted for the entire trajectory. The possibility of an inaccurate representative cross-section for the dike trajectory is treated later in this chapter.

To estimate the impact of the parameters on the expected project costs, first the effect on project costs for a different mean value of the parameter were considered individually. The used scenarios are described in chapter 3. The used soil parameters are shown in Figure 38 and Figure 39. The weakest mean value of the parameters measured in the dike section was used as representative mean value of that section.

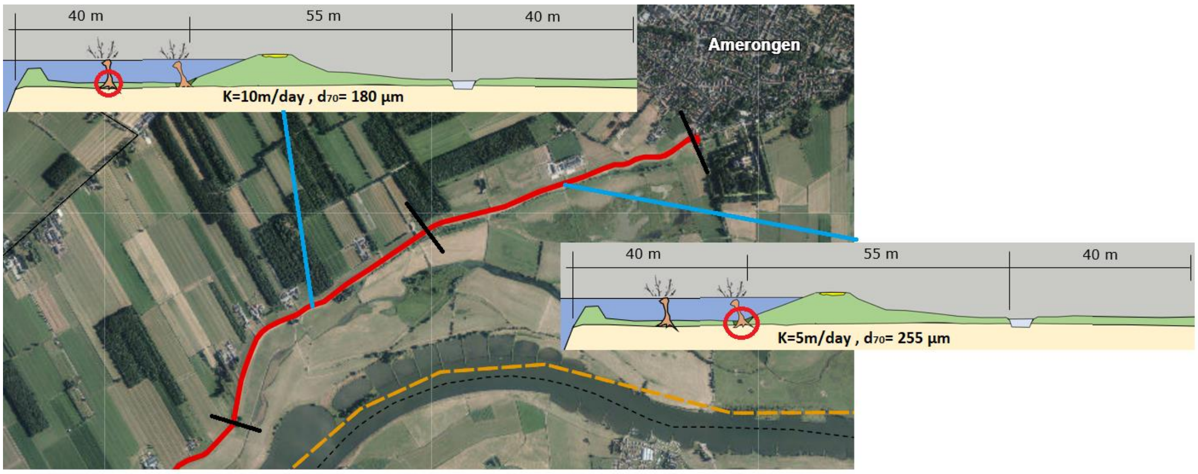


Figure 38: alternative schematisation 1 for the dike trajectory (scenario 2-B-…)

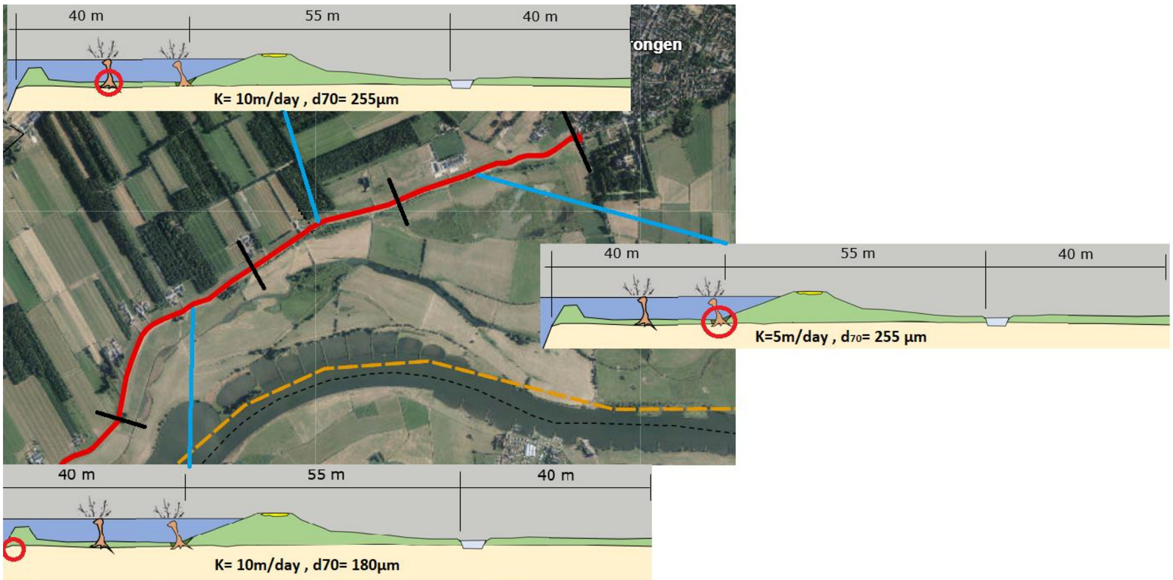


Figure 39 soil characteristics subcategory of the scenarios, number 3-…-…

As stated, the measurement results from soil investigation and monitoring are based on available soil samples and other data. The available data is mostly general classifications. For example, the d_{70} value was calculated as the mean value of the range of grain sizes of the lithological class of the soil. The actual value of d_{70} for the dike trajectory at Amerongen will likely be different than the theoretical values used for this case. The true distribution of the parameter and therefore the expected reduction is unknown, which is illustrated in Figure 40. But the assumed d_{70} is within the same range of the lithology class of the soil, and as such can be used for estimation of the expected cost reduction.

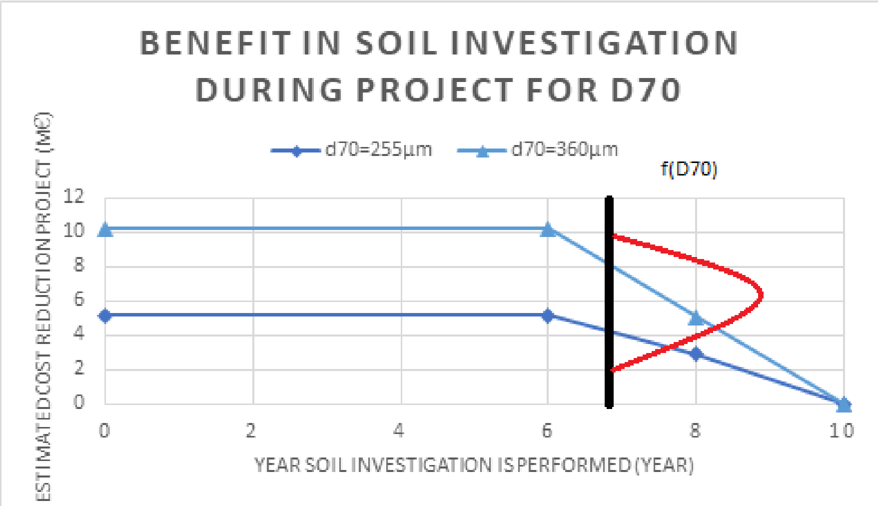


Figure 40: function of d_{70} illustrated over the results

4.6.1 Effect of grain size distribution over the project phases

First the effectiveness of soil investigation into the d_{70} during different project phases is be determined. This is done to determine in which period it is most effective to start with the measurements and to formulate a strategy for soil investigation.

The proposed soil investigation provides a more accurate assessment of the grain size distribution in the sand layers of the aquifer. As representative value of d_{70} for a dike section the smallest grain size found in the upper layer of the aquifer in that the section is used. The scenarios of sections with different soil characteristics used are shown in Figure 38 and Figure 39.

The resulting reduction in project costs is shown in Figure 41. The expected cost reduction for the project costs is the difference between project costs between the result of the scenario and the estimated project costs using the base scenario described in chapter 3. An explanation of how the expected project cost is determined can be found in Appendix E. It takes less than a year to complete the soil investigation.

In the results of the benefit of early monitoring and additional soil investigation the value at year 10 is set at 0. In year 10 the realisation phase starts, which is outside the scope of this thesis. There is a benefit in early monitoring and additional soil investigation in the realisation and maintenance phase for the dike reinforcement. But since this benefit is unknown the value is set at 0.

For this case a solely different value for d_{70} the expected project cost reduction of the soil investigation mostly does not change. The constant expected benefit is because the influence of the larger parameter is not sufficient to change the project scope in the initiation phase starting in year 0, or change the choice of dike reinforcement method in the reconnaissance phase starting in year 6. Only the dimensions of the sheet pile walls are reduced, for which the soil investigation must be performed before the end of the elaboration phase. Since the value of the data from the soil investigation is assumed to not change over time for soil investigation, the expected project cost reduction due to soil investigation does not change if performed earlier.

The exception to the results described above is the scenario for which the d_{70} of the dike trajectory is 360 μm. The cheaper alternative to dike reinforcement then becomes the piping berm, which means the data of the soil investigation can be used in the reconnaissance phase. When the soil investigation is started at year 6 the data can be used for the reconnaissance and the elaboration phase. If the soil investigation is performed in year 8, the reconnaissance phase is finished and the data can only still be used in the elaboration phase. This is based on the assumption that once the design choice has been made this part of the project is finished. In reality, the HWBP recommends that design choices made do not rule out other solutions further on in the project (Bernardini, 2017).

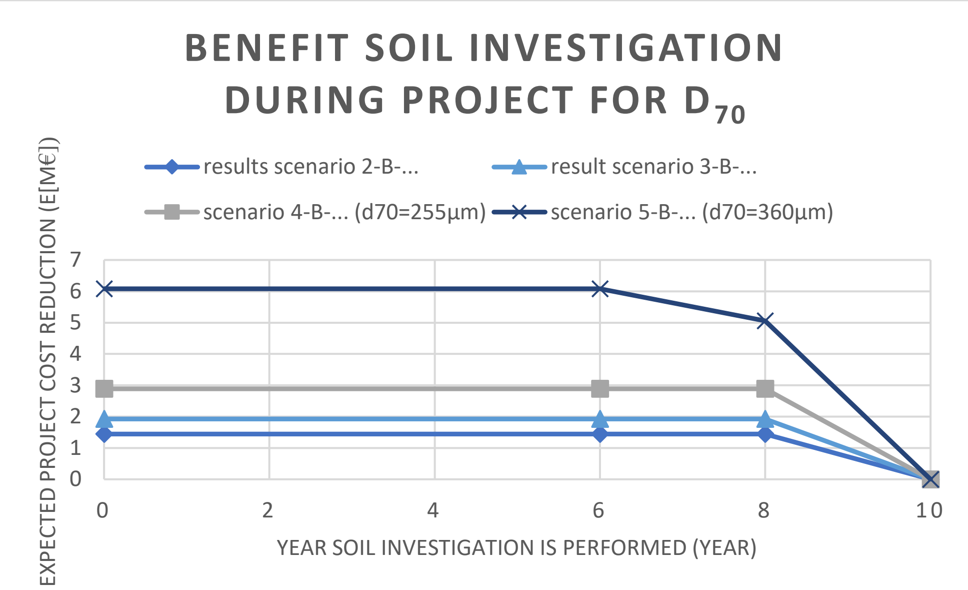


Figure 41 possible reduction in project costs due to more accurate schematisation d_{70}

A different mean value for d_{70} than the original parameter at the start of the project of the representative cross-section is possible when the results from the safety assessment were wrong or inaccurate. If the new parameter is a positive influence on the piping, this reduces the project costs for the entire dike trajectory. What the probability of a wrong or inaccurate parameter in the representative cross-section is depends on the quality of the available data and the amount of sources available.

It is possible that due to an inaccurate assessment of the situation a different mean value for the d_{70} can be found. But due to the high spatial variation found for the grain size distribution, even when the mean is higher sections with a small d_{70} can still occur in the dike trajectory. Even with small measurement interval no correlation length for the d_{70} was found (Kanning, 2014). But this is not a reason to simply take a conservative value for the d_{70} for the entire trajectory and not perform any further investigation into the soil. Even if there are weak spots in the dike trajectory, this does not take away that the rest of the dike might not be that vulnerable to piping as first suspected. Soil investigation still provides a more accurate image of the dike trajectory and which parts are vulnerable and which parts are already strong enough.

4.6.2 Effect of hydraulic conductivity over the project phases

The effect of investigating the hydraulic conductivity on the project cost over its different phases gives some interesting effects. Again the scenarios for which the parameter is different in sections or for the entire trajectory were used, for which the results are shown in Figure 42. The used hydraulic conductivities for the dike sections are shown in Figure 38 and Figure 39.

The results in Figure 42 show that the resulting reduction in product costs reaches 0 when the measurements take place at the start of the elaboration phase in year 8. This is due to the fact that the method of Lane does not incorporate the hydraulic conductivity in the aquifer directly, but instead uses a constant based on the soil type present. Since the preferred alternative for the base scenario is the sheet pile wall, starting measurements after the preferred alternative is chosen will result in a sheet pile wall as alternative. This is based on the assumption that after the preferred alternative is chosen it cannot be changed. It is however recommended to keep the option of a different alternative open during the project.

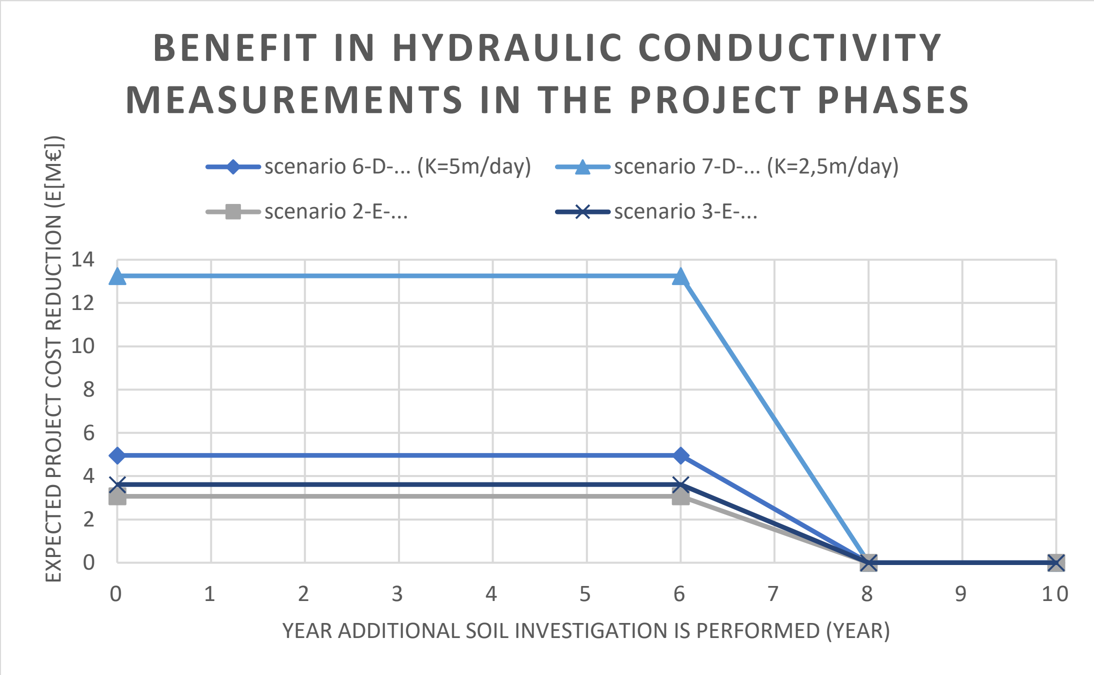


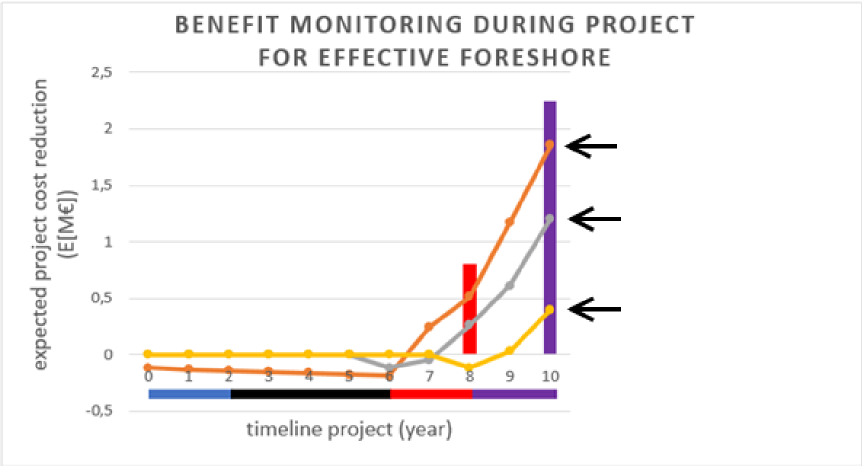
Figure 42 benefit of early monitoring over the design period for hydraulic conductivity

The results for scenario 3-E-…. May seem as if the expected project cost reduction is equal to the estimated project cost for the base scenario (13,7 M€), but for a hydraulic conductivity of 2,5m/day and all other parameters as in the base scenario, the cheapest required dike reinforcement is 1m of piping berm. So a very small reinforcement is still needed to meet the safety requirements.

4.6.3 Effect of the effective foreshore over the project phases

To compare the effect of possible measurement results of the effective foreshore with the other two parameters, the effect of the increased seepage length over the project phases will be discussed. As was mentioned before in the report, the results for the foreshore are different than for the d_{70} or K due to the difference between monitoring and soil investigation. But to compare the resulting reduction in project costs for the different parameters, the parameters need to be in the same framework. For this reason the results from chapter 4.5. have been converted to the same format as used for the possible results for the soil investigation and HPT boring tests. The results in this format are shown in Figure 44.

The results shown in Figure 44 a simplification, as it shows the result as if the value of the information is obtained in the same year as the monitoring has started. In reality, it takes time to collect the relevant data using monitoring (see Figure 43). But Figure 44 serves as comparison of the effect on project costs over the different project phases with the other analysed parameters. For a more detailed calculation of the project costs, please refer to chapter 4.5. .



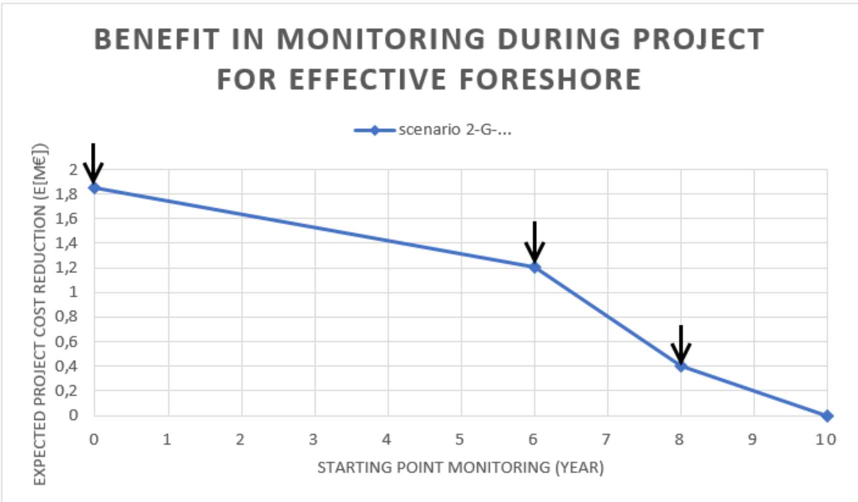


Figure 43 restructured results chapter 4.5.2.

The seepage lengths can be seen in Figure 38 and Figure 39. The points in the graph were determined by taking the final value of the monitoring paths shown in Figure 35 and Figure 36 to the corresponding starting point for monitoring. Furthermore the calculations were also performed for when the seepage length is longer for the entire dike trajectory.

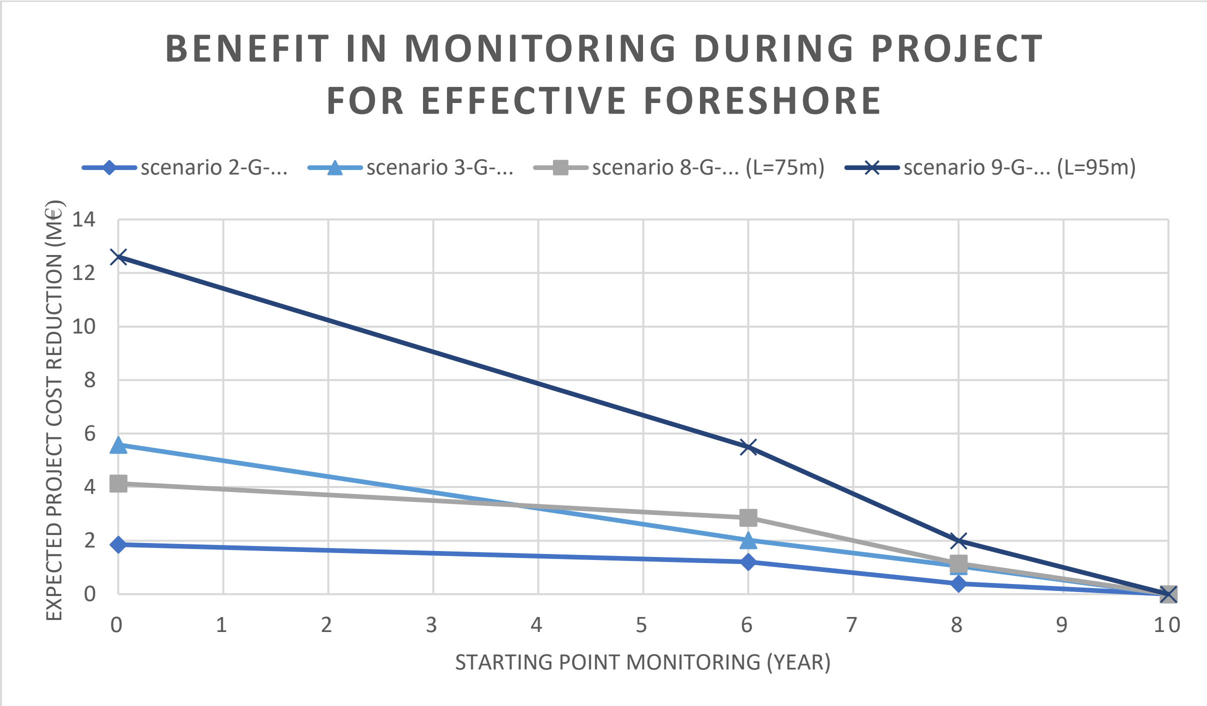


Figure 44 benefit of early monitoring for different scenarios of the seepage lengths

The expected results of cost reduction in the project for different years in which monitoring was started is shown in Figure 44. In contrast to the grain size distribution and the hydraulic conductivity of the aquifer, the impact of the seepage length increases as the monitoring is started earlier. This is due to two factors. First there is the measurement period required to collect relevant data. A sufficiently high water is needed to determine the fictional entry point and the longer the measuring period the higher the probability of such a high water.

Secondly for scenario 3-G-… and 9-G-… there is again the option that no dike reinforcement is needed for a part or the entire dike trajectory, which reduces the project scope. Without the influence of the willows or other weak points in the cover layer of the foreshore the possible increase in seepage length is more than 40m, which sufficiently long seepage length to meet the safety standards.

If there are no measurements to verify the foreshore affects the seepage length, the foreshore is not included in the model. As such it is possible that when measurements are conducted the hydraulic resistance of the foreshore for the entire dike trajectory is higher than was first foreseen. This means a possible reduction of needed reinforcement along the entire trajectory, which results are also shown in Figure 44.

The results are similar in shape to the results found for a longer seepage length in sections of the dike, but the resulting project cost reduction is larger if the seepage length is longer along the entire dike trajectory. One thing that should be noted is that the figure includes the probability of measuring during a high water. For this reason the expected value is not as high as the total possible project cost reduction.

What is interesting is that the scenario with a seepage length of 75m along the entire dike trajectory (scenario 8-G) has a higher expected project cost reduction when started from year 5, but when monitoring is started earlier scenario 3-G has a higher expected project cost reduction. This is because for scenario 3-G the project scope can be reduced when relevant monitoring data is available in the initiation phase (before year 2), therefore reducing the project costs.

4.6.4. Comparison between the parameters

To compare the effect of different soil parameters on the expected project cost reduction, the expected project cost reduction for different monitoring and additional soil investigation were compared (Figure 45). Each measurement method in the figure assesses a single parameter. The found parameters for the sections are: d_{70} is 255µm, hydraulic conductivity 5m/day and the effective foreshore is 20m. This leads to the dike reinforcements for that section shown in Table 16. The calculations for the expected benefit for each project phase given the soil characteristics can be found in Appendix E. The longer seepage length is dependent on if the threshold condition is met for monitoring the hydraulic head.

d_{70} (µm)	K (m/day)	L (m)	Reinforcement measure	Length measure needed (m)
180	10	55	Sheet pile wall	19 (base scenario)
255	10	55	Sheet pile wall	15
180	5	55	Piping berm	18
180	10	75	Piping berm	20

Table 16 preferred alternative and dimensions alternative for soil characteristics

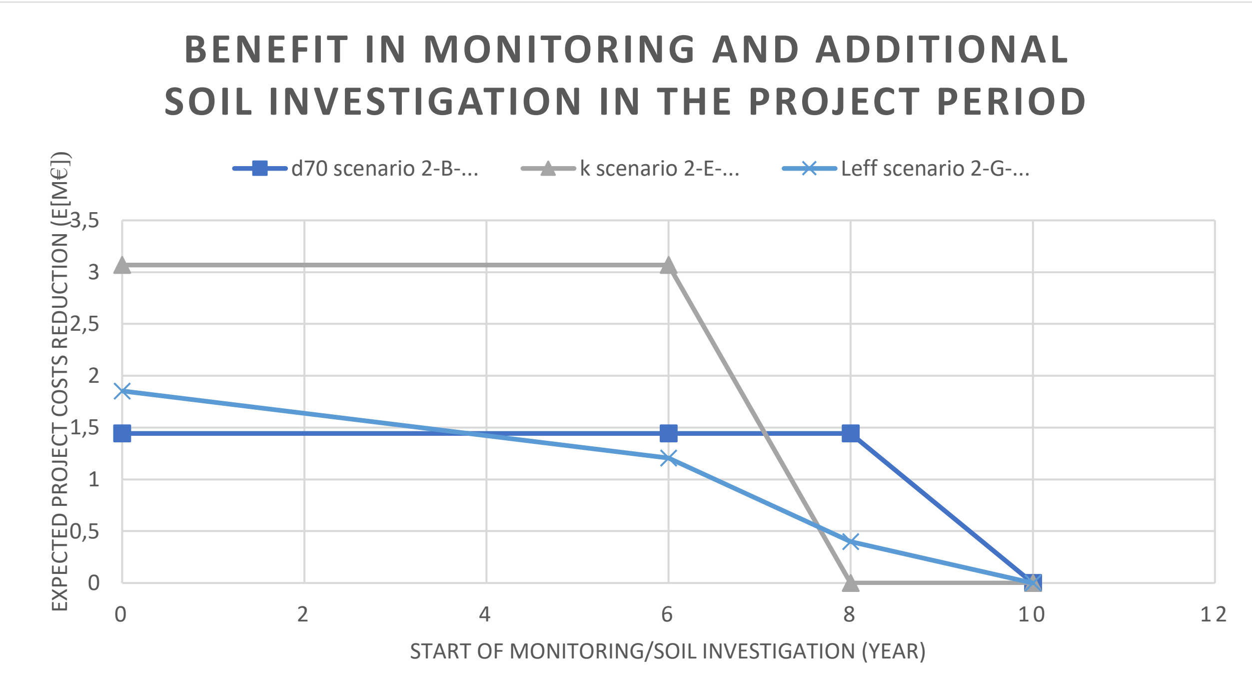


Figure 45 expected benefit in project costs over the project period

The results from Figure 45 seem to be mixed. The horizontal hydraulic conductivity leads to the highest project cost reduction for the earlier phases of the project. But due to the sharp decrease after the start of the reconnaissance phase (year 6) it becomes less favourable to perform HPT borings in later phases. Soil investigation into the d_{70} is the most effective method when the detailed analysis is started at the beginning of the elaboration phase(year 8). But it is generally less effective than the other parameters when monitoring and soil investigation starts earlier. The measurements on the seepage length has less expected benefit when monitoring starts later as due to the short measurement period.

There are 3 aspects of the effective foreshore which have to be taken into account for the results of Figure 45. First there is the difficulty in measuring the seepage length as a long measurement period is needed. Secondly there is the presence of obstacles such as trees and houses on the inner slope, which make it difficult to make a single model for a section which includes an effective foreshore. Too many elements will complicate the design, and it becomes harder to measure the effect of every single obstacle. And finally if the effective foreshore is included as part of the dike, the regional water authority will be responsible for the maintenance of the foreshore. This implies there will be extra costs which are not included in this research. The costs of buying and maintaining the foreshore need to be taken into account.

To summarize, HPT-boring are best performed in year 6 or earlier. Preferably, monitoring is started as soon as possible to have a larger probability of monitoring during a high water event. Finally for the soil investigation of the grain size distribution the best timing is as with the hydraulic conductivity is at the start of the reconnaissance phase. If the operations can be combined, investment costs are diminished.

4.6.5 Relation investment costs monitoring/soil investigation and project costs

In the previous chapters, the different parameters of the dike relevant to piping used for this research were treated separately. However, the parameters are not independent. Some factors might coincide, which possibly results in a different preferred alternative for dike reinforcement or dimensions of the alternative.

The parameters can also negatively affect each other in terms of vulnerability to piping. The hydraulic permeability of the aquifer is dependent on the grain size and shape, for which smaller grains typically have a smaller hydraulic conductivity (Ritzema, 1994). But grain size is not the only factor determining the hydraulic conductivity. Aspects such as the heterogeneity of the soil in the aquifer and anisotropy also affect the hydraulic conductivity. So a smaller grain size does not rule out a lower bulk hydraulic conductivity of the aquifer.

The goal of this chapter is to combine the findings of the previous chapters and to determine the total effect on the project costs. The results of the previous chapters were used to determine different scenarios to estimate the project cost reduction. These were linked to the costs of additional research to verify the scenario. The costs for soil investigation are based on estimations from similar projects provided by Fugro. The cost calculations for dike monitoring and additional soil investigation are shown in Appendix E.

One aspect that is not treated here is the possibility that monitoring and soil investigation uncover that a section of the dike trajectory are more vulnerable than previously thought. If there is a weak spot in the dike this will lead to an increase in the expected risk of dike failure and the expected project cost. The risk caused by the presence of a section with weaker parameters for piping is treated in Chapter 5.1. This chapter analyses the expected project cost reduction for identifying section more resistant to piping than previously thought due to additional soil investigation and monitoring.

The investment costs for soil investigation and monitoring were compared with the expected benefit. The costs and benefit were plotted as percentages of the project costs so they can be analysed independent of the size of the project. The results are shown in Figure 46. The percentage for investment costs is the costs for monitoring and soil investigation divided by estimated project costs of the base scenario. The investment costs are included in the expected project costs to determine the expected project cost reduction.

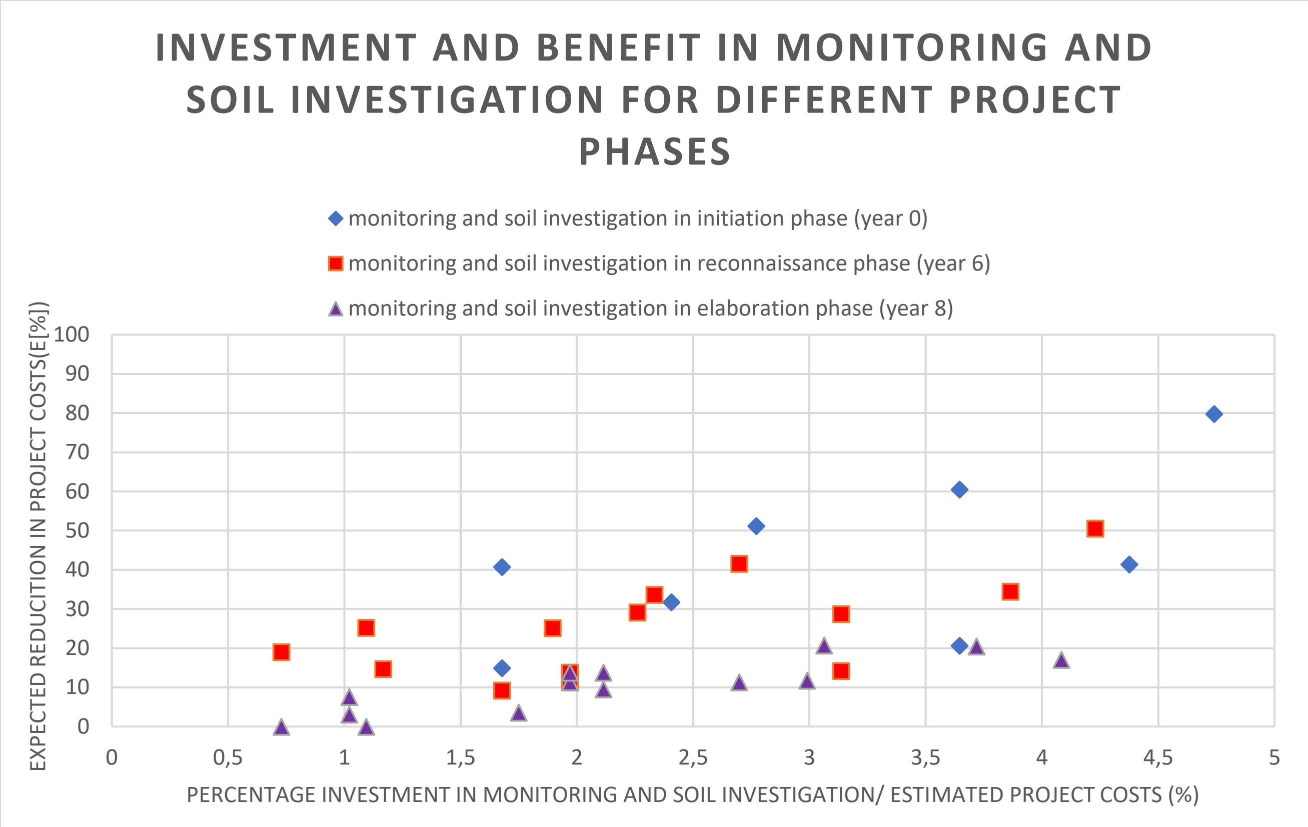


Figure 46 expected reduction in project costs for different scenarios

Each dot in the figures represents a possible outcome of the monitoring, soil investigation or both for a specific scenario for the soil composition. The result of the dot depends on the applied scenario. For example, in Figure 47 the corresponding scenario for the results is shown for the dot. The dot was a possible outcome for implementing piezometers to monitor the hydraulic head in the aquifer to estimate the seepage length and soil investigation performed to determine the d_{70} at an interval of 100m. So the investment costs are equal to the cost of the design, implementation and maintenance of the monitoring system and performing the soil investigation, while the reduction in project costs comes from the design and construction costs saved due to incorporating the effective foreshore in the design. The calculations for each individual scenario used can be found in appendix E.

The corresponding expected project costs and investment costs for Figure 47 are shown in Table 17. First the expected project cost is determined for each section of the dike trajectory given the known values for the parameters. For section 1 the cheapest dike reinforcement given the soil parameters is a 15m long sheet pile wall, while for section 2 it is a 20m berm. Note that the costs for Table 17 are the expected costs, so the probability of cost reduction is included. This is further elaborated in Appendix E. Then the costs for each section is summed up, including the investment costs for monitoring and soil investigation. Finally the total expected project costs are divided by the estimated projects costs resulting from the base scenario to determine the expected project cost reduction. For a more detailed description of the calculations, see Appendix E. The total expected project cost reduction calculated represents a single value or dot in Figure 46 (see Figure 47).

Monitoring and soil investigation methods	Expected project cost section 1 (E[M€])	Expected project cost section 2 (E[M€])	Expected project cost dike trajectory (E[M€])	Investment costs monitoring and soil investigation (M€)	Total expected project cost (E[M€])	Total expected project cost reduction (E[%])
Monitoring seepage length + soil investigation d_{70} probe interval 100m (J)						
Performed/started in year 0	5,4	5,0	10,4	0,5	10,9	20,6

Table 17 expected project cost reduction for monitoring and soil investigation

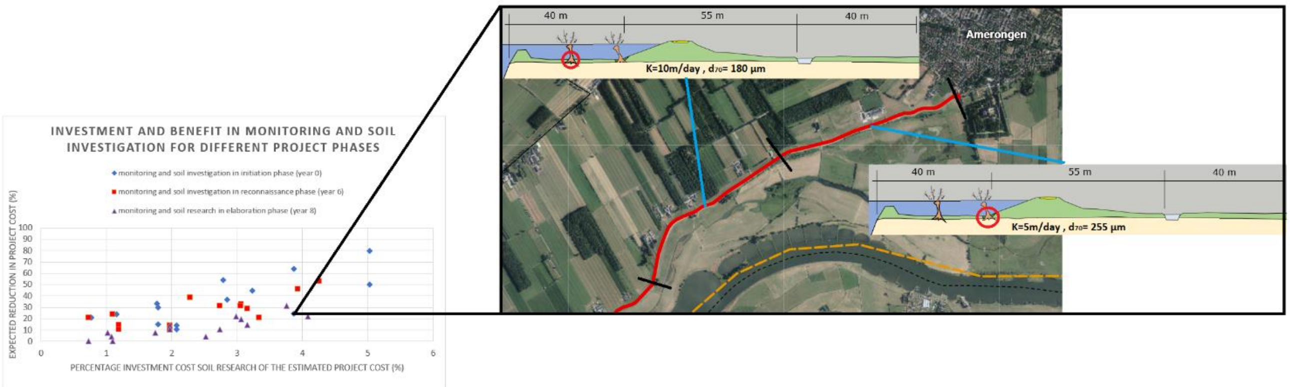


Figure 47: example of what one point in the graph represents

The additional soil investigation and monitoring are performed or started in different project phases. This is to show how the benefit in taking these measurements for the project becomes generally smaller when the measurements are taken later in the project. For additional soil investigation and hydraulic conductivity determination methods this effect is because the data collected can be used in less project phases if the measurements are performed later in the project. In the case of monitoring it is also because the acquired dataset is shorter.

As is shown in Figure 40, the possible results of monitoring and soil investigation are based on general estimates of the characteristics of the dike trajectory, as few data is available. For example, the possible result for the d_{70} is based on the mean value of the range of the found lithology class in the soil profile and the length of the effective foreshore is based on the distance between willow trees to the toe of the dike in the foreshore. The exact result for investment into soil research and monitoring and the reduction in project costs requires actual measurements at the dike trajectory. The results from this thesis are only based on a theoretical case, and require verification by an applied case.

Figure 46 can be used to estimate the cost-effectiveness of investing in additional soil investigation and monitoring. The estimated research costs and the estimated reduction in project costs if the scenario is accurate have already been determined. What still needs to be included is the probability of the scenarios occurring. To estimate the probability of the described scenarios for the case dike trajectory to occur, the available soil data was used to calculate the probability of the value occurring in the dike trajectory. The mean and standard deviation were calculated, and assuming a normal distribution the probability of certain values for the parameters being exceeded was calculated. If multiple parameters were variable for the scenario, the relation between the variable parameters was assumed to be independent. The used data and the calculations for the estimation of the probability of occurrence of the scenarios can be found in Appendix F. The estimation of the probability of occurrence of the scenarios is shown in Figure 48. How to use the results from Figure 48 is discussed in chapter 4.7.

It should be noted that the estimation of the probability of occurrence of the scenarios is not included in the calculation of the expected reduction in project costs in Figure 48. This is because the estimation of the probability of occurrence of the scenarios (Appendix F) is a rough estimation, which is only intended to provide a quantification of the probabilities of the scenarios and to determine which scenario to use in Chapter 6.1.1. Therefore they have been excluded from the calculation of the expected reduction in project costs.

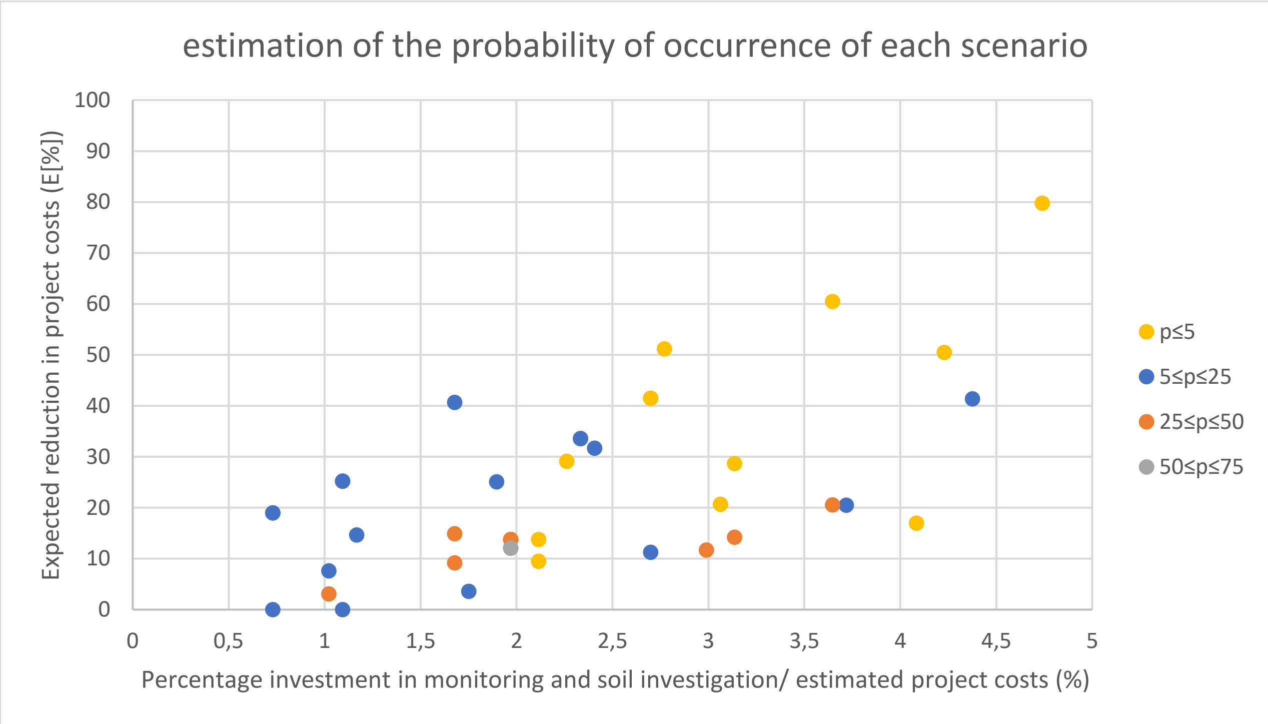


Figure 48 quantification of the probability of occurrence of the scenarios

4.7 discussion reduction project costs

The soil profile for the dike trajectory at Amerongen will most likely not be as such that it can be schematised in neat equidistant sections. The trajectory will more likely look like the trajectory shown in Figure 37. But to come to such a representation of the dike trajectory to be used for the design, the available information needs to be interpreted. This process is shown in Figure 49.

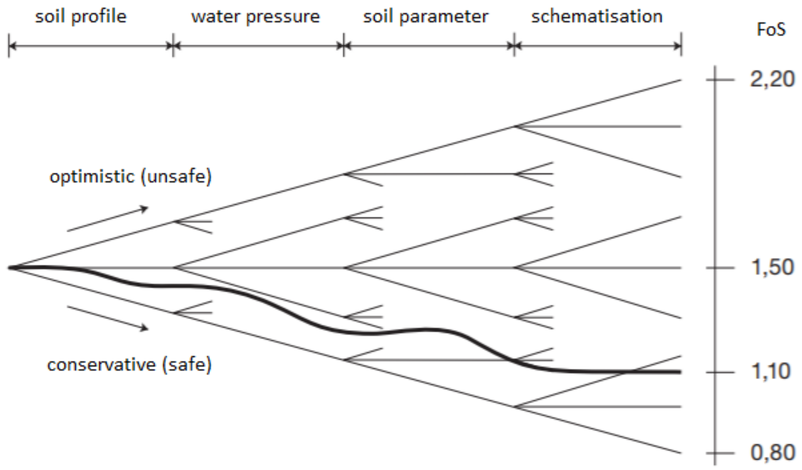


Figure 49: schematisation steps stability analysis dikes (adapted from ENW, 2019)

ENW has published a technical report which describes a risk management strategy to schematise the soil profile for the stability of the slope, which can be applied to piping (ENW, 2009). For the determination of the composition of the soil and the dike profile several aspects have to be taken into account, namely:

- The choice of the soil profile needs to be representative for the entire dike trajectory. In other terms, the most conservative soil profile in regards to the considered failure mechanism should be chosen to represent the dike trajectory
- Depending on the nature and quantity of the available information on the soil profile, uncertainties about the soil profile will appear. These can be divided into two general types of uncertainties.
    1. Uncertainty on the position of the transition between different layers. The positions of the transition is usually interpolated between available soil profiles.
    2. The presence of local weak spots of limited size, which have not been found in the soil investigation.

The risk management strategy to determine the soil profile consists of 6 steps. These are summarized below:

1. Start with a rough estimation of the composition of the soil which the most plausible scenario is given the available information.
2. Add a preliminary schematisation of the water pressures and estimate the soil parameters.
3. Determine which uncertainties present in the soil profile are potentially influential. The uncertainties can for example be the position of the transition between two different layers or the presence of weak spots. These depend on the available information and the geology of the area.
4. Use the determined uncertainties from step 3 to create different scenarios of the soil profile which are consistent with the data already available, but take into the account of the local

geology such as the possible presence of a sand lens. Determine the factor of safety for each scenario. These scenarios can be sorted as optimistic and pessimistic scenarios.

5. Once the factors of safety for the scenarios are determined, the probability of occurrence for the scenarios is analysed. Preferably this is done by quantitative analysis, but this is hardly possible with current knowledge. This means the probability of occurrence of the scenarios can most likely only be estimated qualitatively.
6. Scenarios for which the probability of occurrence is very low (almost impossible) can be rejected. Of all the other conservative scenarios which cannot be eliminated it should be determined what information is needed to determine that the scenario can most likely be eliminated. This is where additional soil investigation and monitoring is possibly profitable.

The result is a number of scenarios arranged from optimistic to conservative, for which quantitatively or qualitatively has been determined what their probability of occurrence is. For the conservative scenarios it should have been determined what information is necessary to reject the scenario. The final dike design is based on the most conservative scenario that remains. Additional research can provide the data needed to reject a conservative scenario, which reduces the required project costs. Whether it is economically beneficial to do so, depends on three factors (EMW, 2009).

I. The costs of the additional research
II. The probability of the additional research having the desired effect of excluding the conservative scenario
III. The reduction in project cost for when the conservative scenario can actually be rejected.

This strategy is applied to the results found in chapter 4.6. The results in Figure 46 show all different scenarios listed for the dike trajectory. If the measurements already available are assumed to be accurate. So already the scenarios for which the parameters in the representative cross-section for the entire trajectory are more resistant to piping were rejected. This leaves the scenarios for which different schematisations of the dike trajectory are possible. To deal with the remaining scenarios, a new strategy is proposed.

The new strategy is a possible method to estimate how much of the project budget should be spent on gathering additional information on the soil profile. The idea of the strategy is shown in Figure 50. The starting point is the same as the risk management proposed by the EMW. This is the most conservative scenario found for the dike trajectory, which in this case is the base scenario. But for this soil research strategy, you want to eliminate the optimistic scenarios. This is because if there is a possibility that additional research can lead to a more optimistic schematisation of the soil profile, the current design is potentially unnecessarily large and expensive.

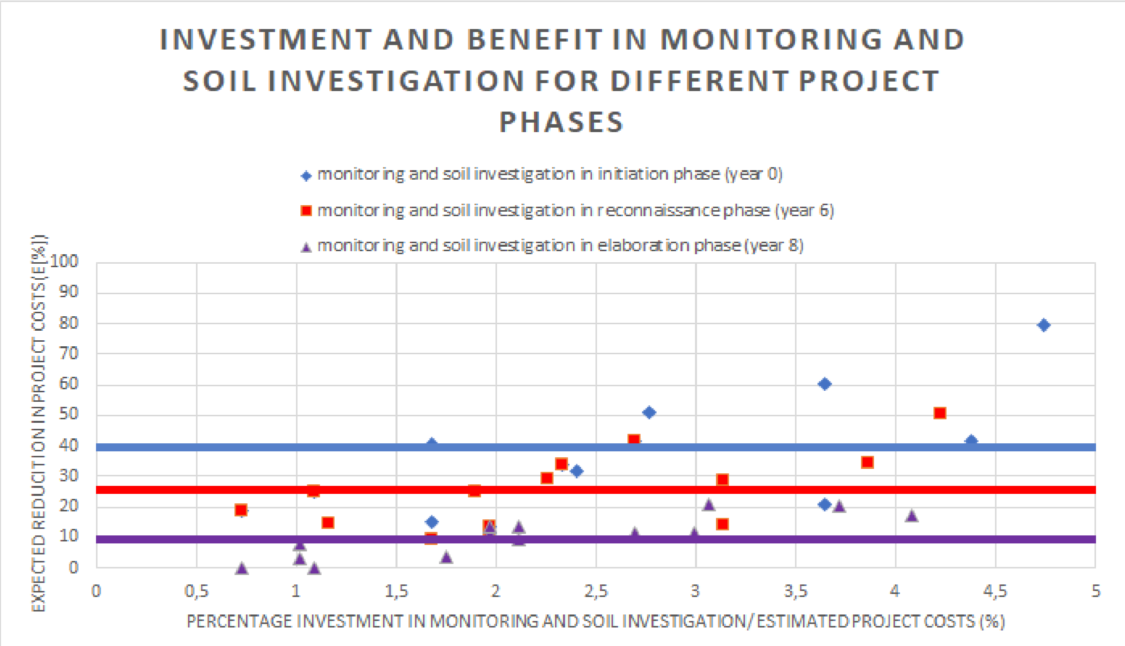


Figure 50: strategy of determining budget for additional soil research

The criteria for each project phase are based on the accuracy margins of the project cost estimates for hydraulic structures (van der Meer, 2016). These margins are +or- 40% for the reconnaissance phase, 25% for the elaboration phase and 10% for the realisation phase. The idea is to apply these margins as upper limit to the possible project cost reduction for implementing monitoring or other additional soil research. For example, before the reconnaissance phase, the scenarios with the potential to reduce the project cost with more than 40% need to be eliminated. This still gives some flexibility as not all information on the project will be available, but prevents a larger project scope than needed if the project cost can be reduced by more than half the cost.

If the expected benefit is high enough that the optimistic scenario could occur, then additional soil research is needed to verify whether less dike reinforcement is needed than was first thought. The corresponding ratio of research cost to the total project cost can give an indication of how large the budget for the soil research should be. The probability of a scenario has to be taken into account though, as when the probability of a scenario occurring is very low the expected benefit will be very low.

Implementation during different project phases is incorporated in the strategy. The different project phases can be identified by the colours of the dots in Figure 50. When the monitoring needs to start and soil investigation needs to be performed has to be taken into account when determining your measurement plan, as if the measures are implemented too late they might not be on time for some project phases or in case of monitoring not enough relevant data could be collected. These aspects play a role in deciding whether enough information has been collected for the current project phase.

To illustrate, the strategy is applied in Figure 51 to the scenarios for the results of monitoring and soil investigation for the initiation phase. For each scenario the expected project cost reduction for the outcome, the required investment and the estimated probability of the scenario occurring is shown. Scenarios for which the probability of occurring is too low can be dismissed. The blue line indicates the cost margin the design should be in at the reconnaissance phase.

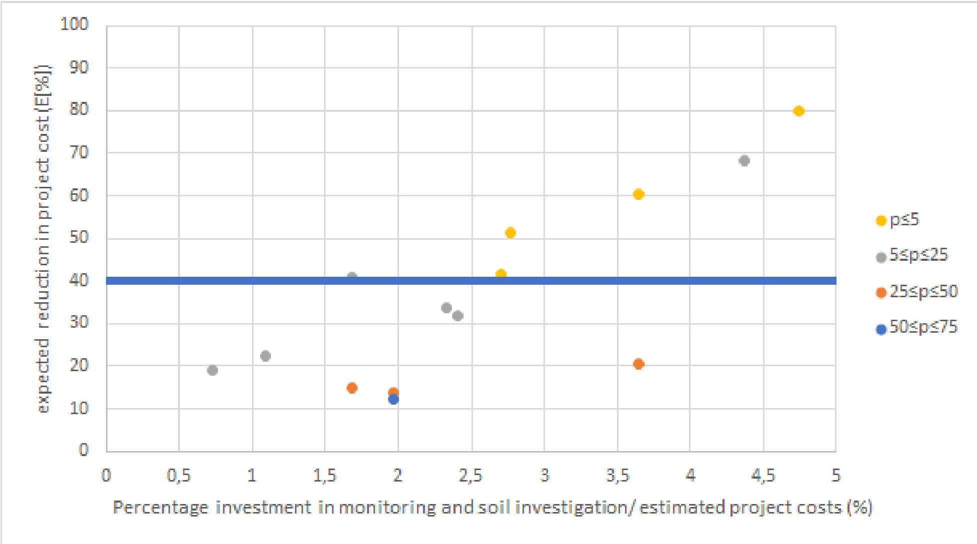


Figure 51 applied strategy for initiation phase

The goal of the developed strategy is to provide a guideline to whether enough information is available to design a cost-effective dike reinforcement. It is unpractical to perform all the measurements in the initiation phase when not much is known on the project yet. But there needs to be enough data collected that the defined project scope is at least in the range of what is required. In each phase the upper limits function as criterion of in what detail the characteristics of the soil profile should have been measured. If the designer can still imagine likely scenarios based on the available data for which the expected project cost reduction is higher than the upper limit of the project phase, additional soil investigation and/or monitoring has to be implemented.

The cost-effectiveness can be estimated by combining the investment costs and projects benefits determined for the scenario with the likelihood of the scenario estimated by expert judgement. An estimation of the probability of occurrence for the scenarios in this thesis is shown in Figure 48. Using the probability of the scenarios in combination with the expected benefit if the scenario occurs the required investment in monitoring and soil investigation and the expected benefit of performing these measurements can be estimated.

In summary

The expected reduction in project costs depends on the project phase in which the monitoring is started and/or additional soil investigation has been performed. When implemented early, the data can be used in more project decisions such as the choice of preferred alternative. For monitoring, the measurement period is also an important factor as the measurement period determines the likelihood of measuring relevant data. Also the local conditions are important to determine the benefit of monitoring and soil investigation. In combination of the estimated probability of stronger soil parameters occurring in the dike trajectory than first anticipated, the estimated costs and the expected benefits can be determined.

5. Monitoring and soil investigation for duty of care

While monitoring and additional soil investigation can reduce the expected project costs for dike reinforcement, early monitoring and soil investigation is also beneficial for the duty of care for the dike. Duty of care means the legal responsibility of the regional water authority of the dike to make sure the primary flood defences are kept safe and up to standard (Dijkmonitoring, 2019). The problem is that as the dike has failed the safety assessment, the dike was deemed unsafe. The risk of dike failure at this location is higher than the safety standards demand, and will remain so until the dike is reinforced. This will takes years before being finished. The risk during the project will be analysed for the case dike trajectory at Amerongen

There are two aspects to the effect of early monitoring and soil investigation on the taken risk and fulfilling the duty of care. There is the danger of a higher risk than first anticipated, and the potential risk reduction by performing monitoring and soil investigation.

The danger of risk being higher than first anticipated is caused by inherent uncertainty in the data or uncertainty in the schematisation of reality. If the probability of failure in the dike trajectory is higher than first anticipated, the risk during the design period becomes larger. Early measurements in the local situation might identify the weaker parameter, providing a more precise risk analysis. This can be used to determine if emergency measures or other methods are necessary to guarantee the safety level specified for the duty of care. In this case monitoring and soil investigation only serves as a means to make a better decision.

There is also the potential risk reduction due to monitoring. Risk is defined as the probability of failure multiplied by the damages failure can cause. This means if monitoring and soil investigation can be used to identify stronger parameters in the dike trajectory, the failure probability of the dike the risk will decrease. For this analysis the original risk calculated using the result of the safety assessment described in chapter 3. is compared with the risk determined for the stronger parameters relative to piping.

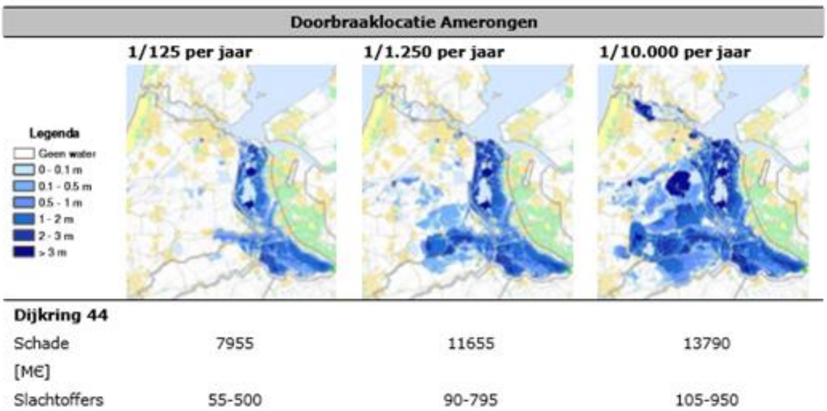


Figure 52: Damage in case of dike failure at case location (Bisschop, 2011)

5.1 Risk due to unknown weak spots

Until now, we have only covered the possibility that monitoring and soil investigation lead to stronger parameters in the dike trajectory relative to piping. But there is also the chance that investigation into the geological and hydrological aspects of the dike trajectory uncovers that the dike is more vulnerable than first anticipated. Especially the grain size distribution can have a large spatial variability, and previous research has suggested that an even more conservative value than the characteristic value is more applicable for dike design (Kanning, 2014).

The difficulty in expressing the benefit of identifying the risk in the project, is that a risk reduction cannot be expressed as reduced project costs. Finding worse parameters than expected actually increases the estimated project costs, since now a larger berm, sheet pile wall or other measure is needed to reinforce the dike to meet the safety standards. However, ignoring the weaker section leads to unacceptable risk of dike failure. The benefit is therefore expressed in risk reduction.

For the weak spot we assume the presence of a sand lens containing weak material related to piping, which is connected to the hinterland behind the dike. To determine the risk due to unknown weak spots, the annual probability of dike failure given the weak parameters is combined with the probability the layer is unnoticed in the soil investigation. The width of the sand lens is normally distributed with a mean of 75m and a standard deviation of 15m. The grain size distribution and the horizontal hydraulic conductivity of the sand lens were based on boring samples and HPT results from Dinoloket and Fugro. In the HPT boring shown in Appendix H a sand layer in the aquifer with a higher hydraulic conductivity of approximately 20m/day is shown. For this chapter it is assumed this layer is connected at the foot of the dike on the riverside to the hinterland. The lowest found d_{70} from the available soil borings is still 180 µm. The probability of missing the weak layer depending on the probe distance is shown in Figure 53.

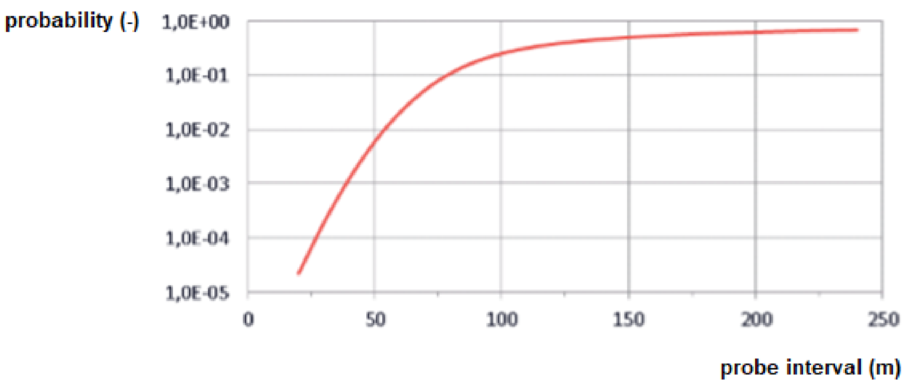


Figure 53: chance of missing layer of weak soil material as function of the probe distance for equidistant probes (ENW, 2012)

With the failure probability of the dike trajectory calculated, the risk can be calculated for different intervals between the probes (Table 18). A common value for the distant between the probes for primary flood defences is 100m (ENW, 2012). The amount of damage was determined based on the design water level at the dike trajectory from Figure 52. The costs for soil investigation was based on experience in Fugro on comparable projects given the probe interval (See Appendix C). The resulting expected annual risk for different probe intervals is shown in Figure 54. The used intervals are 100m, 50m and 25m between probes. For this thesis the probability of a sand lens occurring in a 1km dike trajectory is assumed to be 0,1.

	Probe interval= 100m	Probe interval= 50m	Probe interval= 25m
Annual failure probability due to sand lens with K=20m/day (-)	5,7 $*10^{-3}$	5,7 $*10^{-3}$	5,7 $*10^{-3}$
Probability of missing soil layer vulnerable to piping given probe interval (-)	10^{-1}	$9{,}0*10^{-3}$	$8*10^{-5}$
Damage dike failure for design water level (M€)	$11{,}7*10^{3}$	$11{,}7*10^{3}$	$11{,}7*10^{3}$
Expected annual risk due to sand lens (E[M€]/year)	0,66	$5{,}3*10^{-2}$	$5{,}3*10^{-4}$
Estimated costs soil investigation given probe interval (M€)	0,3	0,5	1,1

Table 18 expected annual risk due to sand lens for different probe intervals

The expected annual risk in Figure 54 was put on a logarithmic scale. The three points in Figure 54 correspond to a probe interval of 100m, 50m and 25m, for which the smaller the probe interval is the larger the estimated costs are. The estimated costs for the soil investigations depend on how many borings are performed. The cost calculations are included in Appendix C.

What can be seen is that the annual risk for a probing interval of 100m and 50m exceeds the acceptable risk. The chance of missing the sand lens is still considerable, and the high amount of damage dike failure at this location will cause makes the excepted annual risk very high for the regular probe interval.

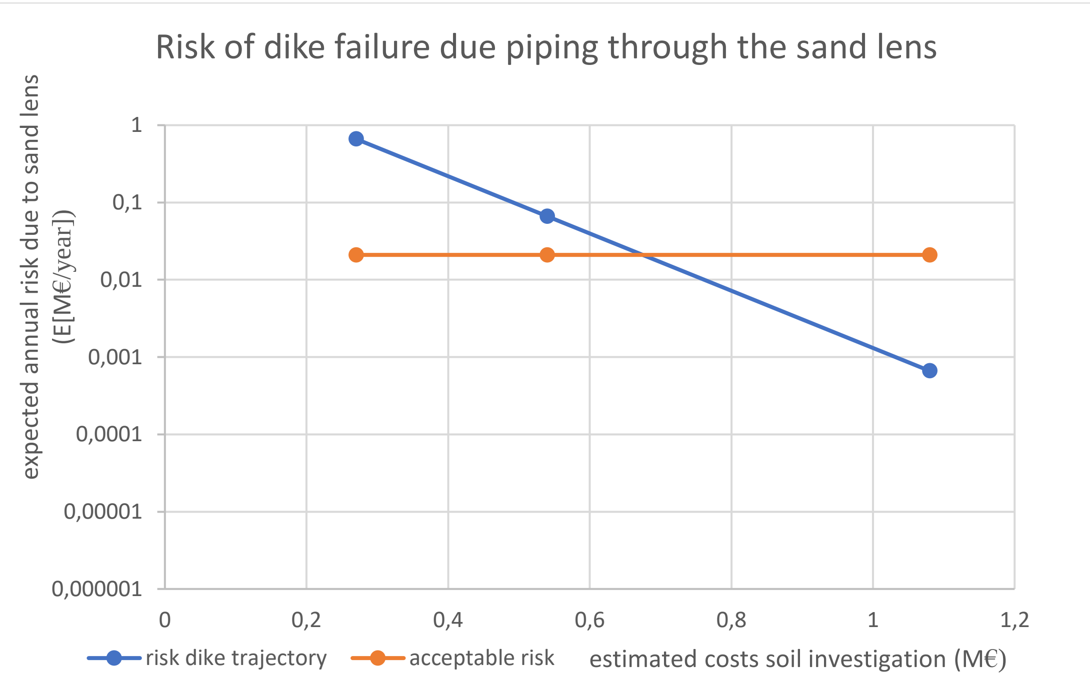


Figure 54 expected annual risk for different probe intervals

For a probe interval of 25m the expected annual risk is significantly reduced by a factor 1000 compared to the common probe interval of 100m, and is lower than the expected annual risk using the failure probability determined as the safety standard of the dike trajectory. For the used case it can beneficial to apply a smaller probe interval, given that costs of the soil investigation is equal to the expected annual risk for one year.

In chapter 4.7 a risk management strategy for the schematisation is described. It is described that whether the scenario must be used as the representative cross-section depends on the probability of occurrence of the scenario. If the probability is very low the scenario can be dismissed, while the conservative scenario which cannot be eliminated must be used as the representative scenario.

Since the probability of occurrence of the described sand lens is an assumed value, no conclusion can be drawn whether the scenario of the sand lens should be used as representative cross-section. But for the used probability of occurrence the gained reduction in expected annual risk would warrant a probe interval of 25m, to reduce the expected annual risk to below the risk set by the safety standard.

5.2 Possible risk reduction due monitoring and soil investigation

It may seem counter-intuitive, but it is possible to reduce the perceived flood risk with monitoring or soil investigation only. Risk is the combination of the probability of a flood with the consequences of the flood (Jonkman, 2018). For this analysis the consequences or damages of flooding are expressed in economic damage, but is also possible to assess risk based on individual risk and societal risk. The representative cross-section for the dike trajectory used for the safety assessment is usually a conservative representation of the dike trajectory, which leads to a high probability of dike failure. This is safe in terms of design, but may lead to an unnecessarily large risk expectancy. For example, the effect of the foreshore is not incorporated in the base scenario from the safety assessment. If the data from monitoring proves there is a longer seepage length than first anticipated, the expected risk is reduced.

This chapter compares the expected annual risk for different scenarios for the dike trajectory. The scenarios are based on possible scenarios for the dike trajectory described in the case description. The consequences were estimated in the VNK 2 project for Dike Ring 44 Kromme Rijn (Bisschop, 2011).

The expected annual risk was determined by multiplying the probability of failure for the dike trajectory with the damage in euros caused by flooding determined in VNK2. For the probability of failure the results from the FORM-analysis performed in Prob2B. For the damage cause by flooding the economic consequences for flooding caused by the design water level corresponding to a 1/1250 return period was used (Figure 52). The analysis is performed to give a general estimate of the expected annual risk. The actual expected annual risk is equal to the integral of all the failure probabilities from occurring water levels in the river multiplied with the consequences of flooding given the corresponding water level. This failure probability was calculated using Prob-2B.

5.2.1 Expected annual risk reduction different hydraulic conductivity

Three values need to be compared to assess the risk the dike trajectory poses during the project period. These are:

1. The expected annual risk that corresponds to the safety requirements set for the dike trajectory.
2. The expected annual risk for the base scenario specified in chapter 3.
3. The new failure probability if it is discovered the subsoil is more resistant to piping than expected.

First the risk reduction due to a lower hydraulic conductivity in the aquifer is analysed, for which the results are shown in Table 19.

	K=5m/day (scenario 6-D-I)	**K=2,5m/day (scenario 7-D-I)**
Annual failure probability due piping for K=…m/day (-)	$1{,}3*10^{-4}$	$2{,}1*10^{-6}$
Damage dike failure for design water level (M€)	$11{,}7*10^{3}$	$11{,}7*10^{3}$
Expected annual risk for K=…m/day (E[M€]/year)	1,5	$2{,}4*10^{-2}$
Expected annual risk at the start of the project (E[M€]/year)	47,2	47,2
Expected annual risk for piping set by safety standards (E[M€]/year)	$2{,}1*10^{-2}$	$2{,}1*10^{-2}$

Table 19 expected annual risk for different hydraulic conductivities

What is striking is the comparison between the expected annual risk corresponding to the set safety standards and the calculated expected annual risk at the start of the project. The expected annual risk during the project is approximately 47 million euro per year. The difference between these two is around a factor of 100. However, this expected annual risk is in the same order of magnitude to the findings of the results of VNK2 of the dike ring 44 (Bisschop, 2011). Their risk assessment found an expected annual risk of 31 million euro per year for the dike ring, of which around 70% of the economic risk due dike failure for the dike ring was resultant from the same dike. But since the VNK2 analysis uses the corresponding damage level to the failure rate of the corresponding water level and the analysis for this thesis uses the consequences for a water level with a 1/1250 return period for the total failure probability the resulting expected risk in this analysis is higher.

The used scenarios in Table 19 reduce the expected annual risk compared to the risk calculated in the base scenario significantly. Due to the large uncertainty in the hydraulic conductivity calculated using the correlation between the grain size distribution of the aquifer and the hydraulic conductivity, it is possible a different value for K is found for the entire dike trajectory. Using a correlation method the hydraulic conductivity can vary with a factor of a 10 depending on which equation is used (Berbee, 2017). It can therefore be beneficial to also use filed methods such as HPT boring tests or to verify the results using different sources such as REGIS or head difference-response analysis.

What's more, the used analytical one layer approach in Sellmeijer's equation can lead to underestimations of the piping resistance of the aquifer (Stoop, 2018). For a heterogeneous and isotropic aquifer the aquifer composition can resist a water level up to 45% higher than when the aquifer is assumed to be homogeneous. So incorporating these characteristics of the aquifer in the safety assessment of the dike trajectory for piping could greatly reduce the calculated risk.

5.2.2 Expected annual risk reduction different d_{70}

The same process in the previous section was performed for the d_{70}. The results for the characteristics of the grain size are shown in Table 20. The used values for d_{70} were based on the derivations based on the lithology class of the available boring samples near the dike trajectory.
The expected annual risk is higher for the two different grain size distributions than for the hydraulic conductivity of the aquifer and the seepage length. This is caused by the higher failure probability for

stronger soil parameters against piping for the d_{70} than compared with the other two parameters. But if measurements of the d_{70} point to a mean value of 360µm for the d_{70} in of the upper layer in the aquifer, the expected annual damage is reduced to near the value for expected annual damage set by the safety standards.

The problem with trying to reduce risk by more accurately assessing the subsoil parameters over the dike trajectory is the spatial variability of these parameters. The coefficient of variation for the d_{70} can remain large even if the grain size distribution is measured at close interval (Visser, 2015). This means that even if there are sections of dike which have a stronger resistance to piping, it is likely there will be weak spots over the length of the dike. Therefore the measurements will not necessarily reduce the failure probability and therefore the risk for dike failure due to piping of the dike trajectory. But it can serve as a better indication which dike sections need to be reinforced, which saves costs.

	d_{70}=255µm (scenario 4-B-I)	**d_{70}=360µm (scenario 5-B-I)**
Annual failure probability due piping for d_{70}=255µm (-)	$5,8*10^{-4}$	$6,3*10^{-5}$
Damage dike failure for design water level (M€)	$11,7*10^{3}$	$11,7*10^{3}$
Expected annual risk for d_{70}=….µm (E[M€]/year)	6,7	0,7
Expected annual risk at the start of the project (E[M€]/year)	47,2	47,2
Expected annual risk for piping set by safety standards (E[M€]/year)	$2,1*10^{-2}$	$2,1*10^{-2}$

Table 20 expected annual risk for different grain size distributions

5.2.3 Expected annual risk reduction different seepage length

Finally the possible risk reduction due to incorporating the effective foreshore in the design is analysed, for which the results can be found in Table 21. For this case the increase in seepage length has the largest reduction in expected annual risk when the longer seepage length is incorporated in the safety assessment.

The effective foreshore has the benefit of it being overlooked in the base scenario. The foreshore has to meet certain standards before it can be incorporated in the design. The effect of the foreshore can be measured due to monitoring, for which long measurement series are needed to have a proper data set for the dike assessment. When these conditions are not met, the foreshore is not incorporated in the design. Measuring the effect of the foreshore early is therefore a suitable method to reduce the expected annual risk for the dike trajectory.

	L=75m (scenario 8-G-I)	**L=95m (scenario 9-G-I)**
Annual failure probability due piping for L=….m (-)	$7,3*10^{-5}$	$1,6*10^{-6}$
Damage dike failure for design water level (M€)	$11,7*10^{3}$	$11,7*10^{3}$
Expected annual risk for L=….m (E[M€]/year)	0,9	$1,9*10^{-2}$

Expected annual risk at the start of the project (E[M€]/year)	47,2	47,2
Expected annual risk for piping set by safety standards (E[M€]/year)	$2{,}1*10^{-2}$	$2{,}1*10^{-2}$

Table 21 expected annual risk for different seepage lengths

However, the fact that monitoring of the hydraulic head in the aquifer is needed to assess the effect of the foreshore on the seepage length also poses problems in using monitoring to reduce the risk. Since longer measurement periods are needed for the monitoring to be effective (Tonneijck, 2018), a longer measurement period is needed to collect relevant data. Either the monitoring has to start well before the initiation of the dike project, or the results for monitoring will be available when the project is already well under way. For the latter the question is then whether the results will still be of much use in the safety assessment as the project might not go on for much longer.

5.2.4. Risk reduction due to monitoring

As is mentioned in chapter 4.2.3, the length of the measuring period for monitoring affects the expected result. For a longer measuring period a higher water level event is expected to occur than for a smaller measuring period. The data on the hydraulic head in the aquifer can then be used to determine the fictional entry point in the foreshore.

The effect of monitoring on the calculated annual risk for dike failure due to piping is shown in Figure 55. Scenario 8-G-I was used, for which a seepage length of 75m is present along the entire dike trajectory. The figure shows how the calculated risk of dike failure due to piping is reduced drastically over time if the seepage length is longer than was expected at the start of the project. The curve levels off at the end, and the lower bound for the calculated risk with monitoring the hydraulic head is equal to the result in Table 21, which is an expected annual risk of 0,9 M€ per year.

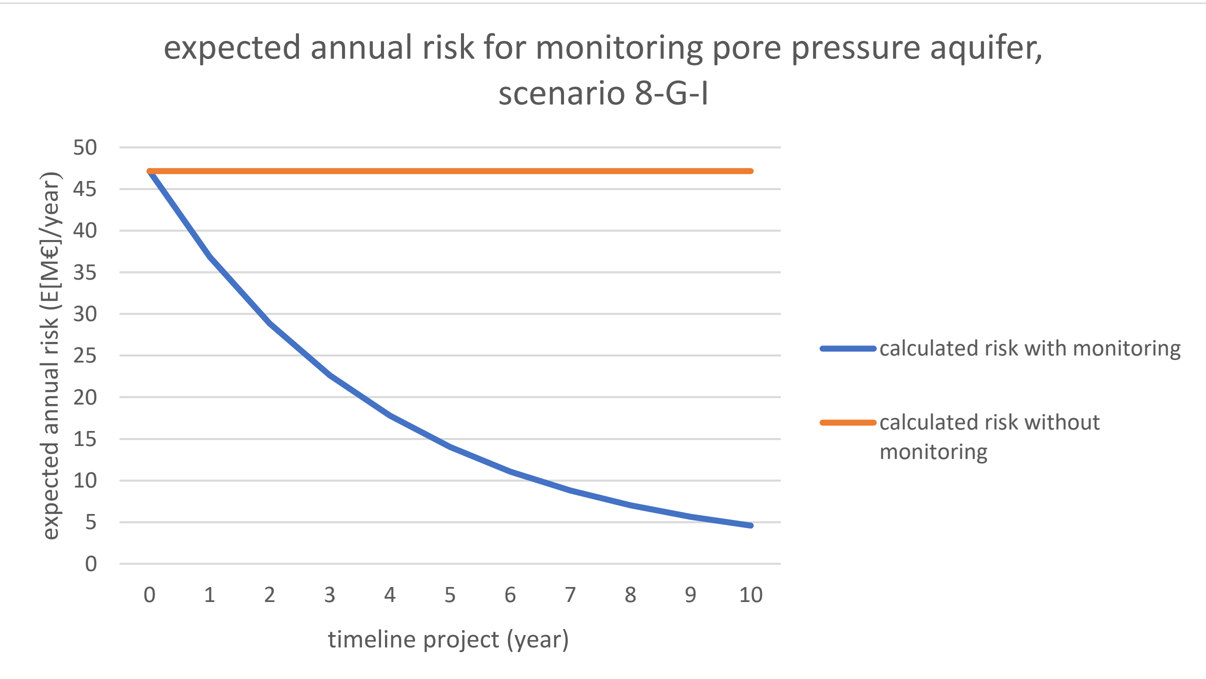


Figure 55 calculated risk due for dike failure due to piping.

One problem for reducing the calculated risk using monitoring is that the higher strength measured needs to be present over the entire length of the trajectory, or there will still be a high expected annual risk due to dike failure. If there is a weak section in the dike trajectory for which the foreshore cannot be included in the design, there is still the probability of dike failure due to piping for the dike trajectory for that section and so a high expected annual risk. It does however provide insight into which sections of the dike are most likely to fail, which can be closely monitored or emergency measures can be prepared for the relevant section.

5.3 Discussion duty of care

The risk assessment performed in chapter 5 is a simplified version of the analysis usually performed for entire dike rings. But the results found in the risk analysis for the dike trajectory are in the same order of magnitude to the results of the risk analysis by VNK2 (Bisschop, 2011). Both determined the current expected annual risk to be between 30-50 M€. There are a few aspects of the used case which have to be mentioned to explain the obtained results.

First of all, there is the issue of the damage caused by the flooding at the case location being exceptionally high. A dike failure near Amerongen floods half the province of Utrecht and parts of Amsterdam. For the Netherlands it is likely only the flooding of dike ring 44 will cause more damage. For this research it meant that any change in the probability of failure of the dike trajectory increased the expected annual risk significantly. This effect gives a good example of the importance of assessing the risk of a dike trajectory and if the risk becomes too large to take action to reduce the taken risk. It also means the results of the risk assessment performed here can’t be directly compared to other dikes for which the damage of failure are lower. The effect of the large consequences will have to be taken into account.

What’s more, this safety assessment only takes the economic damages into account. The group and individual risk also need to be included in the risk assessment. The individual risk determines the risk a person has of drowning for which a maximum of the acceptable risk is set. The group risk considers the amount of casualties a flood event will cause, for which the acceptable amount must be determined. These type of risk were not included in this thesis. But if additional monitoring and soil investigation decreases the probability of failure, it will decrease the individual and group risk as well.

For the reduction in risk only performing the measurements was included. The measurements change the perception of the risk and provide better insight in the risk being taken, but it does not change the actual risk due to dike failure. Measurements do not necessarily decrease the failure probability, but give a better insight into which sections of the dike are vulnerable (Kanning, 2014).

That is why monitoring and extra soil investigation measures are useful for risk analysis, but the option of implementing emergency measures is necessary. Monitoring might provide a warning signal of impending dike failure, but if no emergency measures are in place then there is nothing that can be done to prevent dike failure. And if the expected annual risk is unacceptably high during the project, it will be beneficial to take action to reduce the risk.

The main issue with the duty of care of the dike trajectory for this part of its life cycle is that if it were found the risk is larger than previously thought and what is acceptable, dike reinforcement is already being prepared. If the project is hastened due to the immediate risk, the construction works in and around the dike will weaken the dike even further. Even if the construction works do not lower the failure probability of the dike, the risk analysis has already shown that this is still a hefty risk. In this case monitoring and additional soil investigation are applicable to potentially reduce this perceived risk, and to correctly assess the risk and what the next steps should be. If the risk is still unacceptable, the best solution during the project might thus be to prepare emergency measures to counter the wells if piping begins to occur. These measures can possibly prevent dike failure from occurring, and possibly counteract the vulnerability of the dike during the realisation phase.

In summary

Summarised, in the previous chapter the focus for monitoring and soil investigation was on identifying the strong sections in the dike trajectory and reducing the expected project costs. But the benefit of monitoring and soil investigation is not only finding the stronger sections in the dike trajectory, but also in the identification of the vulnerable sections, as then the risk of dike failure can be acted upon if needed. In this chapter, two aspects of the impact of monitoring and soil investigation on the duty of care of the regional water authority of the dike trajectory were analysed. These are the risk of dike failure due to unknown weak spots in the subsoil under the dike trajectory and the effect of monitoring and soil investigation on the calculated risk.

To determine the risk of dike failure due to a weak spot the risk of dike failure due to piping caused by the presence sand lens with weak characteristics was calculated for different probe intervals. Here it was shown that the commonly used probe interval of 100m for soil investigation can still result in a considerably high expected risk of dike failure due to piping. If there is a probability of such a weak spot as described in the chapter, a smaller probe interval is recommended.

It is also possible to lower the calculated risk of dike failure due to piping with monitoring and soil investigation if stronger soil parameters are found than were assumed in the safety assessment of the dike. This provides a more accurate indication of the expected annual risk for the dike trajectory, especially if the soil parameters in the safety assessment were based on conservative estimates.

6. Discussion, recommendations and limitations

6.1 Discussion

6.1.1 Estimated costs and benefits soil investigation/monitoring

Part of the goal of this thesis is to determine an estimate of the ratio between the investment costs of different soil research methods and the expected benefits from the additional research. In this chapter the amount of investment costs for monitoring and soil investigation will be estimated and compared to the expected benefit. The scenario used for the estimated budget for additional soil investigation and monitoring is the scenario with the highest expected project cost reduction including the estimated probability of occurrence for the scenarios. The calculations for the expected project costs including the estimated probability of the scenarios can be found in Appendix F. The scenario with the highest expected project cost reduction is scenario 10-K-I. The soil characteristics in the dike trajectory are shown in Figure 56.

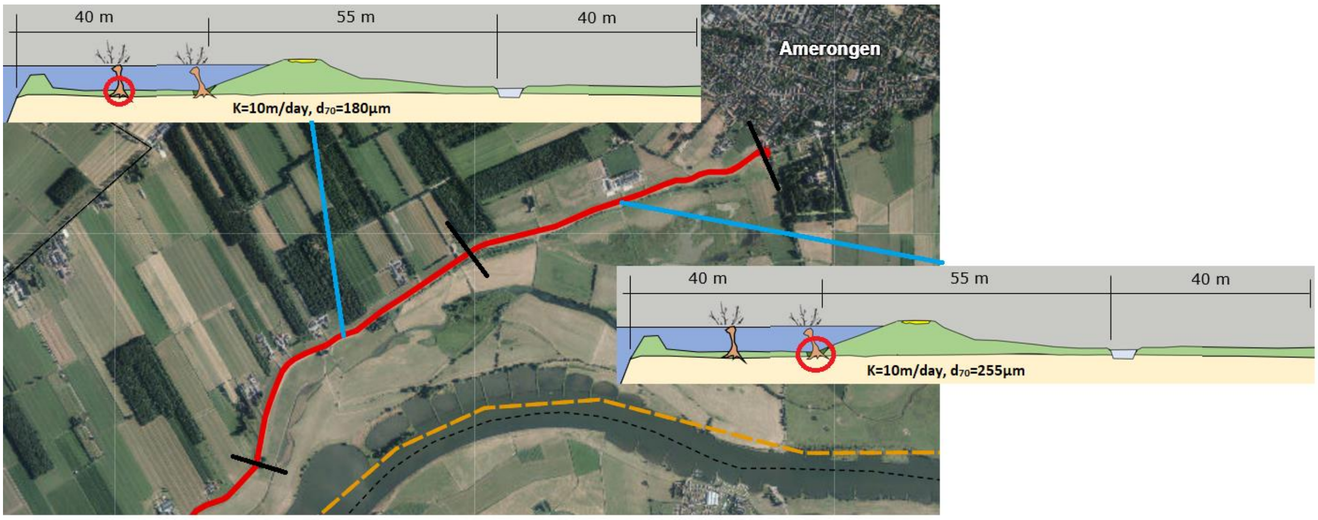


Figure 56 used scenario for estimated budget additional soil investigation and monitoring (scenario 10-K-I)

Investment costs monitoring and soil investigation

First an estimate is made of the investment costs required for monitoring and soil investigation for this case. The investment costs are divided over the project phases were investment is needed and over the measurement methods. The investment cost is determined as a percentage (Table 22) of the original project costs specified in chapter 3 and in euros (Table 25). The monitoring and soil investigation consists of two HPT-borings, borings and taking samples for the d_{70} at a 100m interval, and also 4 piezometers placed in the aquifer at a 100m interval. The repeated costs for monitoring over the project costs is based on the need for maintenance of the monitoring system. The yearly maintenance costs are based on 10% of the initial investment costs (Voortman, 2018). The estimated project costs of the base scenario is approximately 13,7 M€.

For scenario 10-K-I the measurements into the d_{70} and K do not reduce the project scope. This means no additional benefit is gained from doing additional measurements during the initiation phase. So the soil investigation and the HPT boring tests can be performed at the start of the reconnaissance phase. The increase in seepage length also does not affect the project scope, but as mentioned before monitoring is most efficient if there is a long measurement period. Therefore the largest benefit is found for when monitoring is started as early as possible in the project. But to implement a monitoring plan, the soil composition at the dike trajectory must be known to determine where the piezometers must be placed. Therefore a portion of the budget for soil investigation is suggested to be used in the initiation phase. In Table 22 half the total required investment costs for soil investigation were reserved in the initiation phase.

Investment costs as percentage of estimated project cost	initiation phase	reconnaissance phase	elaboration phase
monitoring	1,4%	0,1%	0,1%
soil investigation	1,0%	1,0%	0,0%
HPT boring	0,0%	1,0%	0,0%

Table 22 investment cost required in each phase as a percentage of the estimated project costs of the base scenario

Most of the budget division shown in Table 22 is spend in the earlier project phases with little available during the elaboration and realisation phase. Having no budget for monitoring and soil investigation will make it impossible to perform soil research in case unforeseen circumstances might require them. It might therefore be useful to reserve part of the budget for additional research in the later project phases in case in the later project phases it turns out that additional information is needed to complete the design.

The results for in which phases there needs to be investment in monitoring and soil investigation can be compared with the programme for dike reinforcements of the HWBP for 2019-2024 (HWBP, 2019) and the project structure described by the HWBP (Bernardini, 2017). The result for soil investigation and the HPT-MPT is similar to the project process described in the documents. The reconnaissance phase comes out as the most beneficial phase to perform the soil investigation and HPT borings. Only for the results in Table 22 part of the budget for soil investigation was reserved in the initiation phase to accommodate for implementing a monitoring plan.

However, there is a difference with the project structure described by the HWBP in when monitoring is applied. As current policy, the budget becomes available during the reconnaissance phase. It is possible to include a pre-reconnaissance in which the scope of the project can be determined. But there can be a large period of time between the recommended initiation phase and when the budget is planned to be available in the programme.

For this reason it is recommended to allow the possibility to receive funding to apply monitoring if applicable to the dike trajectory. If a monitoring plan can be applied after the dike inspection deems the dike trajectory is unsafe, the expected benefit from monitoring is higher. One possible solution is to determine monitoring criteria for the application of a dike trajectory to the HWBP programme for dikes deemed unsafe during the safety inspection. These criteria would apply to dike trajectories vulnerable to relevant failure mechanisms. This will guarantee the scope of the applied projects will be more accurate. But to do so, budget and guidance need to be available to ensure the monitoring plan is applied correctly.

Expected project cost reduction

The resulting expected benefit of monitoring and soil investigation for this dike trajectory is analysed (Table 23). The investment costs is plotted against the expected reduction in project costs. Both are a described as a percentage of the total project costs of the base scenario (Figure 57). The investment costs for monitoring and soil investigation are included in the new project costs for the scenario.

	Expected project cost section 1 (E[M€])	Expected project cost section 2 (E[M€])	Expected project cost dike trajectory (E[M€])	Investment costs monitoring and soil investigation (M€)	Total expected project cost (E[M€])	Total expected project cost reduction (E[%])
Started in year 0 (initiation phase)	5,4	5,0	10,4	0,6	11,0	19,4
Started in year 6 (reconnaissance phase)	5,4	5,9	11,3	0,53	11,9	13,3

Table 23 expected project cost reduction for scenario 10

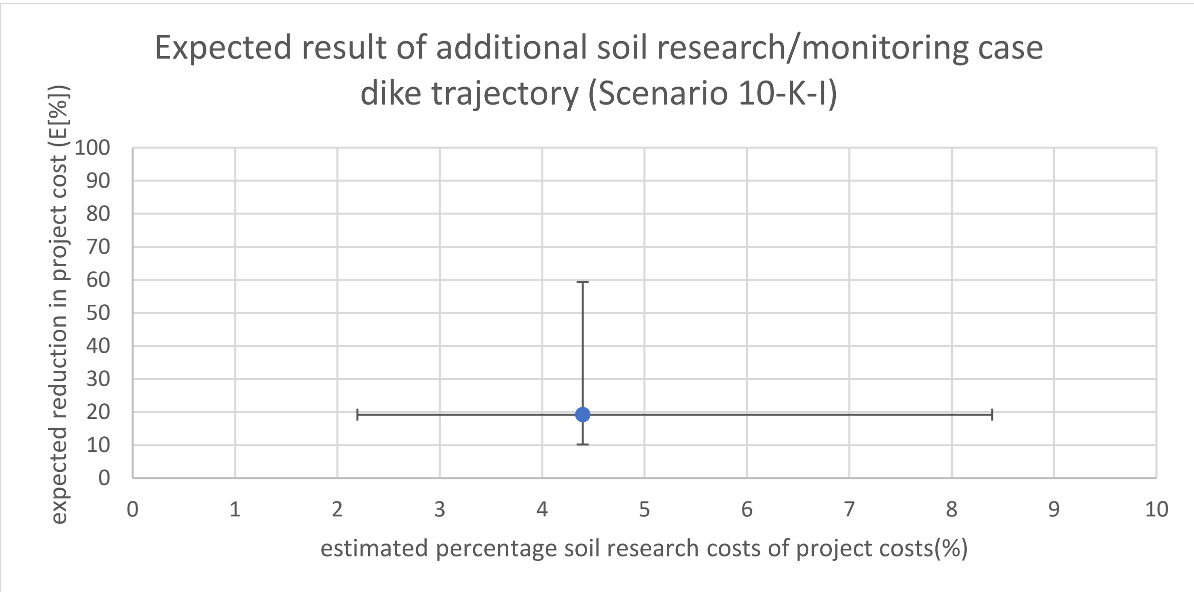


Figure 57 result cost benefit analysis of soil research for scenario 10-K-I

The expected reduction in project costs for the scenario is approximately 19% of the original project costs. Also the uncertainty in the end result is shown in the graph. The lines show the lower and upper bound of the result for the applied parameters, and how the values were calculated is shown in Table 24. For example for the d_{70} the mean of the lithology class is used as mean d_{70}, so to determine the upper and lower the upper and lower limit of the range for the lithology class. This was applied for the parameters for which based on the estimation of the probabilities lead to the highest expected project cost reduction.

From Figure 57 it can be read that there is a large uncertainty in what the expected reduction in project costs are, varying between 10% and 59%. This is caused by the variation in outcomes of the theoretical case. Since there is for example no data on the effective length of the foreshore, the seepage length can vary between 55 and 95m, which is almost twice the length. The accuracy can be increased by applying a practical case, for which the data can be used to verify the results of this thesis and to reduce the uncertainty. Given that the fictional entry point of the foreshore can be further away from the dike than the obstacles connected to the aquifer (Roode, 2019) which is assumed in this thesis, the expected project cost reduction will be higher than described here.

	Sections	d_{70}	K	L	Reinforcement needed
Upper bound	1	300	10	55	21m piping berm
	2	180	10	95	-
Lower bound	1	210	10	55	15m sheet pile wall
	2	180	10	55	19 sheet pile wall

Table 24 upper and lower bound parameters for expected cost reduction

The upper and lower bound is also calculated for the monitoring and soil investigation methods. This was done by varying the interval for the monitoring and the soil investigation, and by varying the amount of HPT-borings that are used to get the result. The interval of the monitoring and soil investigation was 50m for the upper bound and 200m for the lower bound. The amount of HPT borings is 4 for the upper bound and 2 for the lower bound. This results of an uncertainty in the investment costs of between approximately 2% and 8% (Table 25).

	Lower bound		Upper bound
Estimated investment costs soil investigation (M€)	0,14	0,27	0,54
Estimated investment costs HPT boring (M€)	0,05	0,1	0,15
Estimated investment costs monitoring plan (M€)	0,12	0,23	0,47
Total estimated investment costs (M€)	0,30	0,6	1,16
Total estimated investment costs as percentage of estimated project costs base scenario (%)	2,2	4,6	8,4

Table 25 upper and lower bound monitoring and soil investigation

To determine what the benefit is of starting monitoring and soil investigation early in the project, the results can be compared to what the expected project cost reduction is when the project structure described by the HWBP is applied (Bernardini, 2017). For the described dike reinforcement project, the additional soil investigation and monitoring start in the reconnaissance phase. If this is applied to this case, the expected project cost reduction compared to start of the project is 13,3%. The benefit in starting early is 6,1% of the estimated project costs at the start of the project (of the base scenario). This is achieved by an investment of estimated €70.000,-, which is approximately 0,5% of the estimated project costs of the base scenario.

6.1.2 Application to different cases

The result of the costs and benefits of soil investigation and monitoring described in chapter 6.2.1 is only the expected result from one case. If representative values are to be determined for approximate estimation of how much of the project budget needs to be spent on which soil research method, results of different dike reinforcements projects which have been finished are to be included.

One difficulty with analysing the benefit of soil research from the start of the project when the dike trajectory is deemed unsafe to the start of the realisation phase of the dike reinforcement for different projects, is that the result depends on how accurate the cost estimations at these two points in the project are. Especially the cost estimation at the earlier phases can vary wildly depending the available information and used design, as the applied cost margins in these phases contain a wide range (Voortman, 2018).

A different method than the one described in this thesis that could be applied to finished projects is to compare the investment costs into soil research in the project with the exceedance of the budget for construction costs. Such an analysis has been performed for road construction by the British Highway Agency, for which the results are shown in Figure 58. Here the soil research costs in ratio to the construction costs specified in the tender are compared with the recorded exceedance of the construction costs.

When comparing the results in Figure 58 and this thesis, the analysed benefit for the two researches is resultant from different project phases. This is likely to lead to different results. Also the different researches are based on different type of structures, so the researches cannot be directly compared. However, what can be noted is that the results for the investment costs for soil research of the entire budget and the expected benefit from the soil research are of the same order of size for two researches. The investment costs for soil research as part of the total project costs is approximately between 0% and 10%, while the expected benefit expressed in project costs is a factor of 10 larger.

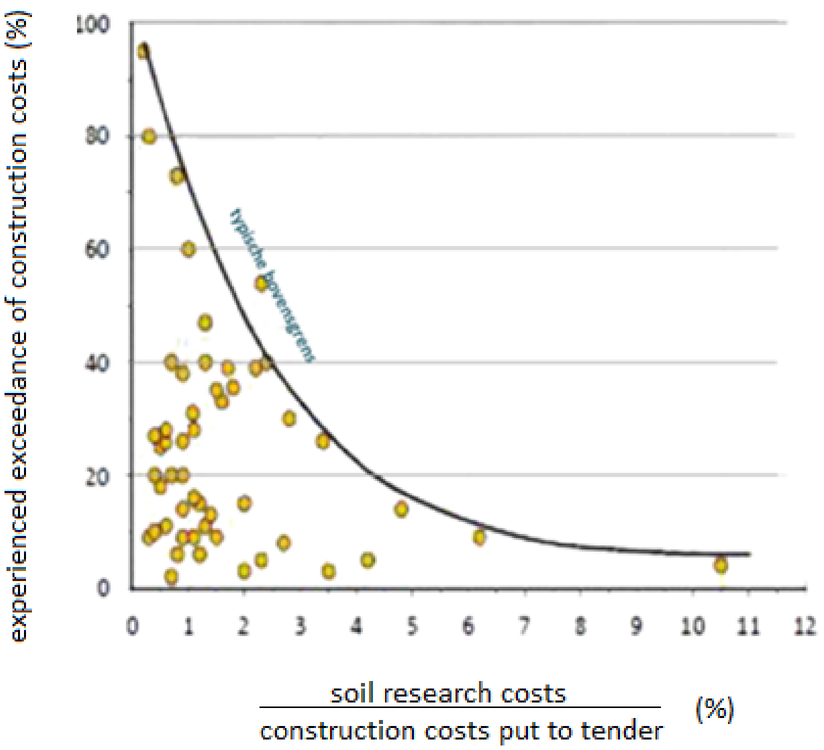


Figure 58: soil research costs in relation to the experienced exceedance in construction costs (UK Highway Agency, 1994)

To apply the results found in this thesis in practice, the results need to be compared to different dike reinforcement projects to verify the result. One way to do so is to create a collection or database of project results in which the investment costs for soil research and the expected benefits are compared. Also the estimate of the relationship between the percentage of the budget spent on soil research and the resultant benefit on the project costs will be more reliable if it is based on different projects. Yet difficulties are predicted to occur when collecting this data from concluded projects which need to be taken into account.

It must be realised that the framework for each project is different, and as such cannot be directly compared. Certain costs which are included under the investment costs for soil research might not be included in the costs for soil research for a different project. Also the costs for the same applied method might be different due to different circumstances or supplier. This makes it more difficult to compare results achieved in one project with results from a different project.

Furthermore evaluation during or short after the project can be complicated due to the confidentiality during the project and the liability of the stakeholders. When the project is put to tender, the supplied documents are to be treated with confidentiality (Minister van Veiligheid en Justitie, 2012). This is done to prevent the competition from gaining information which could give them an unfair advantage. But the confidentiality of data could limit the data available to verify the impact of soil research. What's more, evaluation of the soil research costs and the potential benefits from the soil research will possibly mean the stakeholders are liable to extra costs, which is detrimental to them. This might make the stakeholders more hesitant to openly share the results for the project costs to such a database as described above.

6.1.3 Expert judgement on achieved results

As part of the research an interview has taken place with the HWBP with two goals in mind. The first was to verify if the estimated results for investment costs into monitoring and soil investigation as percentage of the total project costs and the expected reduction in project costs achieved with the monitoring and soil investigation. Since this thesis is based on a theoretical case, it was beneficial to discuss whether the expected results fitted with the experiences within the HWBP. The second reason was to present the results and to discuss which of the findings in the thesis are interesting to their programme and if the HWBP wanted to continue with the research on certain topics covered in this thesis. The interview was conducted with Goaitske de Vries, the innovation coordinator of the programme direction of the HWBP on October 17 2019.

For the verification of the expected results of the investment costs in monitoring and soil investigation and the project cost reduction, the conclusion is that the HWBP possesses no characteristic values for these costs, as they are not recorded from realised projects. It will furthermore be difficult to find data for these characteristics values in old projects, as the documentation on the costs of the project is obscure and not suited for retro-active research in these numbers. More potential can be found in evaluating the investment costs and the reduction in project costs following a project in progress. The data needed for the characteristic values can be collected throughout the project, and expert judgement from the experts involved in the project can be used to estimate the values. The dike reinforcement project near Den Oever of the water board Noorderkwartier was suggested as potential pilot were the information can possibly be collected.

For the second goal, the work done on the possible reduction of project costs and the strategy on determining whether enough information is collected before the project is applied to HWBP can potentially be beneficial for the project costs of dike reinforcement. Especially for measurement methods such as monitoring the head in the aquifer using piezometers for which optimal result is found when the measurement period is longer, it is beneficial that before the application of the project to the programme it is at least analysed whether it is beneficial to implement the method. This will not be in the form of hard requirements of how much funds were invested in monitoring and soil investigation, but more as guidelines on how work with multiple scenarios for the subsoil and whether these can be ruled out.

In combination with the discussion with the HWBP, discussion on the results from this thesis has also taken place with experts from Fugro. The added value for this thesis is that their experience can verify the results on the expected reduction in project costs. Specifically for the results on the estimated investment costs for monitoring and soil investigation as percentage of the total estimated project costs and the expected reduction in project costs.

First of all a discussion with Martin van der Meer, Technical director Fugro Water Services took place where the found results on estimated investment costs as percentage of the estimated project costs were analysed. What was found that the estimated investment costs as 5% of the total project costs found in this thesis are comparable to at least 1 other dike reinforcement project in which Fugro is currently involved. One problem however is that the data is on file, so the details on the project cannot be published due to privacy. This is a problem for future research that was also previously stated in the discussion, that the required data from other projects to analyse the made investment in monitoring and soil investigation and the achieved results of these methods is confidential.

Secondly Fugro has conducted their own research on the effect on the project costs for a more accurate representation of the hydraulic conductivity of the aquifer. Their research focuses on the effect on piping when the heterogeneity and anisotropy of the aquifer are included, instead of the bulk hydraulic conductivity used in Sellmeijer's equation (Stoop, 2018). Here it was found that including these factors for the hydraulic conductivity could lead to a decrease of required seepage length between 20% and 60%. This is comparable to the results for the scenarios of a lower hydraulic conductivity in this research, for which the decrease in required seepage length for the case dike trajectory is 23%. However, verification of the results for the grain size distribution and the effective foreshore are still required.

6.2 Recommendations

This thesis serves as a first step to determine the advantages of implementing monitoring and soil investigation early in the project. To improve the accuracy of the results, it is beneficial to collect information from different projects than the dike trajectory at Amerongen. This might also demonstrate in which situations the monitoring and soil investigation described in this thesis are applicable. However, from this first step already a few recommendations can be made.

Based on the project structure defined by the HWBP, two key points in the project have been identified which can be used to help incorporate monitoring and soil investigation in the optimal project phases. These are for the application of a dike reinforcement project and for the verification of the results of this thesis.

Application dike reinforcement project to the HWBP programme
First of all, the application of a dike reinforcement to the HWBP programme could be altered. The result of the initiation phase is the fixed project scope, a defined framework of the project and the expectations for the project (Bernardini, 2017). This is used to apply the dike reinforcement project to the HWBP programme. To get the most result from a monitoring plan and possibly already reduce the scope of the project, it is ideal to implement a monitoring system in the dike trajectory during this phase. Also the soil investigation needed to determine where the sensors need to be placed needs to be taken into account.

To guarantee these methods are implemented in the relevant cases before the project is applied, criteria can be added to the application process that data collection must already have been started before the project is applied. It is difficult to put criteria on what results must have been achieved due monitoring before the application as there is a chance not enough relevant measurements have been collected yet. But overall the benefit of implementing monitoring for cases comparable to the case used for this thesis the result is estimated to greatly reduce required amount of dike reinforcement needed.

If a part of the soil investigation is moved to the initiation phase and a monitoring plan is implemented, budget needs to be available to perform the measures. Currently it is possible to implement a pre-reconnaissance phase for the additional measurements in which budget is available, but it can be more beneficial to also provide a budget in the initiation if it is found likely that the additional measurements could reduce the project cost and risk considerably.

Verification results thesis
To verify the results from the research done for this thesis, data needs to be collected from different dike reinforcement projects in what project phases which monitoring or soil investigation method was applied, what the costs where and what the impact of the information was on the project. This might be done in the same method as was applied in this thesis, but since the cost estimation for earlier project phases still contain a wide cost margin the results will probably vary.

The method described in chapter 6.2.2. assessing the impact of monitoring and soil investigation based on the exceedance of construction costs is applicable to analysing practical cases. This can be done at the project evaluation after the project has been realised, to insure the privacy of the participant's information and willingness to discuss what could have been done better. Or a pilot project can be used were the participants agree to meet multiple times during the project and assess what the investment for collecting information has been in the project phases and what the experienced benefit has been. Based on the meeting with the HWBP, this is more effective than collecting all the information after the project is finished.

Furthermore, only one failure mechanism was considered for the given case. Only piping was considered due to the vulnerability of the case dike trajectory to the failure mechanism. However, this does not imply that monitoring and soil research are only beneficial for dikes vulnerable to piping. For macro-stability for example, the layout of the phreatic line becomes important. The phreatic line can also be more accurately placed if more measurements are available. Also for instability due to uplift the design could benefit from measurements of the hydraulic head. It is therefore also recommended to analyse the benefits of early implementation of monitoring and soil investigation for different failure mechanisms.

So in short summary, these are the recommendations of this research

- Set guidelines for the implemented monitoring and soil investigation plan before the dike reinforcement project can be applied to the HWBP programme so the uncertainty in the soil profile is addressed. A small part of the budget is made available earlier to make such a plan feasible.
- For verification of the expected benefit in early monitoring and soil investigation results from other projects are incorporated. The results can be collected during the evaluation of the project after the realisation of the project. But preferably the analysis is performed on a running project for its entire length.
- The benefit of early monitoring and soil investigation will also be analysed for different failure mechanisms.

6.3 Limitations thesis

For practical reasons or due to time constraints, some aspects of design or interest have been omitted from the research. This chapter will list these limitations to the research.

- It must be considered that obstacles such as farms, roads or hydraulic structures are not taken into account. Including every element present in the dike trajectory would consume a lot of time while the added value to determining general monitoring strategies is minimal. But if a monitoring strategy is implemented in the case trajectory, their effect on the system need to be taken into account. One example is the presence of the nature reserve in a part of the foreshore of the dike trajectory. The nature reserve will have consequences for the maintenance and judicial agreements (Roode, 2019).

- Due to the vulnerability of the dike trajectory to piping, piping is the only failure mechanism that is included in the thesis. This choice is case-specific, and for different situations other failure mechanisms need to be considered. For other failure mechanisms different aspects of the dike need to be analysed, as other failure mechanisms depend on different parameters of the dike. For macro-instability for example next to the soil composition the layout of the phreatic line becomes important (Jonkman, 2018). As such, different monitoring and soil investigation strategies will most likely need to be implemented.

- The realisation phase and the maintenance phase of the dike reinforcement project are not taken into account. These are outside the scope of the thesis. The value in euros of the benefit in project cost reduction is set as 0.

- One of the assumptions that was made for this research is that for each project phase, certain decisions are made for corresponding to the project phase. To be able to assign value to the information in each of these phases, it was assumed that these decisions cannot be overturned later on in the project. In reality, there is most likely some flexibility in the project to make changes to the design. If for example it turns out in the elaboration phase that a different preferred alternative might be more profitable, it can still be possible to change the preferred alternative.

- Another assumption is that all information is effectively transferred between project phases. The idea of early monitoring and soil investigation is that the information obtained from the soil research becomes available for multiple or all project phases. This however does not have to be the case. If different parties or stakeholders are involved and the information is not correctly collected and transferred information from previous project phases might be lost. So effort must be made for all information to be collected and transferred to the next project phase.

- For the relation between the investment costs for soil research and the expected reduction in project costs, the relation between the parameters is assumed to be independent. This will most likely not be the case, as the hydraulic conductivity of the aquifer and the grain size distribution of the upper layer of the aquifer are most likely correlated to each other (Berbee, 2017).

- Only the aspect of project costs was taken into account. This is not the only factor for the final design. Available space, environmental and social aspects can all have effect on what the final design will be. This will possibly negatively reduce the benefit of monitoring and soil investigation in the dike trajectory. For example, it will be less beneficial to have a more accurate insight whether it is economically more viable to use a piping berm or a sheet pile wall, if there is no space to build a piping berm anyway.

7. Conclusions

The aim of this thesis was to answer the following question:

What will be the benefits to implement monitoring and soil investigation methods early in the project?

To better answer this question, a number of sub-questions were identified at the start of the research. The answers to each of these individual questions will be discussed below.

What type of benefits can be found by implementing a monitoring plan and soil investigation for the dike trajectory?

Monitoring and soil investigation are helpful in dike reinforcement projects by identifying which sections of the dike are vulnerable and which are more resistant. The vulnerable sections need to be identified to correctly assess the risk of dike failure, while identifying the strong sections leads to a smaller dike reinforcement and lower project costs. The two main types of benefits obtained through monitoring and soil investigation are reducing the project costs of dike reinforcement and accurately assessing the expected annual risk of dike failure.

In the theoretical case, what is the scale of the benefit for each project phase in the project period?

The scale of the benefits depends on the project phase in which the monitoring is started or the soil investigation has been executed. The earlier the information on the subsoil is collected, the more project decisions the information can be used for. For monitoring the length of the monitoring period is also relevant, as a longer measuring period will have a higher probability of relevant data being collected. The benefit will also depend on the local characteristics of the dike trajectory that is being analysed. For the case used in this thesis the foreshore was not included in the base scenario as a conservative approach since there was no data on the effect of the foreshore and there was a large uncertainty in the hydraulic conductivity of the aquifer. When the entire foreshore is effective and this information can be added to the seepage length the safety standard of the dike trajectory is met. This made scenarios possible for which dike reinforcement was not necessary.

Whether it is economically beneficial to execute a monitoring plan or additional soil investigation depends on three factors. These are the estimated investment costs, expected benefit from the investment and the probability of occurrence of a specific scenario with the expected benefit. The investment costs for monitoring and soil investigation and the expected reduction in project costs can be calculated based on cost estimates derived from previous projects. However, the difficulty is to determine the probability that a scenario actually represents reality.

How large are the benefits of investing in early monitoring and soil investigation compared to the investment costs?

The investment costs have been determined as a percentage of the total estimated project costs, based on cost estimates from experts at Fugro and from literature. The applied monitoring and soil investigation are two HPT-borings, borings and soil samples taken at an interval of 100m and 4 piezometers placed in the aquifer over the width of the dike at an interval of 100m.This leads to an investment cost of 4,6% of the estimated project costs. The benefit of implementing these measures is an expected reduction in project costs for the case is 19% of the original estimated project costs.

When it turns out the dike is more vulnerable than first anticipated, what is the effect for the dike reinforcement project?

To determine the risk of a weaker section present in the dike trajectory, the risk of a sand lens being present under the case dike trajectory and going unnoticed given the probe interval was calculated. For the commonly applied interval of soil investigation of 100m, the annual risk of dike failure due to missing the sand lens is 3,7M€/year. By using a smaller interval at an extra cost of 0,8M€/year, the annual risk could be reduced by a factor of 1000, which lowers the risk due to an undiscovered weak section of the dike trajectory such that it is lower than the safety standards. For the case used in this thesis, the common probe interval of 100m leads to a higher risk than is acceptable compared to the safety requirements when the described sand lens is expected to occur.

What is the effect of early monitoring and soil investigation?

The benefit in performing additional soil investigation and monitoring is dependent on the project phase in which the measurements are started and/or performed. The aim of this research was to determine when these methods need to be implemented, how much investment is needed and what the expected benefit of the measures is. The benefit can be expressed in lower expected project costs in case of dike sections which are more resistant to piping than first expected, and in expected annual risk if vulnerable dike sections are identified. For the applied case dike trajectory at Amerongen the expected reduction in project costs is 19,4% of the initially estimated project costs if the measurements were performed or started in the initiation phase. In comparison, the expected project cost reduction for the common project structure is 13,3% for starting in the reconnaissance phase. The investment costs due to monitoring and soil investigation was 4,6% of the estimated project cost when starting early and 4,1% for the common project structure. In conclusion, by investing 0,5% of the estimated project costs on starting monitoring and soil investigation early in the dike reinforcement project a 6,1% reduction in the project costs is expected. It can therefore be concluded that investing in early monitoring and soil investigation is cost-effective.

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Appendix A: probability distribution functions of the failure mechanisms

Piping
To determine what failure mechanism is dominant for the case dike trajectory, the fragility curves for piping, overtopping and macro-instability were plotted. The fragility curves were determined by calculating the failure probability for different water levels at an interval of 0,5m. The dominant failure mechanism was determined to be piping, for which the fragility curve is shown in Figure 59. The calculation steps for this figure can be found in Appendix B.

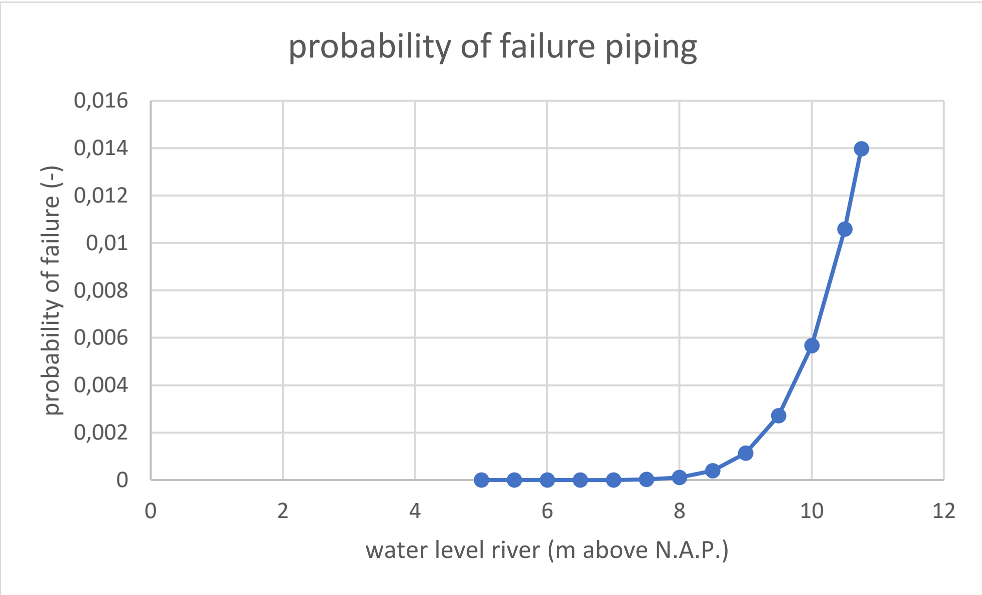


Figure 59 Fragility curve piping

Overtopping
The same interval for water levels was applied to calculate the fragility curve for the failure mechanism overtopping. To determine the mean overtopping rate per unit length, the van der Meer and Bruce equation is used (Equation 15 and 16). In order to do so, the fetch (F) and the wind speed (U_{10}) were estimated to calculate the wave height and wave period. The fetch was estimated using satellite images of the river at the dike trajectory, and the distribution of the wind speed was approximated for the wind near Schiphol (KNMI, 1983). A Weibull distribution was used for the wind speed, for which the parameters are shown in Table 26. The reduction factors for the roughness of the slope, the presence of a berm and oblique wave attack were determined for the dike trajectory. All used parameters can be found in Table 27.

Parameters Weibull distribution wind speed U10	u	k	ε
	6,5	2,1	0

Table 26 parameters weibull distribution U10

Parameter	mean	s.d.	Characteristic value	unit
F	200	20	233	m
tan(α)	-	-	0,6	(-)
γ_m	-	-	0,95	(-)
γ_a	-	-	1	(-)
γ_b	-	-	1	(-)

Table 27 parameters used for overtopping

Using the determined parameters the significant wave height (Equation 11), significant wave period (Equation 12) and wave length (Equation 13) can be determined (Jonkman,2018). Using the slope of the dike, the significant wave height and the wave length the irribarren number can be calculated (Equation 13).

$$\frac{\mathrm{g}*\mathrm{H_s}}{\mathrm{U_{10}}^2} = 0{,}283 * \tanh(0{,}0125 * (\frac{g*F}{U_{10}{}^2})^{0{,}42} \quad [11]$$

$$\frac{\mathrm{g}*\mathrm{T_s}}{U_{10}} = 7{,}54 * \tanh(0{,}077 * (\frac{g*F}{U_{10}{}^2})^{0{,}25} \quad [12]$$

$$L_o = \frac{g*T_s{}^2}{2*\pi} \quad [13]$$

$$\xi = \frac{\tan(\alpha)}{\sqrt{\frac{H_s}{L_o}}} \quad [14]$$

Using Equation 15 and Equation 16 the failure probability can be determined using a FORM-analysis performed in Prob2B. A deterministic value for the water level relative to the N.A.P. at an interval of 0,5m was used to create the fragility curve of the failure probability due to overtopping, which is shown in Figure 60.

$$\frac{q}{\sqrt{g*H_{mo}{}^3}} = \frac{0{,}023}{\sqrt{\tan(\alpha)}} * \gamma_b * \xi_{m-1,0} * \exp(-\left(2{,}7 * \frac{R_c}{\gamma_b*\gamma_f*\gamma_\beta*\gamma_v*\xi_m*H_{mo}}\right)^{1{,}3}) \quad [15]$$

With a maximum of:

$$\frac{q}{\sqrt{g*H_{mo}{}^3}} = 0{,}09 * \exp(-\left(1{,}5 * \frac{R_c}{*\gamma_f*\gamma_\beta*H_{mo}}\right)^{1{,}3}) \quad [16]$$

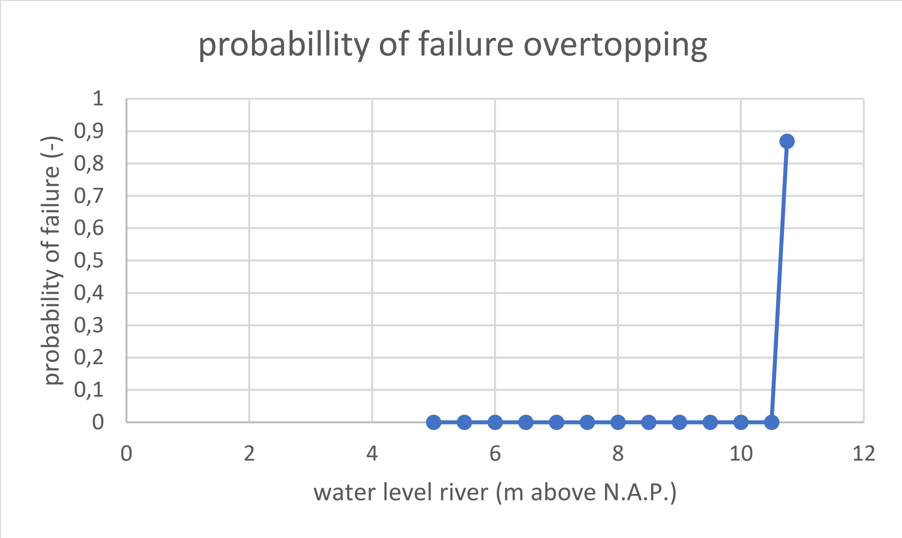


Figure 60 Fragility curve overtopping

Macro-stability

To determine the failure probability of the dike trajectory due to macro-instability, a cross-section of the dike was modelled using D-stability with the parameters shown in Table 28. The phreatic line was schematised based on a clay dike on a sand layer (Deltares, 2017).

	mean	c.o.v.	s.d.	distribution	R(5%) resistance	R(95%) load
unsaturated unit weight sand	23	0,1	2,3	lognormal	19,4	27,0
saturated unit weight sand	26,5	0,1	2,65	lognormal	22,4	31,1
unsaturated unit weight clay	18	0,1	1,8	lognormal	15,2	21,1
saturated unit weight clay	20	0,1	2	lognormal	16,9	23,5
cohesion sand	0	0	0	deterministic	-	-
cohesion clay	5	0,1	0,5	lognormal	4,2	5,9
friction angle sand	30	0,1	3	lognormal	25,3	35,2
friction angle clay	30	0,1	3	lognormal	25,3	35,2

Table 28 parameters model D-stability

The program uses the Bishop method to determine the factor of safety for the failure mechanism of macro-stability. Then a calibration equation (Equation 17) can be used to determine the reliability index for the failure mechanism, and with that the probability of failure.

$$\beta = \frac{\frac{FoSd}{\gamma_d} - 0{,}41}{0{,}15} \quad [17]$$

The phreatic line was plotted for the guidelines of a clay dike on top of a sand layer (Deltaris, 2017). These calculations are repeated for different water levels in the river at an interval of 0,5m. This way the fragility curve for the failure mechanism of macro-stability can be plotted (Figure 63). A sketch of the used model for D-stability is included below in Figure 62.

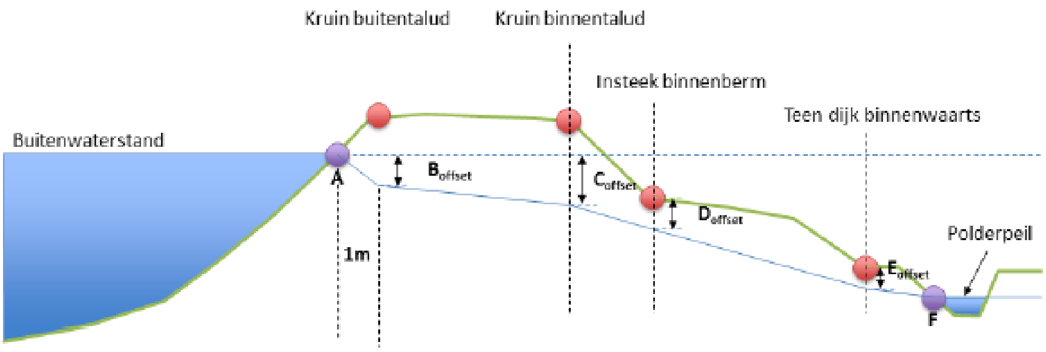


Figure 61 schematization phreatic line dike (Deltares, 2017)

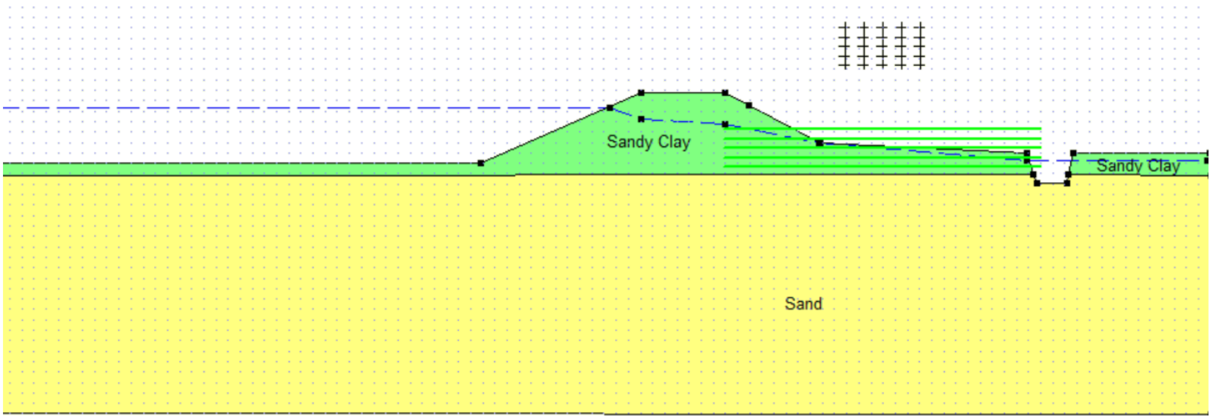


Figure 62 sketch of the applied model in D-Stability

The performed calculations described above lead to the fragility curve in Figure 63. The probability of failure due to macro-instability starts low but increases exponentially after 8m above N.A.P. This is because as the phreatic line in the dike becomes higher, the pore pressures increase and the shear strength of the soil is lowered. This decreases the stability of the dike (van Montfoort, 2018).

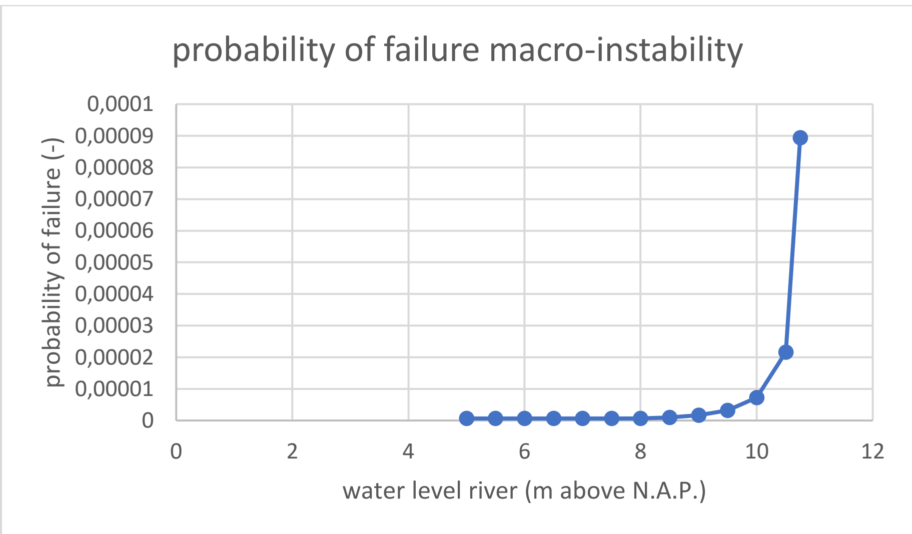


Figure 63 fragility curve macro-instability

Appendix B: semi-probabilistic analysis of the theoretical case for piping

The probability of failure for the dike due to piping is determined based on the system reliability of three sub-mechanisms for piping. These are uplift, heave and piping. The required annual failure probability due to piping is calculated in Table 1. The probability of failure for the base scenario for the three sub-mechanisms is calculated in their respective sections. A semi-probabilistic approach was used to calculate the annual failure probability of the sub-mechanisms. Characteristic values were used for the parameters, which are shown in Table 29.

Parameter	unit	Symbol	Distribution	mean value	s.d.	R(5%) resistance	R(95%) load
Hinterland phreatic level	[m + NAP]	h_p	Normal	5	0,5	4,18	5,83
Thickness hinterland blanket	[m]	d	Lognormal	0,5	0,05	0,42	0,59
Length (effective) foreshore	[m]	L_{eff}	Lognormal	0	0	0	0
Width levee	[m]	B	normal	55	5,5	45,93	64,1
Aquifer thickness	[m]	D	Lognormal	20	2	16,89	23,45
Saturated volumetric weight blanket	[kN/m3]	γ_{sat}	Normal	16	1,6	13,36	18,64
Saturated volumetric weigth water	[kN/m3]	γ_w	Deterministic	10	n.a.		
Volumetric weigth sand grains	[kN/m3]	γ_s	Deterministic	26,5	n.a.		
Critical heave gradient	[-]	$i_{c,h}$	Lognormal	0,7	0,07	0,58	0,82
Bedding angle	[deg]	θ	Deterministic	37	n.a.		
Kinematic viscosity	[m2/s]	v	Deterministic	$1{,}3*10^{-6}$	n.a.		
Constant of White	[-]	η	Deterministic	0,3	n.a.		
Gravitational constant	[m2/s]	g	Deterministic	9,81	n.a.		
70%-fractile of grain size distribution	[m]	d_{70}	Lognormal	$1{,}8*10^{-4}$	$1{,}8*10^{-5}$	$1{,}5*10^{-4}$	$2{,}1*10^{-4}$
Reference value for d70	[m]	d_{70m}	Deterministic	$2{,}1*10^{-4}$			
Hydraulic conductivity aquifer	[m/s]	k	Lognormal	$1{,}2*10^{-4}$	$1{,}2*10^{-5}$	$9{,}7*10^{-5}$	$1{,}4*10^{-4}$
Hydraulic conductivity aquitard	[m/s]	k_h	Lognormal	$1{,}2*10^{-6}$	$1{,}2*10^{-7}$	$9{,}6*10^{-7}$	$1{,}3*10^{-6}$
Model factor uplift	[-]	m_u	Normal	1	0,1	0,84	1,17
Model factor piping	[-]	m_p	Normal	1	0,1	0,84	1,17

Table 29 characteristic values for the parameters

Uplift.

The characteristic values which were used for the calculations are based on their contribution to the failure mechanism as either a load (95% exceedance) or a resistance (5% exceedance). The seepage length is assumed to be the sum of the effective foreshore length, the width of the dike and leakage length of the hinterland (Equation 2). The values in Table 30 are used to calculate the critical head difference. The applied equations are shown in Equation 18, 19 and 20 (Jonkman, 2018).

$$\Delta\phi_{c,u} = d * \frac{\gamma_{sat}-\gamma_w}{\gamma_w} \quad [18]$$

$$\Delta\phi = \phi_{exit} - h_p \quad [19]$$

$$FoS_u = \frac{m_u * \Delta\phi_{c,u}}{\Delta\phi} \quad [20]$$

The calculated factor of safety for the base scenario is 0,05, which is shown in Table 30. Using the reliability index for a failure probability of 1/40.000, Equation 21 can be applied for converting the factor of safety to the reliability index for uplift. By calculating the standard normal distribution function of the reliability index the probability of failure for heave can be determined.

$$\beta_u = \frac{(\ln\left(\frac{FoS_u}{0,48}\right) + 0,27 * \beta_{norm})}{0,46} \quad [21]$$

uplift		
saturated volumetric weight	13,36	γ_{sat}
volumetric weight of water	10	γw
thickness of blanket	0,42	d
critical head difference	0,14	$\Delta\varphi_{c,u}$
length effective foreshore	0	Lf
dike width	45,9	B
thickness aquifer	23,45	D
hydraulic conductivity aquifer	$1,9*10^{-5}$	K
hydraulic conductivity aquitard	$1,3*10^{-6}$	K_h
leakage length hinterland	37,3	λ_h
damping at the toe	0,45	λ
water level	9,25	h
head at landside	4,2	h_p
head at the toe	6,45	φ_{exit}
head difference	2,28	Δφ
Factor of Safety uplift	0,05	FoS_u
reliability index	4,1	β_{req}
reliability index uplift	-2,43	β_u
probability of failure uplift	0,99	

Table 30 factor of safety for uplift mechanism

Heave

To determine the probability of failure for heave, the factor of safety for heave is calculated by determining the critical heave gradient (Equation 22 and 23). The factor of safety for the base scenario is 0,11 (Table 31). As for the uplift sub-mechanism, the characteristic values were used for loads and resistances.

$$i = \frac{\phi_{exit} - h_p}{d} \quad [22]$$

$$FoS_h = \frac{i_{c,h}}{i} \quad [23]$$

Heave		
critical heave gradient	0,58	i_{ch}
head at the toe	6,5	φ_{exit}
head at landside	4,2	h_p
blanket thickness	0,42	d
exit gradient	5,4	i
Factor of safety heave	0,11	FoS_h
reliability index	4,1	β_{req}
reliability index heave	$3{,}6*10^{-3}$	β_h
probability of failure heave	0,5	

Table 31 factor of safety heave mechanism

Equation 24 can be used to determine the reliability index for heave (Jonkman, 2018). By calculating the standard normal distribution function of the reliability index the probability of failure for heave can be determined.

$$\beta_h = \frac{\ln\left(\frac{FoS_h}{0,37}\right)+0,27*\beta}{0,46} \quad [24]$$

Piping

Finally the probability of failure for the sub-mechanism of piping is determined. To calculate this probability of failure the critical head difference is determined. Three factors are needed to determine the critical head difference (Equation 25 to 28).

$$H_{c,p} = F_1 * F_2 * F_3 * L \quad [25]$$

$$F_1 = \eta * \left(\frac{\gamma_s}{\gamma_w} - 1\right) * \tan(\theta) \quad [26]$$

$$F_2 = \frac{d_{70m}}{\sqrt[3]{\frac{v*k*L}{g}}} * \left(\frac{d_{70}}{d_{70m}}\right)^{0,4} \quad [27]$$

$$F_3 = 0,91 * \left(\frac{D}{L}\right)^{\frac{0,28}{\left(\frac{D}{L}\right)^{2,8}-1}+0,04} \quad [28]$$

piping		
White's constant	0,3	η
volumetric weight sand grains	26,5	γ_s
specific weight water	10	γ_w
bedding angle	37	θ
F1	0,37	
thickness aquifer	23,45	D
permeability aquifer	$1{,}9*10^{-5}$	K_h
reference value for d_{70}	$2{,}1*10^{-4}$	d_{70m}
kinematic viscosity	$1{,}33*10^{-6}$	v
specific conductivity	$1{,}9*10^{-4}$	k_{zo}
seepage length	45,93	L
gravitation	9,81	g
F2	0,19	

thickness aquifer	23,45	D
seepage length	45,9	L
F3	1,06	

Table 32 Factors for the piping sub-mechanism

Once the critical head difference is determined, the factor of safety for piping and the reliability index for the failure mechanism can be calculated using Equation 29 and 30. Again the standard normal distribution function of the reliability index is used for the probability of failure, which is shown in Table 33.

$$FoS_p = \frac{m_p * H_c}{h - h_p - 0{,}3 * d} \quad [29]$$

$$\beta_p = \frac{\ln\left(\frac{FoS_p}{1{,}04}\right) + 0{,}27 * \beta}{0{,}37} \quad [30]$$

Critical head difference	3,67	$H_{c,p}$
factor of safety piping	0,62	FoS_p
reliability index	4,1	β_{req}
reliability index piping	3,36	β_p
probability of failure piping	$4{,}0*10^{-4}$	

Table 33 factor of safety for piping mechanism

System reliability

Once the probabilities of the three sub-mechanisms are determined, the probability of failure of the system can be determined. As is shown in Figure 64, the three sub-mechanisms form an AND-system for piping. The correlation between the sub-mechanisms is then needed to determine the failure probability. Since the correlation is unknown, the sub-mechanisms are treated as fully dependent. They are treated as fully dependent as this is the upper bound of the failure probability between independent and fully dependent factors. The resulting failure probability can be found in Table 34.

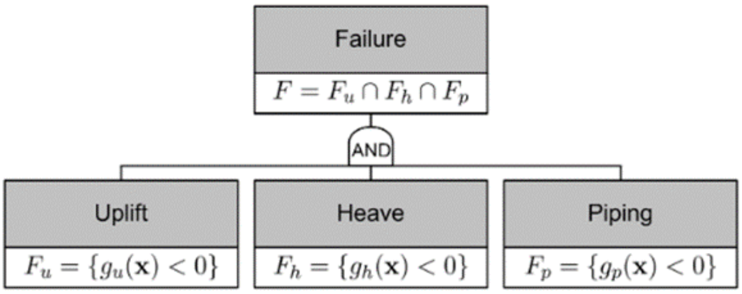


Figure 64 AND-system of the three sub-mechanisms for the piping failure mechanism

probability of failure uplift	0,99
probability of failure heave	0,59
probability of failure piping	$4{,}0*10^{-4}$
independent failure probability	$2{,}0*10^{-4}$
dependent failure probability	$4{,}0*10^{-4}$

Table 34 probability of failure due to piping

To meet the safety standards specified in Chapter 3, some form of dike reinforcement is needed. A piping berm and a sheet pile are used as reinforcement methods. The piping berm increases the seepage length of the dike to make it more resistant to piping. For the sheet pile wall a different calculation is needed, which is shown below. To meet the safety requirements for piping for the soil characteristics used in Table 29, a piping berm of 40m long is needed.

Sheet pile wall

If part of the seepage length under the dike is vertical, the method described above no longer applies. This is the case if sheet pile walls are applied as dike reinforcement against piping. In this case the method of Lane is used. This empirical calculation method uses both the vertical and horizontal seepage length (Förster, 2012). The equation for Lane's method is shown in Equation 31. The characteristic values which were used for the calculations are based on their contribution to the failure mechanism as either a load (95% exceedance) or a resistance (5% exceedance). The used parameters and the required vertical seepage length for the base scenario are shown in Table 35. This corresponds to 16m of sheet pile wall placed at ground level. An extra 3m is needed for the height of the foot of the dike where the sheet pile wall is placed. This is to ensure the factor of safety for heave is larger than or equal to 1,05 (Verruijt, 1995). For the base scenario the required length of the sheet pile wall is therefore 19m long.

$$\Delta H \leq \Delta H_c = \frac{\frac{L_h}{3} * + L_v}{C_b} \quad [31]$$

Lane	unit	Distribution	mean value	standard deviation	R(5%) resistance	R(95%) load
ΔH	m	Normal	4,25	0,425	3,55	4,95
C_b	-	Normal	7	0,7	5,85	8,12
L_{vert}	m	Normal	32	3,2	26,7	37,3
L_{hor}	m	Normal	55	5,5	45,9	64,1

Table 35 values used for Lane's method

Dike reinforcement base scenario

Once the required dimensions for both dike reinforcement methods are determined, it is possible to estimate the project cost for each method. The estimated project costs per m² of dike reinforcement are shown in Appendix C, and the estimated costs of dike reinforcement per m of dike are shown in Table 36.

Estimated cost per m² piping berm (€/m²)	Required length berm (m)	Estimated costs berm per m dike (k€/m)	Estimated cost per m² of sheet pile wall (€/m²)	Required length sheet pile wall (m)	Estimated costs sheet pile wall per m dike (k€/m)
256,12	40	10,2	401,02	19	7,6

Table 36 price dike reinforcement dike scenario

Appendix C: costs calculations measures

For the cost estimations, two categories can be identified. These are the dike reinforcement methods and the monitoring and soil investigation methods. For both categories the costs have been estimated for a given quantity. The estimated costs for dike reinforcement methods are based on literature, and the estimated costs for the monitoring and soil investigation are partly based on experience of comparable projects from Fugro and from literature.

Dike reinforcement methods.

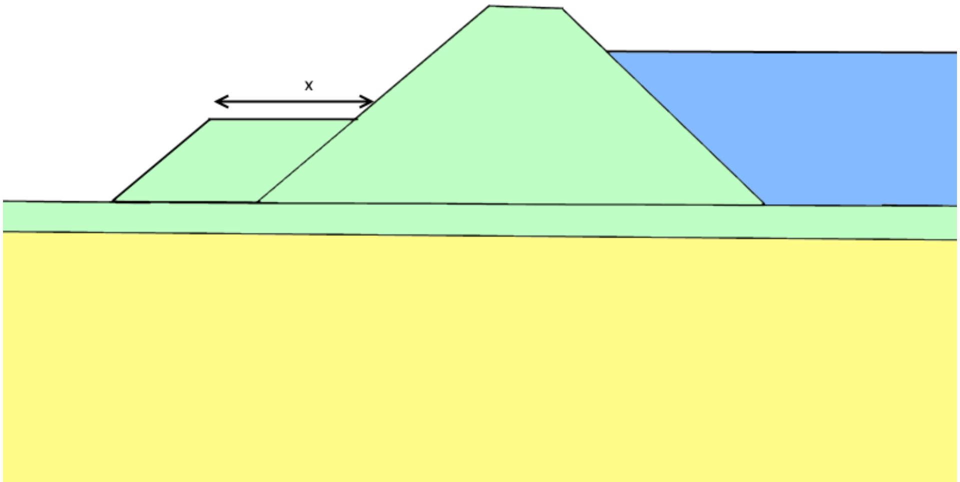


Figure 65: berm costs calculation per x meter berm length for 1 meter of dike

The project costs of constructing a piping berm consist of more than just the construction costs. Overhead, design and unforeseen cost all need to be included as well. (de Grave, 2011). To make an estimate of the project costs based on the dimension of the required reinforcement the project costs have been estimated per m^2 of the required dike reinforcement. All factors mentions above are included as in this estimate (Table 37).

berm	Quantity (-)	Price (€)
processing dike soil m3	3m^3 a €22	66
purchase		
transportation		
processing		
finishing the terrain	1m^2 a €0,25	0,25
replacing fencing		20
moving seepage ditch		50
subtotal		136,25
singular costs	15%	20,44
subtotal		156,69
preparation, direction, design	20%	31,34
subtotal		188,03
sales tax	19%	35,72
subtotal		223,75

unforeseen costs	10%	22,37
purchase soil	1m^2	10
total for 1m piping berm		256,12

Table 37 cost calculations for 1m of dike reinforcement by berm (de Grave, 2011)

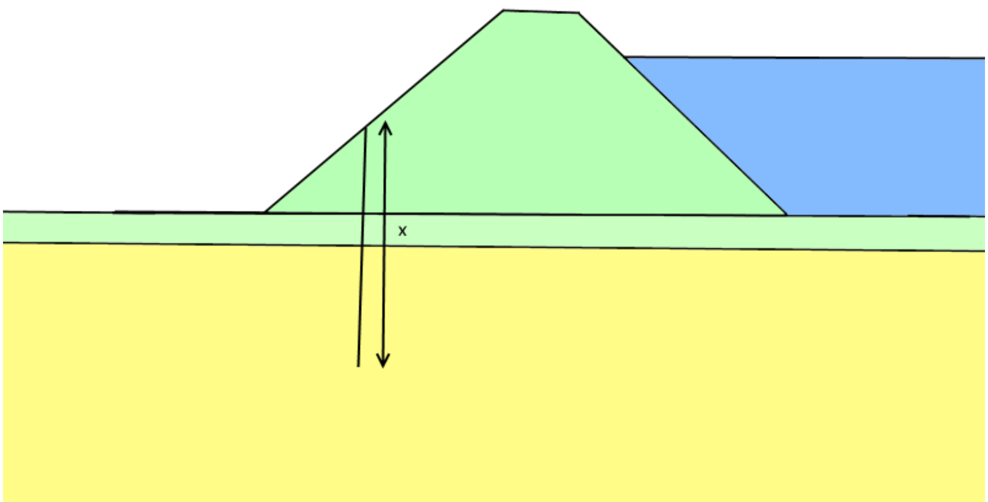


Figure 66: calculation of sheet pile wall per x meter for 1 meter of dike

The same calculation as for the piping berm is performed for the sheet pile wall. The results are shown in Table 38.

sheet pile wall	**Quantity (-)**	**Price (€)**
purchase material	1m^2 a €142	142
transport, piling		80,00
subtotal		222,00
singular costs	15%	33,30
subtotal		255,30
preparation, direction, design	20%	51,06
subtotal		306,36
sales tax	19%	58,21
subtotal		364,57
unforeseen	10%	36,46
total per m2 in dike		401,03

Table 38 cost calculations sheet pile wall (de Grave, 2011)

Monitoring and soil investigation methods.
The estimated costs for the monitoring and soil investigation are determined in two different methods. The costs of the monitoring system was determining how much piezometers would be necessary and what the project costs would be to design and place such a plan based on literature (Table 39). The project costs for the monitoring are based on a system of 4 piezometers spread over the cross section of the dike placed at every 100m of dike. The estimated costs of soil investigation and HPT-borings are based on expert opinion at Fugro based on comparable projects Fugro is working on.

Monitoring system	Quantity (-)	Price (€)
Purchase material		64400
singular costs	15%	9660
subtotal		74060
preparation, direction, design	20%	14812
subtotal		88872
sales tax	19%	16885,68
subtotal		105757,7
unforeseen	10%	10575,77
total for dike trajectory		116333,40

Table 39 cost calculation for dike monitoring system (Bouw, 2015)

The costs for soil investigation and HPT borings are based on the experience of experts at Fugro with comparable projects. From these projects estimates were made of the costs for these measurements.

The costs for soil investigation are based on the estimated costs for soil investigation given the interval between which the borings were performed and the samples taken. From the reference project, for an interval of 100m between borings the estimated costs are €150.000 per km. This leads to an estimated cost of €270.000 for soil investigation in the used case. This estimate was also used if a different interval was applied. For an interval of 50m for example, double the estimated cost for an interval of 100m was accounted for.

For the HPT-boring the estimated costs are based on the amount of borings performed. A single HPT-boring is estimated to cost €50.000. All that is still needed then are the amount of HPT-borings performed in the dike trajectory.

Appendix D: Expected project cost reduction due monitoring

To determine the expected benefit achieved by monitoring, the effect of the length of the monitoring needs to be taken into account. In this chapter the calculation steps to determine the expected project reduction due to monitoring are specified. For the calculations scenario 2 and 3 of the soil characteristics were used, which are shown in Table 40. Also shown here are the cheapest reinforcement measures when only a monitoring plan is applied, given that the needed data to verify that the seepage length is 75m is available.

Soil characteristics scenario 2	Section	d_{70} (µm)	k (m/day)	L (m)	Reinforcement measure
Two sections	1	180	10	75	20m piping berm
	2	255	5	55	19m sheet pile wall

Table 40 soil parameters for scenario 2

Using the soil characteristics for the sections of the dike trajectory, the estimated project costs can be calculated for the different project phases given that the seepage length is accurately assessed. If the seepage length is known in the initiation phase, it can be used during the entire project. So the estimated project costs can be calculated using the reinforcement measures mentioned in Table 40. Since the seepage length is not long enough to reduce the project scope, there is no difference between the initiation phase and the reconnaissance phase. If the seepage length is only accurately known in the elaboration phase, instead of a 20m piping berm a 15m sheet pile wall is the preferred alternative. This is because based on the information available in the reconnaissance phase this was determined to be the cheapest reinforcement measure. But there is still a project cost reduction since the more accurately determined longer seepage length leads to shorter dimensions of the sheet pile wall.

	Initiation phase	Reconnaissance phase	Elaboration phase
Estimated project costs (M€)	11,5	11,5	12,2
Cumulative estimated project cost reduction (M€)	2,2	2,2	1,4

Table 41 estimated project costs for different project phases

Once the estimated project cost reduction is calculated, the expected project cost can be determined by combining the estimated project cost reduction with the probability of exceedance of the threshold water level during the monitoring period. For this case the threshold value of the water level which needs to be exceeded to collect relevant data needed to determine the seepage length of the dike is 8m above N.A.P. By interpolating the project cost reduction of the project phases over the corresponding project phases an expected project cost reduction can be determined for every year of the project. And finally the investment and maintenance costs of the monitoring plan need to be included. These calculations are performed in Table 42 to 44 for starting monitoring in each of the project phases.

Year	0	1	2	3	4	5	6	7	8	9	10
Estimated project cost reduction (M€)	-	-	-	-	-	-	-	-	0,8	-	2,2
Interpolated project cost reduction(M€)	0	0	0	0	0	0	0	0,4	0,8	1,5	2,2
Probability of exceedance water level during monitoring period (-)	0	0,22	0,40	0,53	0,63	0,72	0,78	0,83	0,87	0,90	0,92
Expected project cost reduction minus investment costs (E[M€])	0	0	0	0	0	0	0	0,33	0,70	1,37	2,07
Estimated investment cost monitoring (M€)	0,11	0,13	0,14	0,15	0,16	0,17	0,19	0,2	0,21	0,22	0,23
Total expected project cost reduction (E[M€])	-0,11	-0,13	-0,14	-0,15	-0,16	-0,17	-0,19	0,13	0,49	1,15	1,84

Table 42 calculations for scenario 2-G-I

Year	0	1	2	3	4	5	6	7	8	9	10
Estimated project cost reduction (M€)	-	-	-	-	-	-	-	-	0,8	-	2,2
Interpolated project cost reduction minus (M€)	0	0	0	0	0	0	0	0,4	0,8	1,5	2,2
Probability of exceedance water level during monitoring period (-)	0	0	0	0	0	0	0	0,22	0,40	0,53	0,63
Expected project cost reduction minus investment costs (E[M€])	0	0	0	0	0	0	0	0,09	0,32	0,8	1,39
Estimated investment cost monitoring (M€)	0	0	0	0	0	0	0,11	0,13	0,14	0,15	0,16

Total expected project cost reduction (E[M€])	0	0	0	0	0	0	-0,11	-0,04	0,18	0,65	1,23

Table 43 calculations for scenario 2-G-II

Year	**0**	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
Estimated project cost reduction (M€)	-	-	-	-	-	-	-	-	-	-	1,4
Interpolated project cost reduction minus (M€)	0	0	0	0	0	0	0	0	0	1,1	2,2
Probability of exceedance water level during monitoring period (-)	0	0	0	0	0	0	0	0	0	0,22	0,40
Expected project cost reduction minus investment costs (E[M€])	0	0	0	0	0	0	0	0	0	0,24	0,88
Estimated investment cost monitoring (M€)	0	0	0	0	0	0	0	0	0,11	0,13	0,14
Total expected project cost reduction (E[M€])	0	0	0	0	0	0	0	0	-0,11	0,11	0,74

Table 44 calculations for scenario 2-G-III

Once the expected project cost reduction is calculated for the different starting points, they can be plotted for comparison. The result is shown in Figure 67. The bars represent the cumulative estimated project cost reduction, excluding the probability of collecting the required data. Each of the lines represents the total expected project cost reduction, which includes this probability.

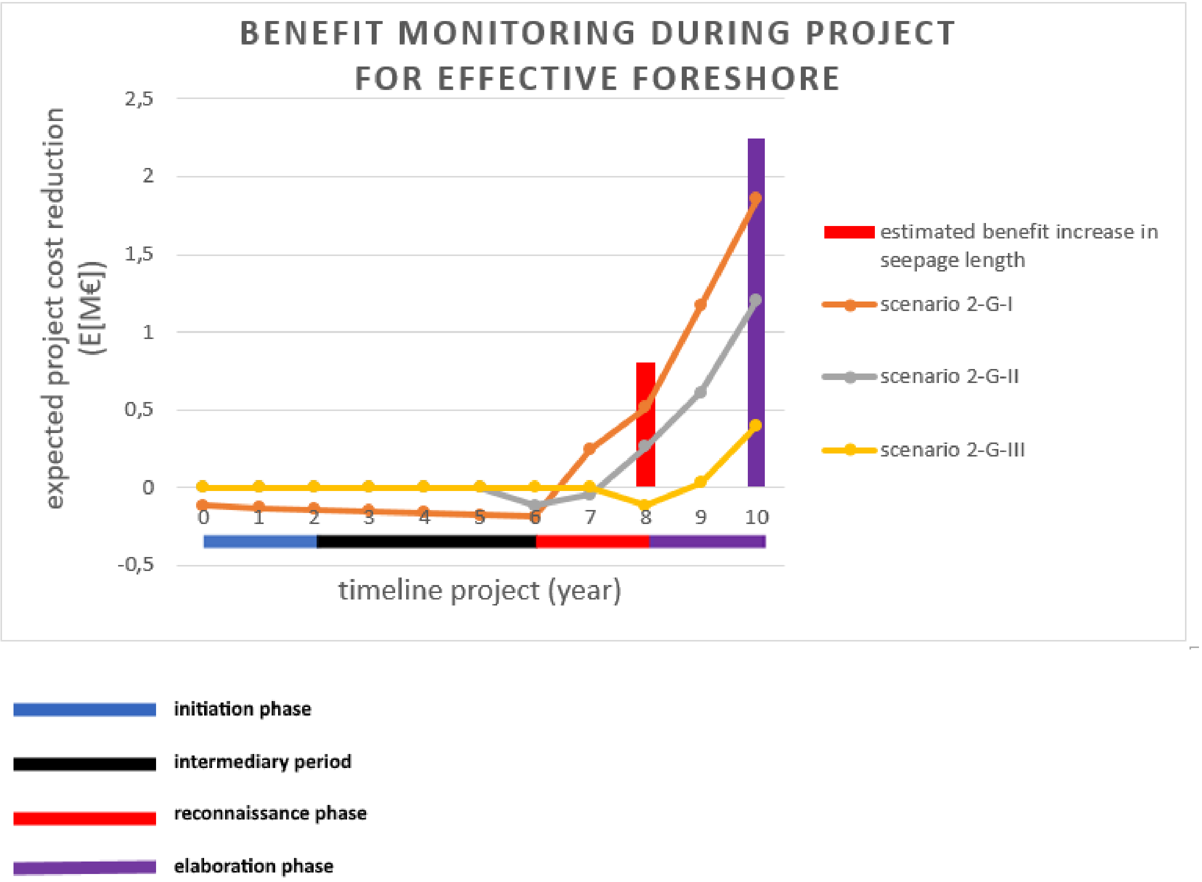


Figure 67 expected project cost reduction for monitoring in scenario 2

The same can be done for scenario 3 of the soil characteristics, which are shown in Table 45. Again the needed reinforcement measures are shown for when only a monitoring plan is implemented, provided that the seepage length is accurately assessed.

Soil characteristic scenario 3	**Sections (-)**	**d_{70} (µm)**	**K (m/day)**	**L (m)**	**Reinforcement measure**
Three sections	1	180	5	95	-
	2	255	10	75	20m piping berm
	3	255	5	55	19m sheet pile wall.

Table 45 soil characteristics for scenario 3

The same calculations are applied as were done for the soil characteristics of scenario 2. The main difference is that for this scenario the seepage length is long enough that for this section no reinforcement is needed. But if the seepage length is not accurately assessed in the initiation phase, this section is included in the project scope. A 12m sheet pile wall is then included, as this is the smallest alternative reinforcement. This same reinforcement is applied if the real seepage length is known in the elaboration phase. The results are shown in Figure 68.

	Initiation phase	Reconnaissance phase	Elaboration phase
Estimated project costs (M€)	7,6	10,5	11,1
Cumulative estimated project cost reduction (M€)	6,1	3,2	2,6

Table 46

Year	0	1	2	3	4	5	6	7	8	9	10
Estimated project cost reduction (M€)	-	-	2,9	-	-	-	-	-	3,4	-	6,1
Interpolated project cost reduction minus (M€)	0	1,4	2,9	3,0	3,1	3,1	3,2	3,3	3,4	4,7	6,1
Probability of exceedance water level during monitoring period (-)	0	0,22	0,40	0,53	0,63	0,72	0,78	0,83	0,87	0,90	0,92
Expected project cost reduction minus investment costs (E[M€])	0	0,31	1,1	1,6	2,0	2,3	2,5	2,7	3,0	4,3	5,6
Estimated investment cost monitoring (M€)	0,11	0,13	0,14	0,15	0,16	0,17	0,19	0,2	0,21	0,22	0,23
Total Expected project cost reduction (E[M€])	-0,11	0,18	0,96	1,45	1,84	2,13	2,31	2,5	2,79	4,08	5,37

Table 47 calculations for scenario 3-G-I

Year	0	1	2	3	4	5	6	7	8	9	10
Estimated project cost reduction (M€)	-	-	-	-	-	-	-	-	0,5	-	3,2
Interpolated project cost reduction minus (M€)	0	0	0	0	0	0	0	0,2	0,5	1,9	3,2
Probability of exceedance water level during monitoring period (-)	0	0	0	0	0	0	0	0,22	0,40	0,53	0,63
Expected project cost reduction minus investment costs (E[M€])	0	0	0	0	0	0	0	0,06	0,21	1,0	2,02

Estimated investment cost monitoring (M€)	0	0	0	0	0	0	0,11	0,13	0,14	0,15	0,16
Total Expected project cost reduction (E[M€])	0	0	0	0	0	0	-0,11	-0,07	0,06	0,85	1,84

Table 48 calculations for scenario 3-G-II

Year	**0**	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
Estimated project cost reduction (M€)	-	-	-	-	-	-	-	-	-	-	2,6
Interpolated project cost reduction minus (M€)	0	0	0	0	0	0	0	0	0	1,3	2,6
Probability of exceedance water level during monitoring period (-)	0	0	0	0	0	0	0	0	0	0,22	0,40
Expected project cost reduction minus investment costs (E[M€])	0	0	0	0	0	0	0	0	0	0,29	1,05
Estimated investment cost monitoring (M€)	0	0	0	0	0	0	0	0	0,11	0,13	0,14
Total Expected project cost reduction (E[M€])	0	0	0	0	0	0	0	0	-0,11	0,16	0,91

Table 49 calculations for scenario 3-G-III

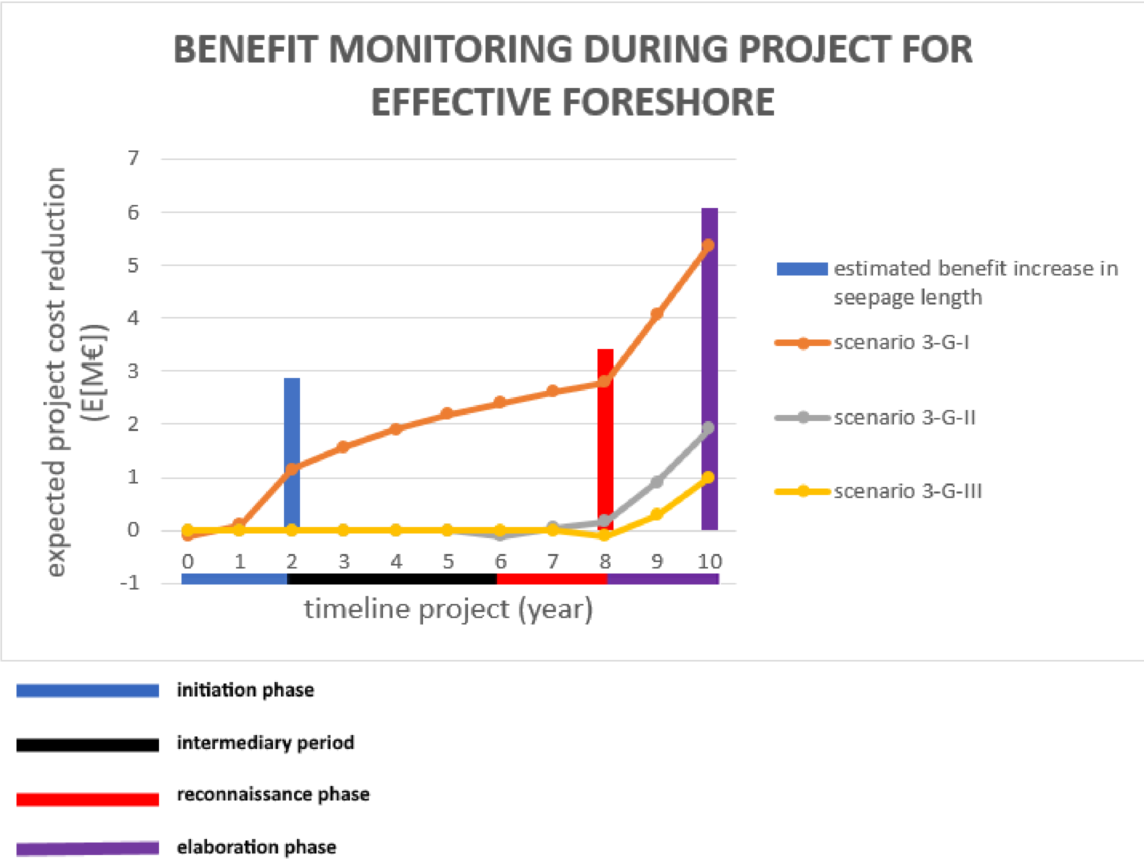


Figure 68 expected project cost reduction for monitoring in scenario 3

Appendix E Expected cost reduction scenarios

To determine the benefit of monitoring and soil investigation in the different project phases, the effect of these measures during the different project phases must be calculated. This was again done for two different sets of soil characteristics, which are shown in Table 50 and Table 55.

Soil characteristics scenario 2	**Sections (-)**	d_{70} (µm)	k (m/day)	L (m)
Length sections=900m	1	255	5	55
	2	180	10	75

Table 50 soil characteristics scenario 2

The required dike reinforcement measure and the dimensions of the measure depend on both which soil characteristics are found in the dike section, and to which level of accuracy the parameter is known. The applied monitoring and soil investigation therefore influence the used parameters. This is represented in Table 51, where the reinforcement measures for different combinations of the resulting parameters found in the dike trajectory by applying additional soil investigation and/or monitoring. If no additional soil investigation and/or monitoring are applied, the results from the soil investigation for the safety assessment are used which are described in the base scenario.

d_{70} (µm)	**K (m/day)**	**L (m)**	**Reinforcement measure**	**Length measure needed (m)**
180	10	55	Sheet pile wall	19
255	10	55	Sheet pile wall	15
180	5	55	Piping berm	18
180	10	75	Piping berm	20[1]
180	5	95	Piping berm	0[2]
255	5	55	Piping berm	8
255	10	75	Piping berm	7[1]

Table 51 reinforcement of the dike needed for different parameters of the dike section for the initiation phase

[1] Of which 20m reduction of the piping berm is conditional on the water level exceeding the threshold
[2] Of which 22m reduction of the piping berm is conditional on the water level exceeding the threshold

For a few of the dimensions of the dike reinforcement measures it is mentioned that the length of the dimension is conditional on the water level exceeding the threshold. This has to do with the nature of monitoring the hydraulic head in the aquifer to determine the seepage length. The water level needs to be large enough to determine what the hydraulic head in the aquifer over the cross-section of the dike is to determine the total seepage length. For this thesis it is assumed the water level in the river needs to be 8m above N.A.P or higher to be able to meet the right conditions to collect relevant data. The conditional part means this water level needs to be exceeded while a monitoring system is in place to correctly assess the seepage length. The 8m above N.A.P. criteria was determined because at this value the head difference at the toe of the dike between daily conditions and extreme conditions is still large enough (>1m), see Chapter 4.5.1.

The dike reinforcement measures and dimensions shown in Table 51 are the result if the parameters are known at the start of the project. Otherwise when the information is available in the project also needs to be taken into account. If the parameters were not accurate in the initiation phase, than possibly sections of the dike are included in the project scope which did not have to be. This means the sections is reinforced while it did not have to be. This impacts the scenario for which part of the dike has a seepage length of 95m. If the real seepage length is not known in the initiation phase, a sheet pile wall of 12m is used for that section as smallest alternative reinforcement.

If the real parameters are only known in the elaboration phase, than this means that the choice of the preferred alternative in the reconnaissance phase was based on the information available from the safety assessment. Based on these parameters, a sheet pile wall is the cheapest alternative. Therefore if the information is only available in the elaboration phase a sheet pile wall is applied, no matter the parameters that result from the monitoring (Table 52). Since the effect of the hydraulic conductivity of the aquifer is not included in the method of Lane (See appendix B), it has no effect on the length of the sheet pile wall.

d_{70} (µm)	**K (m/day)**	**L (m)**	**Length sheet pile wall (m)**
180	10	55	19
255	10	55	15
180	5	55	19
180	10	75	15[1]
180	5	95	12[2]
255	5	55	15
255	10	75	12[1]

Table 52 reinforcement of the dike needed for different parameters of the dike section for the elaboration phase

[1] Of which 20m reduction of the piping berm is conditional on the water level exceeding the threshold
[2] Of which 22m reduction of the piping berm is conditional on the water level exceeding the threshold

Once the required dimensions of the dike sections are known, the expected project costs can be determined. For the expected project costs the cost calculations from Appendix C are used. These give an estimate of the project costs per m^2 of the dike reinforcement measure. The results are shown in Table 53 and Table 54. Table 53 shows the results for if only one method of monitoring or soil investigation is applied in the project, while Table 54 shows the results from different methods are combined.

Monitoring and soil investigation methods	**Expected project cost section 1 (E[M€])**	**Expected project cost section 2 (E[M€])**	**Expected project cost dike trajectory (E[M€])**	**Investment costs monitoring and soil investigation (M€)**	**Total expected project cost (E[M€])**	**Total expected project cost reduction (E[%])**
Base scenario (A)						
-	-	-	13,7	0	0	0
Soil investigation probe at 100m interval (B)						
Performed in year 0	5,4	6,9	12,2	0,27	12,4	11,3
Performed in year 6	5,4	6,9	12,2	0,27	12,4	11,3
Performed in year 8	5,4	6,9	12,2	0,27	12,4	11,3

Two Hydraulic conductivity measurements (E)						
Performed in year 0	4,1	6,9	11,0	0,10	11,1	19,0
Performed in year 6	4,1	6,9	11,0	0,10	11,1	19,0
Performed in year 8	6,9	6,9	13,7	0,10	13,8	-0,7
Monitoring implemented (G)						
Started in year 0			See Appendix D	0,23		14,9
Started in year 6			See Appendix D	0,16		9,2
Started in year 8			See Appendix D	0,14		3,1

Table 53 single measure of monitoring and soil investigation applied for two sections

When only monitoring is applied, it is easier to calculate the expected project costs as all costs were dependent on the result of monitoring. But when combined with additional soil investigation and HPT-borings, not all of the expected project costs are dependent on the probability of exceedance. To calculate the part of the expected project costs due to the changes in the design caused by monitoring, Equation 32 is used. The equation calculates the conditional costs by multiplying the estimated project costs of dike reinforcement for when no change in the seepage length was found with the probability that the water level does not exceed the water level. Using this method the expected project costs can be determined when different measurement techniques are applied.

$$E[C] = C_c * (1 - p_{exc}) \quad [32]$$

Monitoring and soil investigation methods	**Expected project cost section 1 (E[M€])**	**Expected project cost section 2 (E[M€])**	**Expected project cost dike trajectory (E[M€])**	**Investment costs monitoring and soil investigation (M€)**	**Total expected project cost (E[M€])**	**Total expected project cost reduction (E[%])**
Soil investigation probe interval 100m + Two HPT borings (I)						
Performed in year 0	1,8	6,9	8,7	0,37	9,1	33,6
Performed in year 6	1,8	6,9	8,7	0,37	9,1	33,6
Performed in year 8	5,4	6,9	12,3	0,37	12,7	11,3
Monitoring seepage length + soil investigation d_{70} probe interval 100m (J)						
Performed/started in year 0	5,4	5,0	10,4	0,5	10,9	20,6[1]
Performed/started in year 6	5,4	5,9	11,4	0,43	11,8	14,2
Performed/started in year 8	5,4	6,3	11,8	0,41	12,1	11,7
Monitoring seepage length + two HPT probes (K)						
Performed/started in year 0	4,1	5,0	9,1	0,33	9,4	31,7
Performed/started in year 6	4,1	5,8	10,0	0,26	10,3	25,1
Performed/started in year 8	6,9	6,1	12,9	0,24	13,2	3,6

Monitoring seepage length + two HPT probes+ soil investigation d_{70} probe interval 100m (M)						
Performed/started in year 0	1,8	5,0	6,8	0,6	7,4	41,4
Performed/started in year 6	1,8	5,9	7,8	0,53	8,3	34,4
Performed/started in year 8	5,4	5,0	10,4	0,51	10,9	20,5

Table 54 multiple measures of monitoring and soil investigation applied for two sections

[1] It should be mentioned that this is the scenario which is used to calculate the estimated investment costs and the expected benefit of implementing monitoring and performing additional soil investigation. Only the estimated costs of performing HPT borings is not included here, since only the investment costs needed of the soil research for the specific parameters is shown. Therefore the expected project cost reduction in Table 54 is slightly higher.

The same calculations which were described above were performed for scenario 3, with different soil characteristics. The soil characteristic are described in Table 55, while the results can be found in Table 56 and Table 57.

Soil characteristic scenario 3	**Sections (-)**	**d_{70} (µm)**	**K (m/day)**	**L (m)**
Three sections	1	180	5	95
Length sections is 600m	2	255	10	75
	3	255	5	55

Table 55 soil characteristics scenario 3

Monitoring and soil investigation methods	**Expected project cost section 1 (E[M€])**	**Expected project cost section 2 (E[M€])**	**Expected project cost section 3 (E[M€])**	**Expected project cost dike trajectory (E[M€])**	**Investment costs monitoring and soil investigation (M€)**	**Total expected project cost (E[M€])**	**Total expected project cost reduction (E[%])**
Base scenario (A)							
-	-	-	-	13,7	0	0	0
Soil investigation probe at 100m interval (B)							
Performed in year 0	4,6	3,6	3,6	11,8	0,27	12,1	13,8
Performed in year 6	4,6	3,6	3,6	11,8	0,27	12,1	13,8
Performed in year 8	4,6	3,6	3,6	11,8	0,27	12,1	13,8
Three Hydraulic conductivity measurements (F)							
Performed in year 0	2,8	4,6	2,8	10,1	0,15	10,3	25,2
Performed in year 6	2,8	4,6	2,8	10,1	0,15	10,3	25,2
Performed in year 8	4,6	4,6	4,6	13,7	0,15	13,9	-1,1
Monitoring implemented (G)							
Started in year 0				See Appendix D	0,23		40,7

Started in year 6				See Appendix D	0,16		14,7
Started in year 8				See Appendix D	0,14		7,6

Table 56 single measure of monitoring and soil investigation applied for three sections

Monitoring an soil investigation methods	**Expected project cost section 1 (E[M€])**	**Expected project cost section 2 (E[M€])**	**Expected project cost section 3 (E[M€])**	**Expected project cost dike trajectory (E[M€])**	**Investment costs monitoring and soil investigation (M€)**	**Total expected project cost (E[M€])**	**Total expected project cost reduction (E[%])**
Soil investigation probe interval 100m + Three HPT borings (I)							
Performed in year 0	2,8	3,6	1,2	7,6	0,42	8,0	41,5
Performed in year 6	2,8	3,6	1,2	7,6	0,42	8,0	41,5
Performed in year 8	4,6	3,6	3,6	11,8	0,42	12,1	13,8
Monitoring seepage length + soil investigation d_{70} probe interval 100m (J)							
Performed/started in year 0	0,5	1,3	3,6	5,4	0,5	5,9	60,5
Performed/started in year 6	3,5	2,2	3,6	9,4	0,43	9,8	28,7
Performed/started in year 8	3,9	3,3	3,6	10,8	0,41	11,2	20,7
Monitoring seepage length + three HPT probes (L)							
Performed/started in year 0	2,2	3,3	2,8	6,3	0,38	6,7	51,2
Performed/started in year 6	2,8	2,8	3,8	9,3	0,31	9,7	29,1
Performed/started in year 8	3,5	4,0	4,6	12,1	0,29	12,4	9,5
Monitoring seepage length + three HPT probes soil investigation d_{70} probe interval 100m (N)							
Performed/started in year 0	0,2	1,3	1,2	2,8	0,65	3,4	79,8
Performed/started in year 6	2,2	2,8	1,2	6,2	0,58	6,9	50,5
Performed/started in year 8	4,6	3,6	3,6	11,8	0,56	12,1	17,0

Table 57 multiple measures of monitoring and soil investigation applied for three sections

Appendix F: Estimation probabilities scenarios

To determine which scenario for the soil characteristics leads to the highest expected benefit, the probability of occurrence of the scenarios has been roughly estimated. For the estimation of the probability of occurrence of the scenarios the probability of exceedance of the corresponding parameters were used. The available data on the parameters of points in the dike trajectory are shown in Table 58.

K (m/day)	d_{70} (µm)	L (m)
5	255	95
5	255	95
10	360	95
20	180	95
	255	95
	255	95
	255	75
	360	75
	255	75
		75
		55
		55
		55
		55
		55
		55
		55
		75
		75
		75

Table 58 available data on the parameters

The available data of the hydraulic conductivity consists of three soil samples and interpretation by REGIS II available on Dinoloket (Appendix G). The hydraulic conductivity of 20m/day is the hydraulic conductivity of an upper layer found in the HPT boring provided by Fugro (Appendix H). Note that the Bulk permeability of the layer is likely lower than 20m/day, as the sand layer below has a lower hydraulic conductivity. The hydraulic conductivity of 2,5m/day shown in Appendix G was found further along the dike than the trajectory which is analysed, and is therefore not included here. The d_{70} values of the upper layer of the aquifer were determined from soil samples retrieved from Dinoloket. The values used are the mean values of the range of d_{70} for the found lithology classes of the upper sand layer in the aquifer. For the data on the seepage length an estimate was made using Google maps. At an interval of approximately 100m it was identified if obstacles were present in the foreshore which could mean a reduction in effective foreshore length. The distance to the obstacles was then estimated using visual inspection. If no obstacles were present in the foreshore, a seepage length of maximum 95m was used.

	K (m/day)	d_{70} (µm)	L (m)
Mean	10	281,25	19
Standard deviation	7,1	54,55	16,1

Table 59 mean and standard deviation of the parameters

Based on the available data, the mean and standard deviation can be determined (Table 59). Using these the probability of exceedance of the parameters can be calculated. The distribution of the parameters is assumed to be normal. These distributions are then used to estimate the probability of occurrence for the scenarios (Table 61).

Soil characteristics dike trajectory	**Sections (-)**	**d_{70} (µm)**	**K (m/day)**	**L (m)**
1. **Base scenario**	1	180	10	55
2. **Two sections**	2	255	5	55
		180	10	75
3. **Three sections**	3	180	5	95
		255	10	75
		255	5	55

Table 60 parameters used for the estimation probability of occurrence scenarios

The probability of occurrence for the scenarios is determined for all different combinations of parameters provided in Table 60. The parameters mentioned are the parameters which differ from the base scenario. For example, d_{70} two samples means there is a section present for which d_{70}=180 µm and a section for which d_{70}=255µm. The sections are equidistant of each other. The other two parameters are the same for the entire trajectory, with the values specified in the base scenario. The relation between the parameters are treated as independent. This is most likely not true for the d_{70} and K (Stoop, 2018), but there is too few measurements available to determine the correlation. So for the estimation they are treated as independent.

Value parameter	**Probability of exceedance (-)**
d_{70} =180 µm	0,96
d_{70} =255 µm	0,68
K=10m/day	0,5
K=5m/day	0,23
L=55m	0,88
L=75m	0,48

The probability of occurrence is estimated using the probability of exceedance for the parameter different to the parameter in the base scenario. If more than one parameter is different in the scenario, equation 33 is used (Jonkman, 2018). The same equation is applied when the parameter exceeds the given value for multiple sections, so for example when two sections of the scenario have d_{70}=255 µm.

$$p_{scenario} = \prod_{i=1}^{n} p_{exc,i} \quad [33]$$

The equation used to calculate the expected reduction in project costs including the estimated probability of occurrence of the scenario is shown below (Equation 34):

$$R_{scenario} = (p_{scenario} * R) - I \quad [34]$$

For which:

$R_{scenario}$=Expected reduction in project costs including estimated probability of occurrence scenario
$p_{scenario}$= estimated probability of occurrence scenario
R= Expected reduction in project costs if monitoring and/or soil investigation is started in initiation phase
I= estimated investment costs monitoring and/or soil investigation

	Estimated probability of occurrence scenario (-)	**Expected reduction in project costs if monitoring and/or soil investigation is started in initiation phase (E[%])**	**Estimated investment costs monitoring/ soil investigation (%)**	**Expected reduction project costs including estimated probability of occurrence scenario (E[%])**
d_{70}=180µm	0,68	11,3	1,97	5,7
d_{70}=255µm				
d_{70}=180µm	0,47	13,8	1,97	4,5
d_{70}=255µm				
d_{70}=255µm				
L_{eff}=75m	0,48	14,9	1,68	5,40
L_{eff}=55m				
d_{70}=180µm L_{eff}=75m	0,33	20,6	3,65	3,2[1]
d_{70}=255µm L_{eff}=55m				
K=10m/day	0,24	19,0	0,73	3,8
K=5m/day				
d_{70}=180µm K=10m/day	0,16	33,6	2,33	3,0
d_{70}=255µm K=5m/day				
K=10m/day L_{eff}=75m	0,11	31,7	2,41	1,2
K=5m/day L_{eff}=55m				
L_{eff}=95m	0,10	40,7	1,68	2,2
L_{eff}=75m				
L_{eff}=55m				
d_{70}=180µm K=10m/day L_{eff}=75m	0,08	41,4	4,38	-1,1
d_{70}=255µm K=5m/day L_{eff}=55m				
K=5m/day	0,06	25,2	1,09	0,4

K=10m/day				
K=5m/day				
d_{70}=180µm K=5m/day	$2,7*10^{-2}$	41,5	2,70	-1,5
d_{70}=255µm K=10m/day				
d_{70}=255µm K=5m/day				
d_{70}=180µm L_{eff}=55m	$4,5*10^{-2}$	60,5	3,65	-1,2
d_{70}=255µm L_{eff}=75m				
d_{70}=255µm L_{eff}=95m				
K=5m/day L_{eff}=55m	$5,5*10^{-3}$	51,2	2,77	-2,5
K=10m/day L_{eff}=75m				
K=5m/day L_{eff}=75m				
d_{70}=180µm K=5m/day L_{eff}=95m	$2,6*10^{-3}$	79,8	4,74	-4,5
d_{70}=255µm K=10m/day L_{eff}=75m				
d_{70}=255µm K=5m/day L_{eff}=55m				

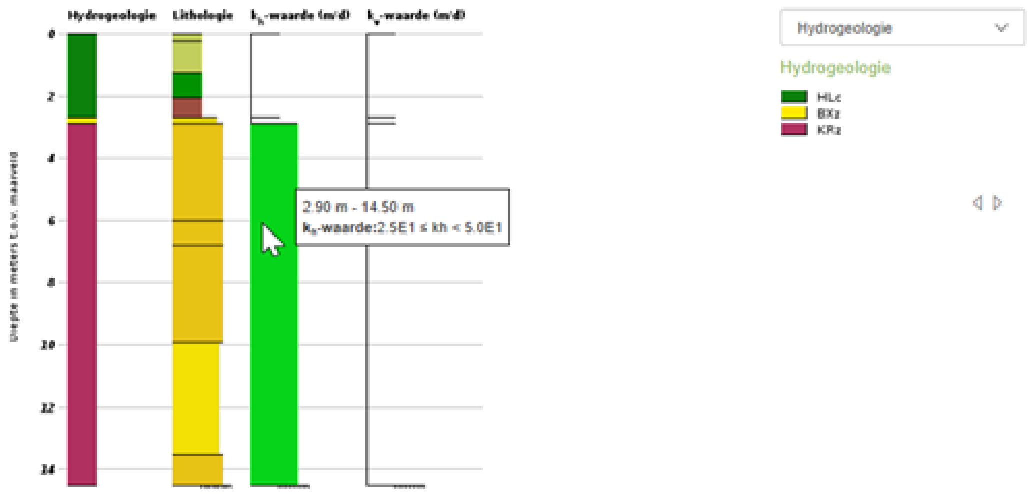
Table 61 expected reduction project costs including probability of occurrence scenarios for monitoring/ soil investigation in initiation phase

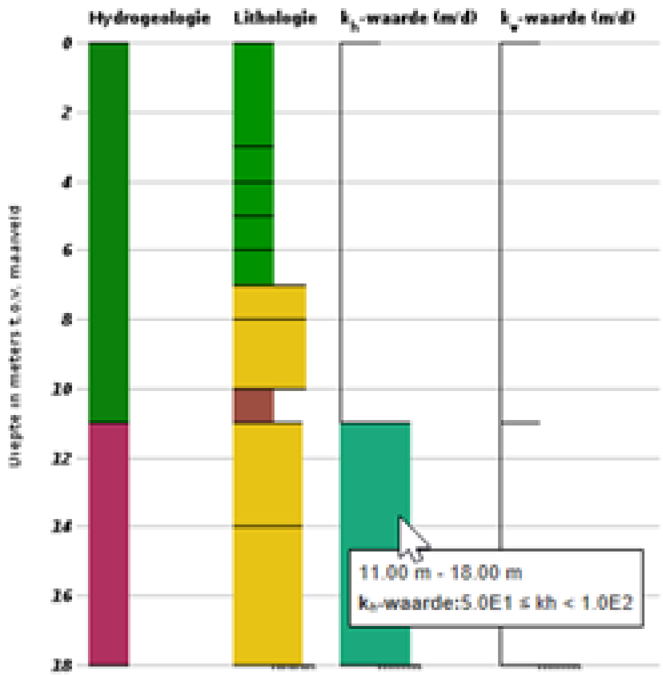
Table 61 contains a few scenarios for which the expected reduction in project costs including the probability of the scenarios is negative. This is caused when the expected reduction is lower than the estimated investment costs. The required investment to achieve the project cost reduction is subtracted to determine the total expected project cost reduction. This leads to a negative cost reduction when the expected reduction is lower than the estimated costs.

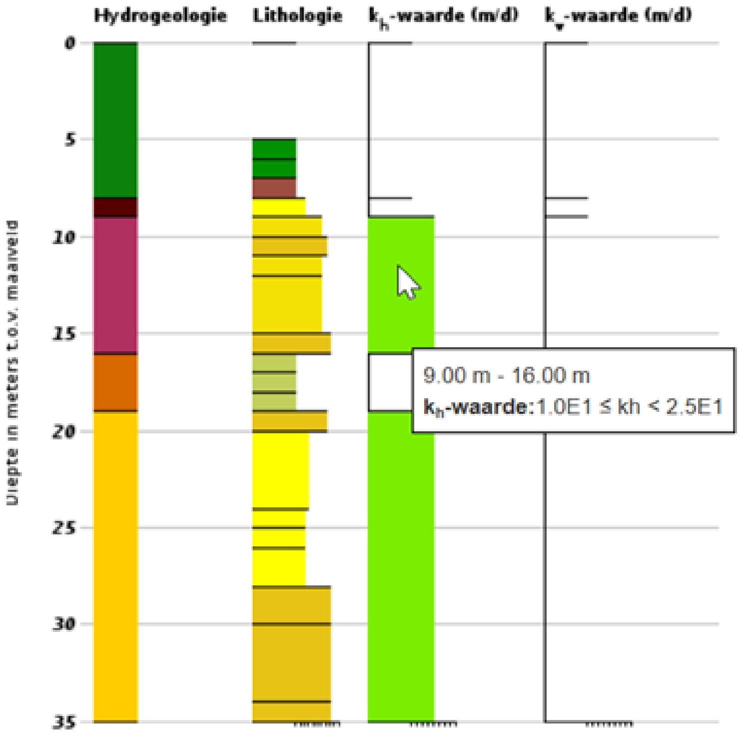
[1] The scenario for which the combination of exceeded parameters lead to the highest expected reduction ([1]) was used to determine the estimated costs and expected benefits of monitoring and additional soil investigation. This means the scenarios for which only one parameter exceeded the value of the representative cross-section were not included. This is because the parameters were assumed to be independent. The probability of both exceeding the value in a section is equal to the product of the probabilities of exceedances, which leads to a low expected benefit. But the available data has shown his does occur for sections of the dike, therefore these scenarios were used. This led to the scenario for which the values for the seepage length and the d_{70} to be stronger than first thought but the hydraulic conductivity being constant the same to be the used scenario.

Appendix G: Soil composition Dinoloket

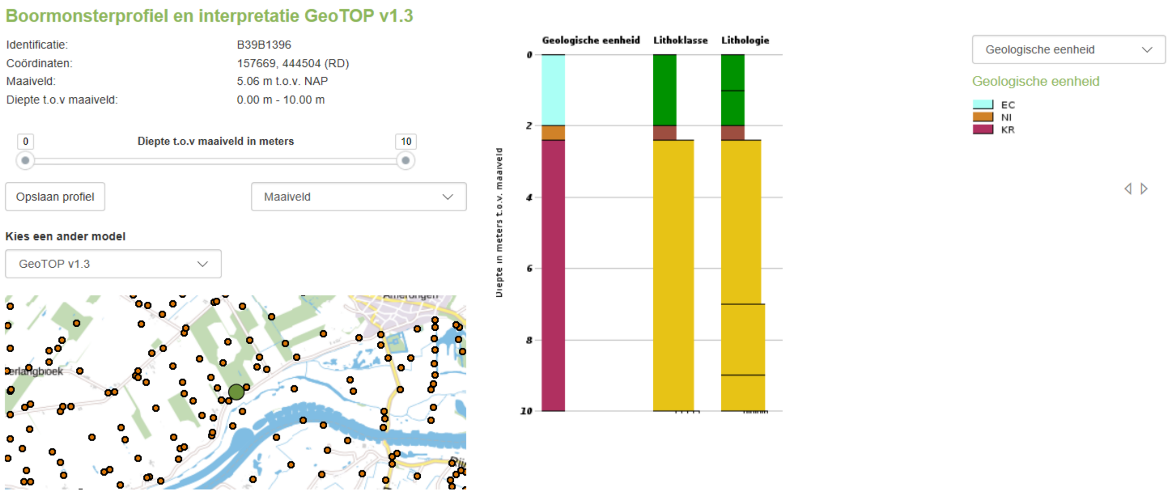
Horizontal hydraulic conductivity:







Soil Lithology:



Boormonsterprofiel en interpretatie GeoTOP v1.3

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Diepte t.o.v maaiveld:	0.00 m - 10.00 m

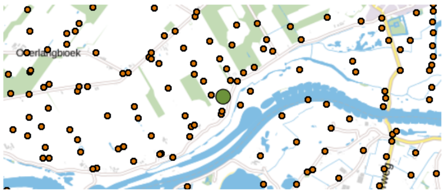
0 Diepte t.o.v maaiveld in meters 10

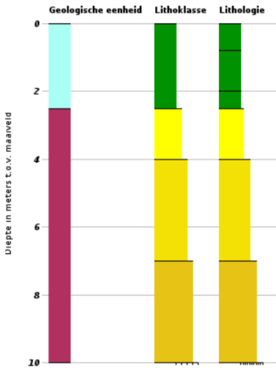
Opslaan profiel

Maaiveld

Kies een ander model

GeoTOP v1.3





Geologische eenheid

Geologische eenheid

EC
KR

Boormonsterprofiel en interpretatie GeoTOP v1.3

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Diepte t.o.v maaiveld:	0.00 m - 1.80 m

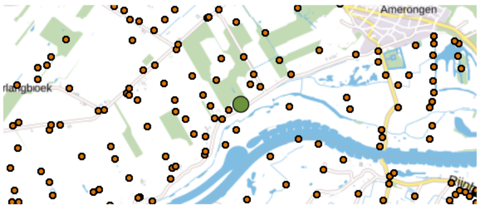
0 Diepte t.o.v maaiveld in meters 1.8

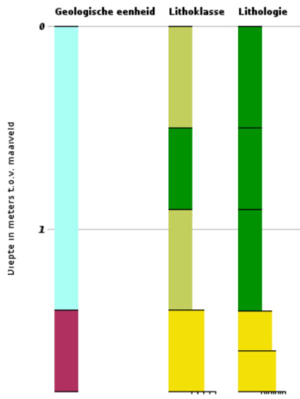
Opslaan profiel

Maaiveld

Kies een ander model

GeoTOP v1.3





Geologische eenheid

Geologische eenheid

EC
KR

Appendix H: HPT results dike trajectory Amerongen by I&B