Investigation in the Effects of Piping on the Safety of a Levee Including a Structure

Bachelor Thesis

Source figure: Peter van Aalst

UNIVERSITY OF TWENTE.

hoogheemraadschap Hollands Noorderkwartier

JANUARY 27, 2022

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Bachelor's Program Civil Engineering and Management

Colophon

Title	Investigation in the Effects of Piping on the Safety of a Levee including a Structure
Version	Final
Research period	November 2021 to January 2022
Date of publication	27-01-2022

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Preface

Before you lies my thesis 'Investigation in the Effects of Piping on the Safety of a Levee including a Structure'. This thesis was written as part of my graduation from the Bachelor's programme in Civil Engineering at the University of Twente and commissioned by the Hoogheemraadschap Hollands Noorderkwartier (HHNK). From November 2021 to January 2022, I worked on the research and writing of the thesis. I have learned a lot in this short period of only 10 weeks. I learned what it is like to carry out an investigation myself and I got a glimpse of the daily work of a water board. Despite the fact that I had to carry out the research mainly from home due to Corona, I still got a good picture of the work at the HHNK during the online meetings with my supervisors. It was nice that not only I but also the people I talked to during my research became enthusiastic about this topic.

I would like to thank my technical supervisors at HHNK, Niels Tenhage and Sarah Wiggers, for all their help in carrying out this study and for giving me an educational insight into the work of a waterboard. I would also like to thank Peter van den Horst for the process guidance and of course the opportunity to carry out my assignment within the High Water Protection Department of the HHNK. Thirdly, I would like to thank Huub de Bruijn and Helle Larsen of Deltares for sharing their specific knowledge on the piping failure mechanism. I would also like to thanks my internal supervisor Jord Warmink (University of Twente) for the helpful insights and feedback. Lastly, I would like to thank everybody who helped me by sharing their knowledge and information.

Ivan Leegwater

Enschede, January 27, 2022

Abstract

English

This study investigates the effect of a structure on the piping failure mechanism at a primary flood defence. In the Netherlands the primary flood defences are periodically assessed according to the Statutory Assessment Instrument (WBI). Within the WBI, there are several tests that a flood defence must satisfy for the various failure mechanisms. The available testing methods are limited in the case of a Non Water retaining Object (NWO) near or within a flood defence. As a result, a NWO is often not properly included in the assessment process. Little research has been done into the possible impact on the probability of piping when a structure is located near a water barrier. Therefore, this study focuses on developing a flow chart that describes the approach to a safety assessment for piping of a flood defence with a structure.

The study is divided into two parts. In the first part, the flow chart is developed. To do this, there is first looked at which characteristic features of a structure can influence the piping failure mechanism. After identifying these characteristics there is looked at the positive or negative ways in which these characteristics influence the sub-mechanisms within the piping process. Once an overview of the characteristics and their influences is made, criteria are established to determine whether a certain effect occurs in the presence of a structure and whether this has consequences for the failure probability calculations. Finally, the criteria are placed within the flow chart of piping in such a way that based on this flow chart it can be determined whether additional calculations are necessary at the location of the structure.

In the second part, the developed flow chart is applied to a case study of the flood defence around Den Helder. In the latest assessment by the managing water board Hoogheemraadschap Hollands Noorderkwartier, this barrier was partially rejected and the NWOs present were not included in the calculations. In order to receive subsidy from the Flood Protection Programme (HWBP) for reinforcing the dike, it is important that the NWOs are also included in the assessment. The case study therefore first identifies potential structures expected to influence piping. Additional calculations are then performed for some of these structures in order to determine their impact on the probability of failure. Finally, the structures are also assessed using the flow chart developed in part 1, to determine whether the flow chart leads to a logical assessment.

The research shows that the influences at a structure come from parts located in the subsoil. The most important characteristics of a structure that influence piping are the type of foundation and the possible presence of substructure below the structure. A pile foundation can cause the seepage path to move to the structure if a deep clay layer is perforated. In addition, the vertical permeability and therefore the probability of pipe growth increases here. A shallow foundation increases the soil pressure and thus reduces the risk of piping. In the presence of a substructure, it is possible that the barrier effect will occur. This effect occurs when the phreatic groundwater flow is hindered and affects the hydraulic head over the flood defence. Therefore, this effect can have a positive and negative influence on the probability of piping. The possible effects of a crawl space under a structure have also been studied but the exact influence on piping is still uncertain.

The flow chart developed in this study should make it easier to include the structures present in the assessment, during future inspections of the primary flood defences. By following the flow chart, it can quickly be determined whether a structure has characteristics that influence the risk of piping. It is also recommended which additional calculations can be made when a structure has such an influential characteristic. All this leads to a more complete and thus more reliable test assessment of the water barrier.

Dutch

Deze studie onderzoekt het effect van een bouwwerk op het piping faalmechanisme bij een primaire waterkering. In Nederland worden de primaire waterkeringen periodiek beoordeeld volgens het Wettelijk Beoordelinsinstrumentarium (WBI). Er zijn binnen het WBI verschillende toetsen waar een waterkering aan moet voldoen voor de verschillende faalmechanismen. Wanneer er sprake is van een niet waterkerend object (NWO) bij een waterkering zijn de beschikbare toets methoden echter beperkt. Dit heeft tot gevolg dat een NWO vaak niet goed wordt meegenomen in het beoordelingsproces. Er is nog weinig onderzoek gedaan naar de mogelijke impact op de kans op piping wanneer er zich een bouwwerk bij of in een waterkering bevindt. Daarom focust deze studie zich op het ontwikkelen van een flow chart, die de aanpak van een veiligheidsbeoordeling voor piping van een waterkering met een bouwwerk beschrijft.

Het onderzoek is opgedeeld in twee delen. In het eerste deel wordt de flow chart ontwikkeld. Hiervoor wordt eerst gekeken welke karakteristieke kenmerken van een bouwwerk invloed kunnen uitoefenen op het faalmechanisme piping. Na het identificeren van deze kenmerken wordt er gekeken op welke positieve of negatieve manieren deze kenmerken de sub-mechanismen binnen het piping proces precies beïnvloeden. Als er een overzicht is van de kenmerken en hun invloeden worden er criteria opgesteld, waarmee kan worden vastgesteld of een bepaald effect optreedt in de aanwezigheid van een bouwwerk, en of daar consequenties voor de faalkansberekeningen aan verbonden zijn. Tot slot worden de criteria binnen de flow chart van piping geplaatst waardoor aan de hand van deze flow chart kan worden bepaalt of aanvullende berekeningen ter plaatse van de NWO nodig zijn.

In het tweede deel wordt de ontwikkelde flow chart toegepast op een case study bij de waterkering rond Den Helder. In de laatste keuringsronde door het beherende waterschap Hoogheemraadschap Hollands Noorderkwartier is deze deels afgekeurd en zijn de aanwezige NWO's niet meegenomen in de berekeningen. Om subsidie vanuit het Hoogwaterbeschermingsprogramma te kunnen ontvangen voor het verstevigen van de dijk is het van belang dat ook de NWO's worden meegenomen in de toetsing. In de case study worden daarom eerst potentiële bouwwerken geïdentificeerd waarvan wordt verwacht dat ze van invloed kunnen zijn op piping. Vervolgens worden voor een deel van deze bouwwerken aanvullende berekeningen gedaan om hun impact op de faalkans te bepalen. Uiteindelijk worden de bouwwerken ook nog beoordeeld met de in deel 1 ontwikkelde flow chart, om te bepalen of de flow chart tot een logisch oordeel leidt.

Het onderzoek laat zien dat de invloeden bij een bouwwerk komen van delen die zich in de ondergrond bevinden. De belangrijkste karakteristieken van een bouwwerk die van invloed zijn op piping zijn het type fundering en de aanwezigheid van onderbouw bij het bouwwerk. Een paalfundering kan er bij perforatie van een diepe kleilaag voor zorgen dat de kwelweg zich verplaatst naar het bouwwerk. Daarnaast neemt de verticale doorlatendheid en daarmee de kans op het groeien van een pipe hier op. Een staalfundering zorgt voor een toename in de bodemdruk en daarmee voor een afname van de kans op piping. Bij de aanwezigheid van onderbouw is het mogelijk dat het barrière effect optreedt. Dit effect ontstaat bij de opstuwing van freatisch grondwater en heeft invloed op het hydraulisch verval over de waterkering. Daardoor kan dit effect positieve en negatieve invloed hebben op de kans op piping. De mogelijke effecten van een kruipruimte onder een structure zijn ook bestudeerd maar de precieze invloed hiervan op piping is nog onzeker.

Met de in dit onderzoek ontwikkelde flow chart moet het tijdens toekomstige keuringen van de primaire waterkeringen gemakkelijker worden om de aanwezige bouwwerken mee te nemen in de beoordeling. Door het volgen van het flow chart kan snel worden bepaald of een bouwwerk kenmerken heeft die van invloed zijn op de kans op piping. Daarnaast wordt er aanbevolen welke aanvullende berekeningen kunnen worden gedaan wanneer een bouwwerk een dergelijk invloedrijk kenmerk heeft. Dit alles leidt tot een vollediger en daarmee betrouwbaarder toetsoordeel van de waterkering.

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List of Abbreviations

CPT	Cone penetration test
HHNK	Hoogheemraadschap Hollands Noorderkwartier
HWBP	Hoogwaterbeschermingsprogramma
NWO	Niet Waterkerend Object
NWObe	Niet Waterkerend Object bebouwing
RWS	Rijkswaterstaat
STPH	The failure mechanism which focuses on piping at a levee
WBI	Wettelijk Beoordelingsinstrumentarium
WBN	Waterstand bij norm

1. Introduction

1.1. Problem context

1.1.1. Historical background

Nederland is known for its long-standing struggle with water. Over the centuries, the Dutch have become accustomed to living with water and, thanks to the Water Boards, the Dutch no longer notice much of the threat that lurks in their daily lives. It is thanks to these water authorities that the Dutch are always protected from the water by good management of the flood defences.

There have been many floods in the history of the Netherlands. One of the most decisive for water policy was the flood disaster of 1953. The flood disaster caused a total of 1,826 deaths and a loss of €5.4 billion (Rijkswaterstaat, 2021). After this flood disaster, it became even clearer how important it is to protect the country properly against floods. Stricter safety standards were implemented for primary water defences. It was the reason for setting up the Delta Committee, which advised the responsible minister on measures to be taken to prevent another flood disaster. In 2006, a start was made with the policy programme Flood risk management 21st century, which was continued in the Delta Programme Safety. Subsequently, new standards were incorporated into the Water Act in 2017.

In the Netherlands, the 21 water boards are the public bodies that have the task of regulating water management in certain regions of the country. The management areas are partly determined by municipal or provincial boundaries, but mainly by river basins or drainage basins in a certain region. In the area of Noord Holland above Amsterdam the responsible water board is the Hoogheemraadschap Hollands Noorderkwartier (hereafter HHNK). The HHNK is part of the so-called high water protection programme (hereafter HWBP). It is expected that climate change will increase water levels in the future, which will place a heavier burden on the dikes, which will therefore have to be made more robust. The HWBP is the largest dike reinforcement program since the Delta Works. The objective of the HWBP is to have all major Dutch flood defences meet a new, stricter safety standard by 2050.

1.1.2. WBI Assessment 2017

In the Netherlands, various standards have been laid in place for dike sections to ensure safety. These standards are in place by law in the so-called Water Act. In 2017, the new Water Act came into force and the Statutory Assessment Instrument (WBI 2017 or WBI) contains the regulations for assessing the safety of primary water defences. The WBI is used to assess the extent to which a water defence meets the standard. The reporting of the assessment takes place periodically so that one speaks of an assessment period. For each assessment period, the WBI is updated, so that there are successive WBI editions (Waal, 2016).

The updated standards are aimed at controlling the risk of flooding at a politically and socially acceptable level of risk. The standards are expressed in terms of a maximum permissible probability of flooding, the probability of a breach of a section that leads to actual flooding of the area behind it. Figure 1 shows a schematic representation of this risk approach. In 2017, the administrators of the primary flood defences started the assessment. If an assessment shows that a dike trajectory does not meet the signalling value, a water board can submit projects to the HWBP and subsequently apply for subsidy to reinforce the dikes. The prioritisation and programming of the subsidy for reinforcement measures takes place in the HWBP (STOWA, 2021).

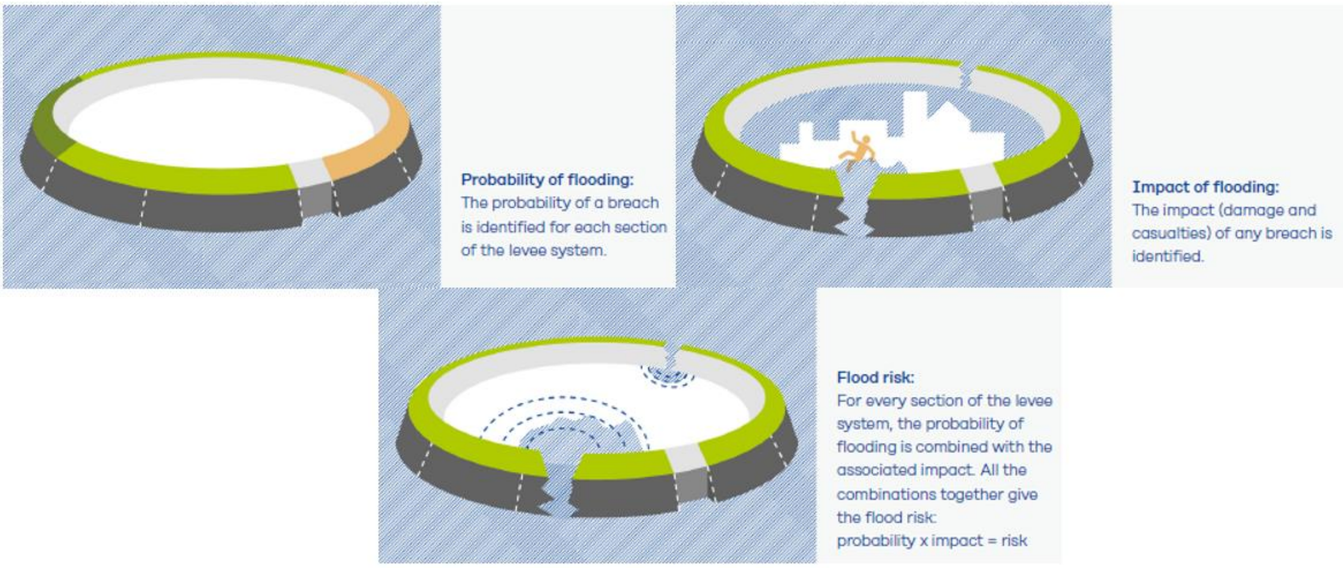

Figure 1: Schematic representation of risk approach (Source: (ENW, 2017))

The goal is to have a first national safety overview available in 2023. The aim for the report is to provide a more accurate safety overview in 2035, with the majority of primary flood defences meeting the standard by 2047. The target is for all primary flood defences to meet the standard by 2050. The starting point of flood risk management policy is that prevention remains the key factor in achieving the envisaged level of protection. To this end, dike reinforcements are usually carried out. In some cases, a decision may be made to widen the river or take measures to limit the consequences as part of a smart combination (STOWA, 2021).

1.1.3. Non Water Retaining Structures

The dikes are assessed for the following failure mechanisms: macro stability inland, macro stability outward, piping, micro stability, stability of asphalt revetment, stability of grass revetment, stability of stone revetment, dune defences, engineering structures and foreland. There are different assessment tracks for all failure mechanisms. A test track is the way in which a mechanism or part of the water barrier is assessed. Structures at a primary flood defence are also part of these test tracks.

A structure on, in or near a primary flood defence that has no direct contribution to the water safety function of the dike is called a Niet Waterkerend Object (NWO), or in English: Non Water Damming Structure. Building in and on flood defences is as old as the defences themselves. Examples of NWOs are: vegetation, underground infrastructure and buildings or structures. Over time, the laws and regulations governing construction on flood defences have been tightened, and the management of the barrier and the enforcement and testing of safety also impose conditions that are not always compatible with construction. At present, the Dutch live in a time where they want to use their - limited - space as versatile as possible within the available preconditions. A dike in combination with the water present often also has an appealing character for the community to recreate. Multifunctional (co-)use of flood defences is therefore an important subject in the Netherlands. In the context of the multifunctional use of water barriers, building in and on the water barrier is aimed at combining water safety with other (capital-intensive) spatial functions such as housing (homes on the water barrier), work (windmills on the water barrier) and infrastructure (an underground car park) (STOWA, 2021).

In this study, the focus is on the influence of buildings or structures. Building in and on flood defences is not a reinforcement measure and has no direct water safety purpose. However, the buildings may have a water retaining function. In addition, the buildings may affect the strength or the load on the flood defence. Building on flood defences affects flood defences and therefore layer 1 of multi-layer safety: prevention.

The central knowledge gap in this research area is the method of assessment and the associated knowledge for application within the WBI. Revision of dike standards creates uncertainty for multifunctional flood defences, as these are intensive and demanding projects. The statutory testing and design instrumentation (WTI) is not applicable to multifunctional dikes, although there is scope within the WTI for applying additional rules by means of a customised test (STOWA, 2021). This knowledge gap can be reduced if water boards set up an expert group in which employees from different water boards exchange experiences with multifunctional use of flood defences and discuss when shared use is and is not possible. In this way, a uniform policy can be developed (Ellen, et al., 2011).

Within the WBI a clear and practical method for the assessment of the influence of NWOs, specifically for structures, on the probability of failure of the dike are lacking. The WBI only provides methods to calculate the failure probabilities for basic general levee embankment configurations. The Helpdeskwater has only provided a factsheet with a focus on the process of selecting NWOs that influence the failure probability (Ministerie van Infrastructuur en Waterstaat, 2021). However, limited practical methods are available. The high number of NWOs along primary flood defences makes it unrealistic to do custom calculations at all locations within the given time span for the dike assessment. Therefore, there is a demand for clear guidelines on how to assess the impact of NWOs on the probability of failure of the levees.

This study will contribute to future safety assessments, by providing an approach to a safety assessment of a primary flood defence with NWO. With regard to the assessment at NWOs, the influence of NWOs on piping is one of the least investigated topics. Therefore, in this study it was decided to focus on this topic. The approach that was developed is applied in a case study on a dike section near Den Helder.

1.2. Objective and Research Questions

This study will focus on the safety assessment of the piping failure mechanism of a primary flood defence including the influence of a NWO. This will be done by looking at the possible impacts of a NWO on the flood defence system and subsequent safety. It is important to note that only the effects of buildings and their foundations will be studied and that the results are probably not easy to extrapolate to other types of NWOs (vegetation or underground infrastructure for example). The objective of this study is:

To develop a flow chart describing how to approach the safety assessment for piping of a levee including a structure.

This flow chart as well as the decision making process on how to decide whether a favourable effect or unfavourable effect can then be included in future assessments. Therefore the following research question is posed:

How can the influential characteristics of a structure on piping be translated into a flow chart describing how to approach the safety assessment for piping of a levee including a structure?

This question is split up into three sub-questions, namely:

1. *Which characteristics of a structure have an influence on the occurrence of piping related phenomena and how do these characteristics influence the piping failure mechanism?*
2. *How can the influential characteristics of a structure be translated into criteria, that together form a flow chart, describing the approach to the safety assessment for piping of a levee including a structure?*
3. *How can the developed flow chart be applied in a case study for the assessment of a levee including a structure?*

In chapters 3 and 4, the methodology and results of research questions 1 and 2 will be discussed. Chapter 5 will then answer sub-question 3 by means of a case study.

1.3. Scope

The study entails determining which factors will influence the probability of failure of a dike, with a building as NWO, through a piping like failure mechanism which will either act favourably or unfavourably in the WBI assessment. Based on the determined factors, a diagram will be constructed with the needed combinations of characteristics of a structure and the location in order for the dike to fail due to the piping mechanism evolving from the building and its foundation. Following the construction of the diagram with possible the characteristics or paths leading to failure, the proposed steps are applied to the case study of the flood defence barrier of section 13-4. This is a primary flood defence within the management area of the HHNK that borders the North Sea on the North-West side and the Wadden Sea on the East. The study focuses on this section of dike, but the aim is that the flow chart developed can ultimately be used for other sections of dike still to be assessed.

It is not possible within the given time frame for the study to investigate the consequences for all failure mechanisms and different NWOs. It is therefore necessary to focus on one particular failure mechanism and one type of NWO. The choice which failure mechanism and which type of NWO will be investigated is briefly described. First of all, a quick scan was made of the magnitude that different failure mechanisms have on the probability of failure with the presence of a NWO In addition, comparable studies from the past in this area were examined. These studies focused mainly on the failure mechanism macro stability. The HHNK has indicated that macro stability and piping are the main failure mechanisms for the study area and associated NWOs. Given the absence of research into the influences of piping, the choice has therefore been made to focus on piping in this study. For the selection of the NWO type, the various NWOs present in the area were studied. NWOs such as trees, roads, cables and pipes, and buildings were considered. For this study, it was finally decided to look at buildings, or in other words structures. It is important to note that the focus of the study was mainly on a structure behind the dike. The influences of structures on top or in front of the dike were not so widely considered. The main reason for choosing a structure is that this type of NWO has foundations which affect the subsurface deep below ground level. It is therefore likely that this type of NWO will influence the piping failure mechanism. In addition, the case study location that will be examined has already shown that the dike body does not meet the requirements for piping. Based on these observations and consultation with experts from the HHNK, the choice was made to put the focus of this study on piping at a structure.

2. Theoretical Framework

The most important part of this study is to perform a safety assessment of the dike at various properties so that criteria for a flow chart can then be developed. In the safety assessment of the dike the HHNK has to comply with the Water Act. For the assessment of structures at dikes, there is no room within the WBI to include the unique characteristics of these objects in the assessment process. There is a general path that can be followed, but this often does not lead to an assessment for all structures. The flow chart being developed has to ensure that the unique characteristics of these structures can be easily incorporated into this assessment process. It is important that the flow chart fits within the existing assessment framework. Therefore, this chapter will elaborate on the documents supported by the WBI.

The assessment of primary flood defences is done according to the Wettelijk Instrumentarium voor de Beoordeling (WBI) which is provided by the Ministry of infrastructure and environment. The Statutory Assessment Instruments 2017 contains both the regulations for determining the hydraulic loads and the strength, as well as the procedural regulations for assessing the safety of the water defences. In order to understand the process of how the assessment is currently performed, the document: "procedure assessment safety primary flood defences" (Rijkswaterstaat, 2017) is studied.

2.1. General assessment procedure

In the WBI the safety of the dike is expressed in terms of probability of failure. Within the framework, failure is equal to loss of the dike to be able to perform its primary function, which is preventing a flood event. Before continuing to the procedure it is important to understand the difference of the concepts of the failure probability at the signalling level and lower limit of the probability. Exceeding the signalling value is usually an early signal that a barrier needs to be reinforced in the near future. This leaves sufficient time to implement reinforcement measures. The aim is to complete these measures before the lower limit is exceeded, i.e. before the barrier no longer complies with the maximum permissible flood probability or failure probability (Rijkswaterstaat, 2017).

2.1.1. Procedure

In general, the assessment consists of the following phases:

1. Preparation
   a. Gathering information about the dike section
2. Implementation
   a. General filter
   b. Testing procedure
   c. Safety assessment
3. Reporting

The three components of the implementation phase will now be discussed. The preparation and reporting phases are not discussed, as the report does not elaborate on the preparation phase and reporting according to the WBI is not important for this study.

<u>Step I: General filter</u>

The general filter consists of a set of criteria on the basis of which the manager can determine whether it is possible to arrive at an opinion on the route directly. If the criteria of the general filter are not met, the assessment continues according to the prescribed assessment procedure (Rijkswaterstaat, 2017). The purpose of this filter is to limit the amount of effort involved in the testing.

The general filter at section level selects dike sections where the probability of flooding is much higher or lower than the signalling value. For the selected diked sections, the administrator may immediately formulate a safety assessment based on the results of the VNK report and expert judgment. The general filter at section level applies per section and per test track. Before the general filter can be applied at section level, the dike trajectory must be divided into sections. If the filter at section level is applicable, a tailored test can be carried out immediately (Rijkswaterstaat, 2017).

Step II: Testing procedure

For the sections that do not meet the conditions of the general filter, the assessment is continued according to the testing procedure and the prescribed hydraulic loads and strength and safety requirements. The procedure consists of the following different types of tests (from global to detailed) (Rijkswaterstaat, 2017):

1. Simple safety assessment: this is carried out per section and per test track
    a. The simple test uses simple decision rules per section and per test track to check whether the test track is relevant. The decision rules are based on safe dimensions of (parts of) the barrier, general characteristics of the barrier which prevent a failure mechanism from occurring or simple calculation rules. If the decision rules are not met, the assessment must be continued with a detailed test per section.
2. Detailed test per section: carried out per section and per test track
    a. In the detailed test per section, the requirements for the section are derived from the statutory probability of flooding of the diked area (the standard). To do this, this probability of flooding is allocated in a prescribed manner to the failure mechanisms that are assessed in the different assessment tracks and then to the sections. In this way the maximum allowable failure probability is derived for each section per test track: the failure probability requirement per section. The test consists of assessing whether the calculated probability of failure meets the failure requirement.

After performing the simple test, the detailed test per section and/or the tailor-made test (for those sections where the filter at section level is applicable), an assessment of the section for each test track can be given, the test assessment per section. Based on the first assessment per section and per test track, the manager has to assess which follow-up steps are necessary to arrive at a safety assessment per section. There are four possible follow-up steps (Rijkswaterstaat, 2017).

1. Detailed test per track: this test is carried out for the entire section of dike where sections or tests are combined.
    a. The test assessment is made more stringent by using a probabilistic approach at track level, whereby (among other things) the failure probability budget is abandoned. The result is an assessment per track.
2. Refining the schematization of the flood defence
    a. The test assessment is made more stringent by improving (refining) the schematization of the flood defence or the schematic properties. This can be considered for both the detailed test per section and the detailed test per trajectory.
3. Tailored assessment: this assessment can be carried out per section and per test track as well as for the entire section of dike.
    a. The test assessment per section and per test track will in most cases be determined with generic and widely applicable failure definitions and models. It may happen that the test assessment does not give a reliable result for the specific local situation. The test assessment can be refined by applying site-specific knowledge or advanced analyses.
4. Stopping the assessment
    a. The administrator determines that the assessment of a route is terminated. This can be done when it meets the following criteria:

i. The administrator can substantiate that carrying out further analyses will not lead to the test assessment falling into another category.
ii. The assessment provides enough information for the report to be drawn up.
iii. If it can be demonstrated by means of a cost-benefit analysis that tightening up the test assessment is not cost-effective compared to carrying out a repair or improvement measure.

Step III: Safety assessment

If the general filter applies at section level, or if all tests have been carried out, the administrator draws up the safety assessment of the section. The safety assessment is expressed in categories. These categories can be seen in Table 1. The categories provide insight into the extent to which the route does or does not meet the standard (Rijkswaterstaat, 2017).

Table 1: Safety assessment categories (Source: (Rijkswaterstaat, 2017))

Category	**Safety category designation**
A+	Flood probability of the dike section is much lower than the signalling value. *The diked section easily meets the signalling value.*
A	Flood probability of the dike section is less than the signalling value. *Dike section meets signalling value.*
B	Flood probability of the dike section is higher than the signalling value, but lower than the lower limit. *The diked section meets the lower limit, but does not meet the signalling value.*
C	Flood probability of the diked section is higher than the signalling value and the lower limit. *The diked section does not meet the signalling value or the lower limit.*
D	Flood probability of the diked section is much higher than the signalling value and lower limit. *The diked section falls well short of the signalling value and lower limit.*

In Appendix A: Schematical overview of assessment process, all the steps in the implementation of the assessment process are shown schematically.

2.1.2. Relative failure contribution STPH

The focus in this study is on the failure mechanism piping. It is therefore important to further examine the relative contribution of the test track STPH to the total probability of failure. Therefore “Appendix III Strength and Safety” of The Statutory Assessment Instruments 2017 has been studied.

The total failure probability budget is divided over different failure mechanisms. The available failure probability space of a certain failure mechanism is indicated with a failure probability factor (ω). This factor represents the contribution of a certain failure mechanism to the total probability of failure at the cross section level. The sum of all failure contributions is equal to 1. Table 2 shows this failure probability factor for the different test tracks (Rijkswaterstaat, 2017).

Table 2: Failure probability factor (ω) for different failure mechanisms for detailed test per section (Source: (Rijkswaterstaat, 2017))

Failure mechanism	**Dunes**	**Levees**
Heigh of hydraulic structure (HTKW) or grass cover erosion crest and inner slope (GEKB)	0	0.24
Piping (STPH)	0	0.24
Macrostability inward (STBI)	0	0.04
Grass erosion outer slope (GEBU)	0	0.05
Other revetments outer slope	0	0.05
Reliability closing the hydraulic structure (BSKW)	0	0.04
Piping at hydraulic structure (PKW)	0	0.02
Strength and stability hydraulic structure (STKWp)	0	0.02
Dune erosion (DA)	0.7	0
Other failure mechanisms	0.3	0.3

The failure probability factor for piping in a dike is 0.24. The failure requirement for the trajectory is equal to ωP_{eis} where (Rijkswaterstaat, 2017):

- P_{eis} [1/year] lower limit of the dike section
- ω [-] failure probability factor for the relevant test track, prescribed for the detailed test per section

The failure requirement per section that is set for a test track is derived as follows (Rijkswaterstaat, 2017):

$$P_{eis;dsn} = \frac{\omega P_{eis}}{N_{dsn}} \quad (1)$$

Where:

- $P_{eis;dsn}$ [1/year] failure requirement per section or hydraulic structure
- N_{dsn} [-] Length-effect factor for section or hydraulic structure

The way in which the failure probability and the length effect must be included in the analysis differs per failure mechanism. If it is likely that an important uncertain parameter is likely to change substantially from point to point, the length effect factor becomes higher. For the STPH test track the lower limit for the section of dike is used to derive the hydraulic loads. The failure probability space and the length effect for the relevant test track are taken into account in the calculation rules for determining the probability of failure (Rijkswaterstaat, 2017).

2.2. Detailed test for piping failure mechanism

Now that it is clear how the general assessment procedure works and what the contribution of the STPH test track is, the specific characteristics of this test track can be discussed further. In the detailed test per section, failure due to piping is defined as exceeding the critical head whereby the progressive erosion process no longer reaches equilibrium. In the detailed test, as shown in Figure 2, the probability of occurrence is determined in which the following three sub mechanisms play a role:

- Hydraulic fracture
- Heave
- Backward erosion

In the detailed test per section, the probability of failure due to piping is determined by taking the smallest of the probability of failure due to one of these three sub mechanisms (Rijkswaterstaat, 2017).

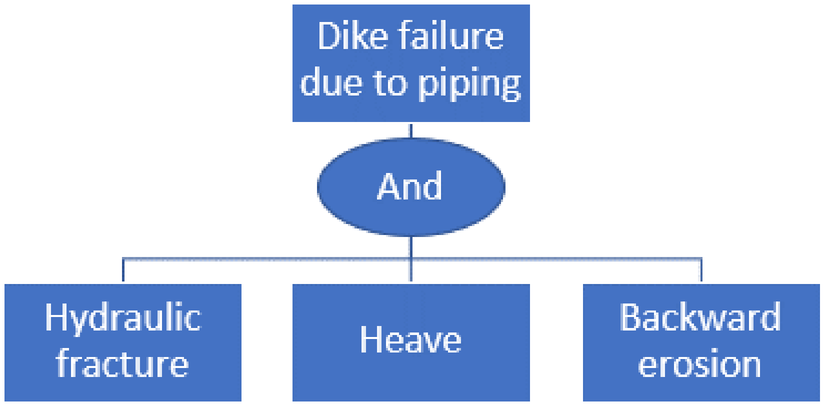

Figure 2: Assessment tree piping (Source: (Rijkswaterstaat, 2017))

Sub mechanism hydraulic fracture

The first condition for piping to occur is the hydraulic fracture of the top cover layer. First the increasing water pressure causes uplift of the covering sand layer. Hydraulic fracture occurs when the water pressure in the sand layer exceeds the weight of the top layer. The check for hydraulic fracture is based on the vertical balance of the top layer behind the dike. For hydraulic fracture, the stability factor related to hydraulic fracture must be determined. The stability factor is the quotient of the critical height difference over the covering layer and the occurring head difference (Rijkswaterstaat, 2017). In Appendix B.1. Stability factor hydraulic fracture, the calculations for the determination of the stability factor for hydraulic fracture can be found.

Sub mechanism heave

The second condition for the occurrence of piping is if the vertical flow in the hydraulic fracture channel is large enough that the sand grains are carried away from the aquifer to ground level. The stability factor, which determines whether or not the sub-mechanism heave can occur, is the quotient of the critical gradient and the occurring gradient in the hydraulic fracture channel (Rijkswaterstaat, 2017). In Appendix B.2. Stability factor heave, the calculations for the determination of the stability factor for heave can be found.

Sub mechanism backward erosion

The third condition for piping to occur is the occurrence of backward erosion. Backward erosion is an erosion process that creates a pipe under the dike. The erosion process starts at the exit point. Which is usually the point where the cover layer is hydraulicly fractured or where the vertical seepage channels are developed due to the high artesian pressure. The stability factor, which determines whether or not the sub-mechanism of retrograde erosion can occur, is the quotient of the critical hydraulic gradient and the occurring hydraulic gradient over the water barrier (Rijkswaterstaat, 2017). In Appendix B.3. Stability factor backward erosion, the calculation for the determination of the stability factor for backward erosion can be found.

2.3. Assessment of NWOs

2.3.1. General procedure

The assessment of NWOs is also briefly discussed in the procedure provided by Rijkswaterstaat. The assessment of a NWO follows the test track of indirect mechanisms. An indirect mechanism is a mechanism that does not directly lead to failure of the water barrier, but increases the probability of failure by a subsequent mechanism. This includes the test track wave overtopping foreland, shearing of the foreland, subsidence foreland, port dams and non water retaining objects. Because it is not yet known how the possible positive contribution of a NWO can be calculated, it is currently stated that there is only a negative effect. However, NWOs can reduce the strength of the flood defence or increase the load. The first question when assessing NWOs is therefore whether the object contributes to the probability of flooding or whether this contribution is negligible (Rijkswaterstaat, 2017).

The assessment starts with the identification of potentially risky NWOs. The manager may choose to postpone the assessment of the NWOs until the next assessment period, if this choice meets the conditions described earlier. The assessment method for NWOs includes three steps (Rijkswaterstaat, 2017):

1. Simple test: if the simple test is passed, the contribution of the NWO to the probability of flooding is negligible.
2. Detailed test: modelling determines whether the contribution of the potentially high-risk NWOs to the probability of flooding is small in comparison with the other failure mechanisms of the flood defence.
3. Advanced assessment method: based on scenarios the contribution of the NWO to the probability of flooding is taken into account.

2.3.2. Buildings (NWObe)

The available tests for the NWO building type are discussed in more detail in this section. The assessment of NWObe consists of a simple test and the tailor-made test.

<u>Simple test</u>

Figure 4 shows the steps of the simple test. Depending on the outcome of the previous step, the assessment is continued or the influence of the NWO is considered negligible. The NWO assessment is only relevant if the specific flood defence or harbour dam without NWOs meets the requirements of the relevant test tracks. The relevant test tracks are listed in Table 3 depending on the location of the NWO in the cross-section and the type of water barrier, port dam or foreland. If the assessment of the relevant test tracksis already insufficient, a further NWO assessment does not need to be carried out. If the water barrier without NWOs does meet the requirements on the relevant test tracks, the assessment continues. The relevant test tracks determine which hydraulic load must be used (Rijkswaterstaat, 2017).

Table 3: Relevant test tracks per type of flood defence, port dam or foreland in relation to test track NWO (Source: (Rijkswaterstaat, 2017))

Location NWO	**Foreland**	**Dike/ (Port) dam**	**Hydraulic structure**	**Dune**
Foreland	VLGA	STBU	STKW	DA
	VLAF	Revetment	PKW	
	VLZV	STPH		
Flood defence		STBU	STKW	DA
		STBI		
		Revetment		
		STMI		
Hinterland		STBI	STKW	
		STPH	PKW	

When the NWO meets the requirements of the relevant test tracks it can be assessed on the basis of general characteristics. Relevant characteristics are the disturbance profile in relation to the zone of influence and the assessment profile, presence of compensatory facilities and the size of the built-up area. The disturbance profile depends on the depth of any cellars and crawl spaces in relation to ground level (Rijkswaterstaat, 2017). The disturbance profile is determined according to Figure 3.

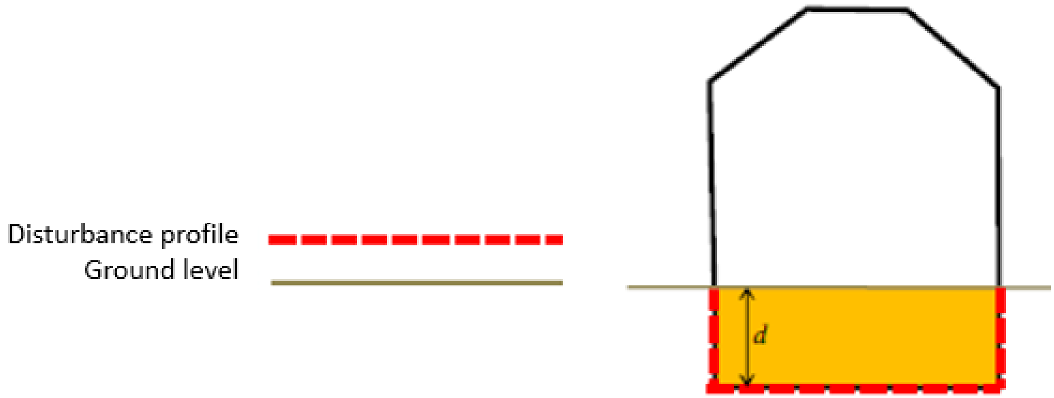

Figure 3: Determination disturbance profile (Source: (Rijkswaterstaat, 2017))

The zone of influence is the area belonging to the water barrier which actually contributes to ensuring stability, both on the inside and the outside of the water barrier. The assessment profile is a minimum dike profile that is sufficient to retain high water for a short period. The assessment profile may not be intersected by objects that do not have a water retaining function. A compensatory facility is seen as a functional separator between flood defence and buildings and should be assessed as a hydraulic structure (Rijkswaterstaat, 2017).

With regard to the assessment at NWOs, the influence of NWOs on piping is currently one of the least investigated topics (as far as the author is aware of). Therefore, in this study it was decided to focus on this topic. If the NWO does not pass the simple test, a more detailed assessment is required with the expert assessment (Rijkswaterstaat, 2017).

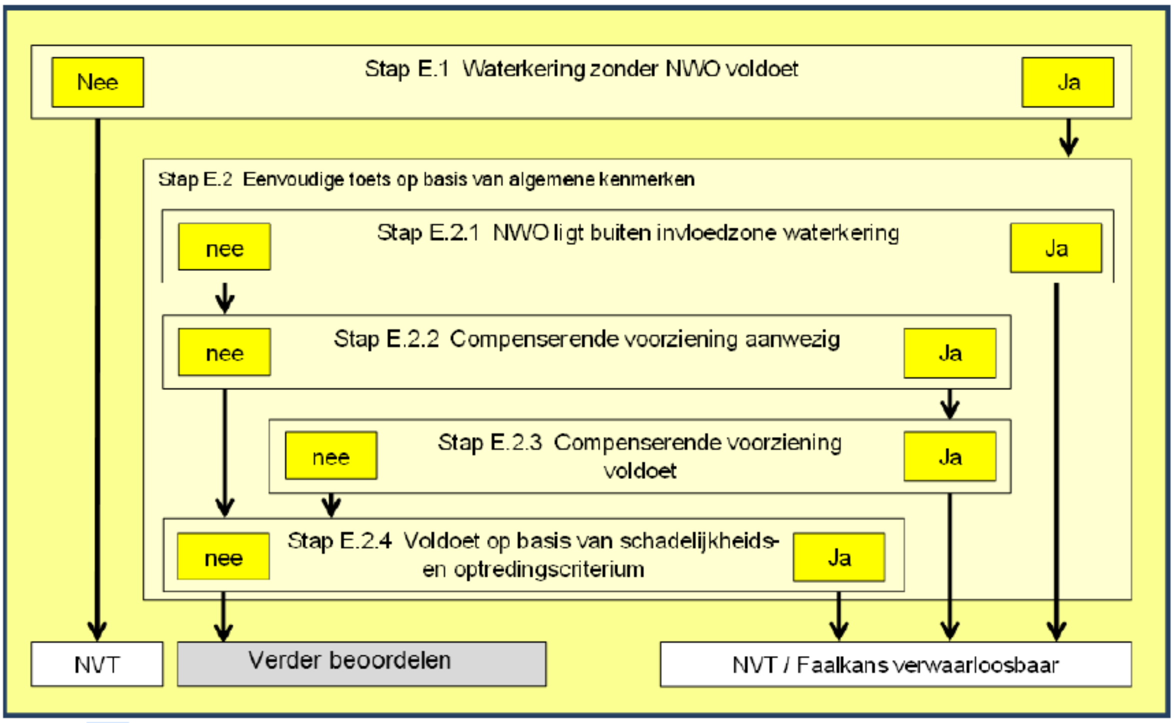

Figure 4: Schematization simple test for NWOs (Source: (Rijkswaterstaat, 2017))

<u>Advanced Assessment Method</u>

If the simple test is not sufficient, the influence of the NWO can be included in the assessment within the expert assessment for the building per mechanism. In the case of buildings, the influence of a building is generally approached by modelling the building as an excavation pit which can be included as a scenario in the model schematization for various direct mechanisms. On the other hand, further analyses can show that the influence of the NWO is negligible on the failure probability of the flood defence (Rijkswaterstaat, 2017).

2.4. Flow Chart

The flow chart that is developed in this study aims to fill the gap between the simple test and the expert assessment. It is not feasible to carry out an expert assessment test for all available structures, but within the simple test the unique characteristics of a structure are not taken into account. Within the existing assessment process, the flow chart has to fit between the simple test and the customised test. Therefore, the flow chart must meet a number of requirements. It has to provide more information on the structures than the simple test does, but at the same time not be more time and economically demanding than the expert assessment. In addition, the results of the flow chart must have consequences where necessary. If the assessment shows that the structure has a significant negative effect on the probability of failure, the flow chart must clearly show which follow-up steps must be taken.

3. Methodology

The development of a flow chart for the approach to piping at a NWO is divided into two parts. In the first part the characteristics of a property that may influence piping are identified. Then it is also investigated how exactly these characteristics influence the failure mechanism by means of a literature study. In the second part, a flow chart will be developed using the collected knowledge to simplify the determination of whether piping is a relevant risk.

3.1. Identifying influential structure characteristics and their influence on piping failure mechanism

3.1.1. Identifying influential structure characteristics

The first step in this research is to determine the main characteristics of a structure that may affect the failure mechanism. After analysing the piping calculations, the most noteworthy thing was that the entire process takes place in the subsurface. Therefore everything that takes place in the subsurface has a direct or indirect influence on the failure mechanism. This means that indirect influences on the subsurface can also affect the probability of failure. Thus, it was first examined which characteristics of a structure influence the subsoil.

3.1.2. Identifying influence of influential structure characteristics on piping failure mechanism

A literature study was carried out to determine the influences of the structures characteristics on the probability of piping. For each property different researches have been examined to determine how it affects the probability of piping and how big the influence may be.

3.1.2.1. Influence of foundations

First of all, the effect of a pile foundations was investigated. In the past, research was done on the influence of driven piles on archaeology. (Caspers, Knol, & Kars, 2011) look at how the driving of piles can cause perforation of different type of soil layers and the associated consequences. (Huisman, Müller, & Doesburg, 2011) look at three archaeological sites to see how they are affected by pile driving. It focuses, among other things, on how big the area of soil is that is affected by a pile. In (Hird, Emmett, & Davies, 2006) the effects of piling through different sand and clay layers were studied at scale. The disturbance of the layers as well as the possible increase in groundwater flow are discussed in detail here. The literature studied did not elaborate on the influences of shallow foundations. However, in consultation with experts from the HHNK and Deltares, the possible effects of building on a shallow foundation were included in the development of the flow chart.

3.1.2.2. Influence of substructures

The possible effects of a basement or cellar under a building were investigated. Several research reports by Fugro, an engineering firm specializing in soil mechanics and foundation engineering, describe these effects. (Fugro, 2010) and (Wimmers, Sweijen, Brugman, Meinhardt, & Maljaars, 2020) both look at how groundwater flow might be affected by the presence of a basement under a property. These and similar research reports have therefore been reviewed to gain a better understanding of the issue. The literature was also studied for possible effects of a foundation with or without a crawl space. No literature was found in which possible influences of a crawl space are discussed, but there are reports in which erosion in basements and crawl spaces are mentioned. The most interesting example of this is the report "De Onderste Steen" which looks at piping below houses along the Almelo - De Haandrik canal as a result of dredging work in the canal (Baars, 2020).

All possible effects described in the literature were mapped for each property characteristic. These effects were then discussed in expert sessions with experts from both the HHNK and research institute Deltares. Based on their advice, adaptions were made to the criteria for these effects when necessary.

3.2. Development of flow chart describing the approach to the safety assessment of a levee including a structure

3.2.1. Criteria

Once all the influences of the various influential characteristics of a structure had been mapped out, criteria could be developed for the flow chart. For the development of these criteria, there was looked at what a structure and the location should comply with in order for a certain effect to occur. In most cases, the effects of the various influential characteristics of a structure only occur through a combination of circumstances. To develop the criteria, first there was started with the characteristics of the structure. The presence of these characteristics form the first criteria for the occurrence of the effect. Subsequently, these characteristics must meet certain criteria before they can have a measurable influence. When a structure meets all criteria, it is possible that the location still has some characteristics that will prevent the structure from having a measurable impact. Therefore, location-specific criteria were developed with regard to the soil structure and hydraulic conditions at the location of the structure. If both the structures criteria and the site-specific criteria are met, it can be assumed that the influence of the structure is in force.

3.2.2. Flow chart

When the criteria were determined, a flow chart could be developed. This flow chart should make it easier for the assessor to decide whether additional calculations are required. The approach of developing the flow chart is described in this section. Before starting the development of the flow chart, failure paths for all failure mechanisms and literature regarding their development were searched.

A failure path describes a sequence of events leading to a flood. By focusing on events that lead to flooding and combining these events, analyses and calculations with available models, a better understanding of flood probability and its explain ability is created. (Ministerie van Infrastructuur en Waterstaat, 2020) describes a generic approach to considering the probability of flooding with failure paths. The approach consists of 5 generic steps which can be further subdivided depending on the application:

1. Drafting a story: describing the ways in which a flood can occur;
2. Draw up event trees/failure paths based on narrative: structure components to be considered further;
3. Elaboration of failure paths per component: elaboration of nodes in an event tree/failure path;
4. Analysis of relevant nodes in the event tree/failure paths;
5. Determining probability of flooding;

There are many possible failure paths leading to flooding, specifically when including all the diverse building characteristics. Only a few are normative and will determine the probability of flooding. With regard to a failure path for piping, first the approach "chain of events of flooding by piping" for a standard dike profile free of any NWOs, as described in (Deltares, 2021) was analysed. This describes the 5 steps after which piping leads to a breach.

This chain of events consists of:

1. Increase in water pressure and hydraulic fracture
2. Heave
3. Horizontal pipe growth
4. Continuous pipe and pipe widening and deepening
5. Slope lowering, overflow and breach growth

The knowledge of existing failure paths and event trees has been combined with the knowledge gained in this study. The events chain as described in (Deltares, 2021) was combined with the criteria developed in the previous step. Due to the earlier determination of how each characteristic affects the sub-mechanisms of piping, the characteristics could be directly linked to the steps in the event chain. After this link of the structural characteristics to the event chain, the subsequent criteria could also be linked automatically. These criteria were already specifically determined per structure characteristic as described in the previous section. After linking the structures characteristics and the criteria to the event chain, the flow chart was created. Recommendations and references to schematizations for additional calculations were added at the end of the various branches of the flow chart. It is important to note that the end result is a flow chart and not a failure path. The most important difference with a failure path is that a failure path also considers the probability of a certain effect occurring. In the case of the flow chart developed in this study, the probability that a structure meets a certain criteria is not considered. Therefore, it is not a failure path.

4. Results

For the development of the flow chart, characteristics of structures that may influence the probability of piping were examined through a literature study and discussions through expert sessions. Following the results from the study and the sessions, it was examined how exactly these characteristics influence the probability of piping and how this can be included in the model calculations. In this section, the results of this research will be presented by linking the determined influences of the structure characteristics (where possible) to the piping failure mechanism.

4.1. Influential structure characteristics and their influence on piping failure mechanism

In this section, the found influential characteristics of a structure are described first. Then, in the second part, the influences per characteristic are described.

4.1.1. Influential structure characteristics

4.1.1.1. Foundations

The first distinction is made in the type of foundation. Figure 5 shows two different foundations under a structure. On the left, a shallow foundation and on the right, a house resting on a pile foundation. Different types of foundations can affect the subsurface in different ways. There are different ways of applying both foundation types.

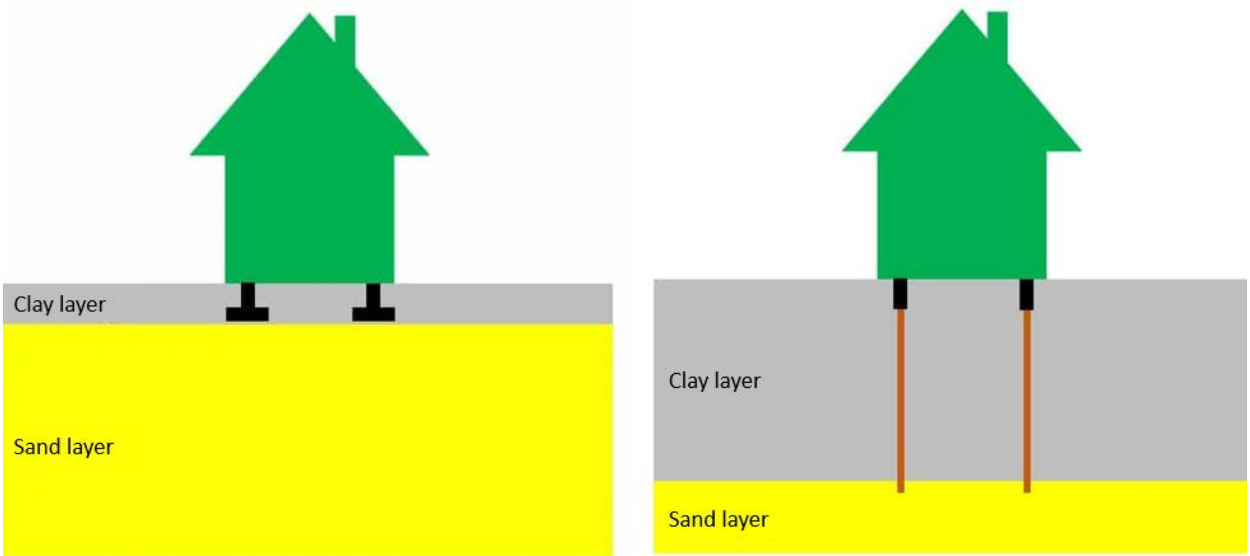

Figure 5: House with shallow foundation (left) and house with pile foundation (right) (Source: vastgoedmentor.com)

Pile foundation

Generic properties related to a pile foundation are the diameter of the piles and the length of the piles. In addition to these generic characteristics, there are also more specific characteristics of a pile foundation that can have a different impact on the subsurface. When building on a pile foundation, there are different types of piles and ways of installing the piles. First of all, there are different types of piles: timber piles, steel piles, concrete piles, precast concrete piles, composite piles. All these types of piles can have different shapes and dimensions and therefore have different impacts on the subsoil. In addition, different types of piles are installed in the ground in different ways. Figure 6 shows two examples of how different types of piles can be installed. In the left image, a steel tube is driven into the ground and then the hollow tube is filled with concrete and reinforcement. In the illustration on the right, the steel tube is screwed into the ground and then concrete and reinforcement are poured into the screw pipe, after which the screw pipe is removed. Besides these techniques, there are numerous other techniques for installing piles. The most important distinction is between soil displacement piles and non-ground displacement piles. With soil displacement piles, the soil at the location of the pile is pushed away, whereas the soil at the location of the pile is removed with non-graded piles.

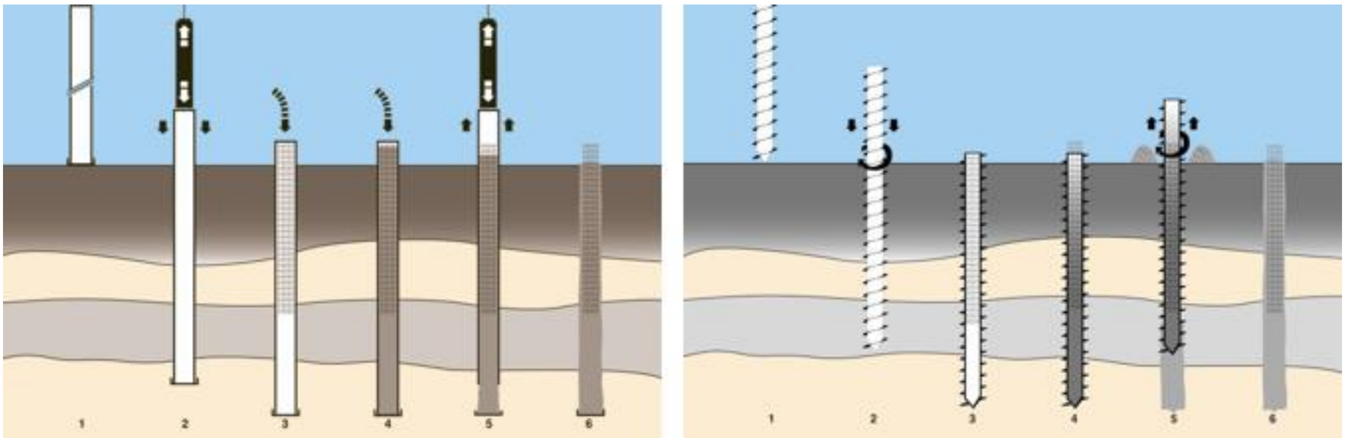

Figure 6: Installation of driven tubular piles (left) and screwed piles (right) (Source: Vroom funderingstechnieken)

When the piles are in the ground, the concrete of the floor is poured so that the weight of the floor and the building is transferred to the piles. The soil under the floor between the piles may settle over time due to settlement. This can create hollow spaces under the structure.

Shallow foundation

When a building is resting on a shallow foundation, a base is usually built on the load-bearing sand layer, and the walls are then built on this base. However, in newer houses, prefabricated or cast-in-place mat foundations are often used. Figure 7 shows in a 3D drawing how these shallow foundations are usually constructed.

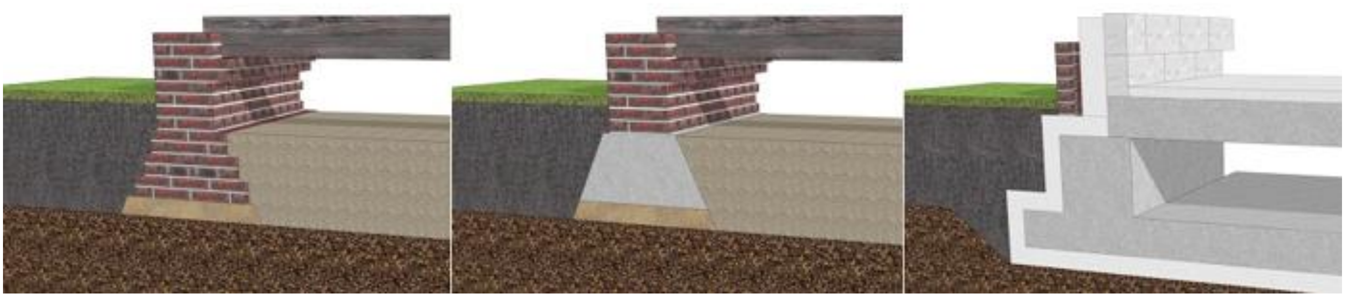

Figure 7: Different types of shallow foundations (Source: bouwhistorisch bureau www.moned.nl)

4.1.1.2. Substructures

The following distinction is made in the types of substructure that can be present. The main distinction is made between a cellar or basement and a crawl space. A cellar refers to a (habitable) part of the house that is completely below ground level. This can also be seen in Figure 8 on the left. A basement is a (habitable) part of the house that is partly below ground level but also partly above it. This can be seen in Figure 8 on the right. Not only is the presence of a cellar or basement considered, but also the possible presence of a hollow space under the property. Some buildings have a so-called crawl space. This can also be seen in Figure 8 on the right. Even in a house with a cellar or basement, it is possible that there is a crawl space underneath. For all the different types of substructure, it was investigated how and whether they could influence the piping failure mechanism.

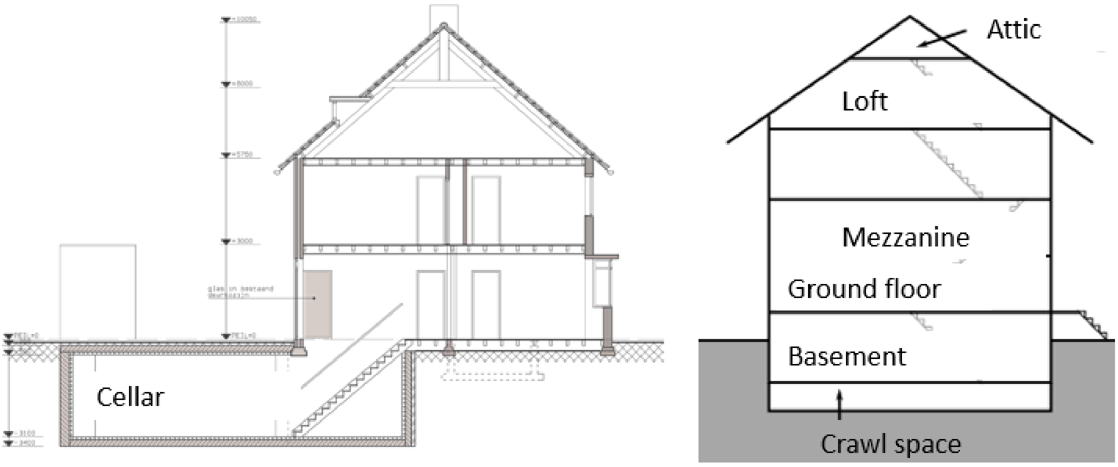

Figure 8: House with cellar (left) or basement (right) (Source: left: 3dwarehouse.sketchup.com; right: Wikipedia)

4.1.2. Influence of influential structure characteristics on piping failure mechanism

4.1.2.1. Foundations

Pile foundation

With regard to the influences of a pile foundation, the most important results are found in (Hird, Emmett, & Davies, 2006). Several tests with different pile types have been performed at a nominal geometrical scale 1:10. The most interesting result from this research is that when the ratio between the thickness of the existing clay layer and the pile diameter is 1:1, perforation of the clay layer occurs. In other words, at the location of the pile, it then becomes possible for the water to move upwards along the pile shaft, from the lower aquifer to the upper sand layers.

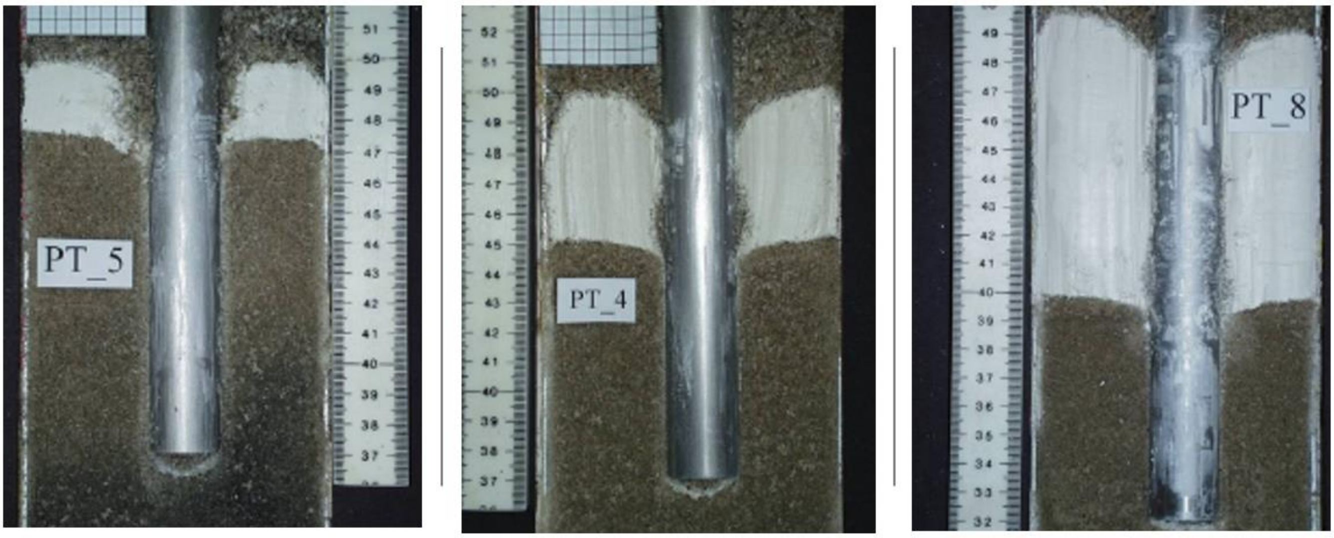

Figure 9: Photographs of dissected axisummetric models: cylindrical piles (Source: (Hird, Emmett, & Davies, 2006))

Figure 9 shows what such a perforation looks like. It is easy to see how sand from the top layer has been pushed down through the clay layer. The key features for the cylindrical driven piles are the downward-dragged sand at the top of the clay layer, the downward-dragged sand shed along the sides of the pile within the clay, and the small amount of sand carried down beneath the pile tip through the clay and into the lower stratum (Hird, Emmett, & Davies, 2006).

In the study by (Hird, Emmett, & Davies, 2006) not only cylindrical driven piles but also H-section piles and so-called CFA piles are considered. Since H-section piles are not commonly used in the Netherlands, these results are not considered relevant here. The CFA piles are comparable to the so-called screw piles that are commonly used in the Netherlands. Figure 10 shows how the soil around a CFA pile behaves.

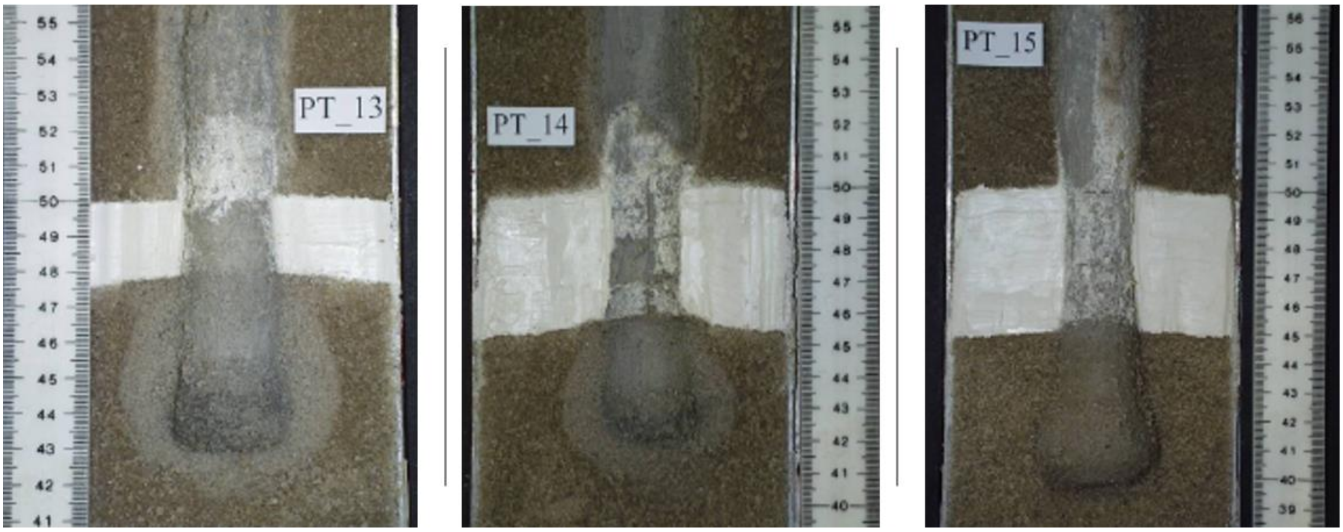

Figure 10: Photographs of dissected axisymmetric models: CFA piles (Source: (Hird, Emmett, & Davies, 2006))

The CFA piles show modelling defects such as variation of diameter and excessive penetration of the sand layers by grout. Nevertheless, the photographs clearly show that deformations in the clay around the pile are relatively small compared to those with the other pile types. What is noticeable is that the way in which the pile is screwed into the ground, does cause considerable displacement of the lower sand layer. The leftmost picture clearly shows how first the clay layer is screwed up and then part of the aquifer is screwed into the clay layer.

In the study by (Hird, Emmett, & Davies, 2006) the consequences for the permeability of the clay layer were also examined when piles are installed. Tests were carried out for various clay layer thicknesses with the different pile types. In the tests, water pressure was increased within the lower sand layer and it was checked whether an increase in the flow from the lower to the upper sand layer could be measured after the piles had been installed. In Appendix C: Groundwater Flow Results, a summary of these groundwater flow results can be found. The most important thing that can be concluded from these results is that in some cases there is a significant increase in permeability as a result of driven pile construction. However, it should be noted that it cannot be assumed that this significant increase is also of effect for a full scale situation. This is dependent on the ratio between the full scale pile diameter and the scale model diameter. In order to interpret the results at full scale it is better to think of either the increase of flow caused by the construction of a single pile, under a given hydraulic gradient across the clay layer, or the dimensions of an equivalent column of overlying soil adjacent to the pile in the clay. It is also assumed that individual piles do not interfere with one another, which is likely to be true if the spacing exceeds about three pile diameters.

After studying the research results of (Hird, Emmett, & Davies, 2006) it was examined how these could be applied to a structure on pile foundations. As the research showed, perforation of the clay layer only occurs at a 1:1 ratio between the thickness of the clay layer and pile diameter. The average pile diameter was therefore examined first. Prefabricated piles vary in diameter between 180 and 500 mm (IJb groep, 2022). These findings were presented to experts of the HHNK and Deltares and it was discussed whether it is plausible that water leakage along pile foundations can occur in this way. Both the experts from the HHNK and Deltares find it plausible that at a 1:1 ratio between clay layer and pile diameter water from the aquifer can move along the pile to the sand layers above. Figure 11 shows what such a process would look like for a dike with a home in the hinterland.

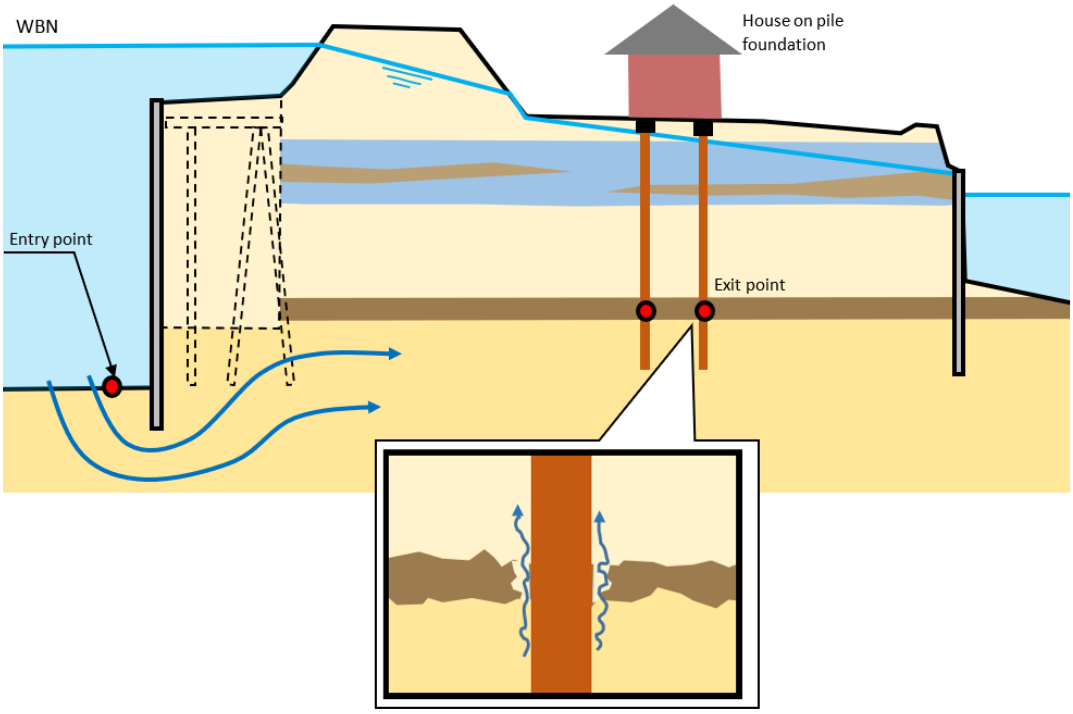

Figure 11: Hydraulic fracture of the clay layer at pile foundation

With the present knowledge that the driving in of piles can affect the permeability of the clay layer, it can be determined how this affects the piping calculations. Figure 12 shows how the driven piles can influence the length of the seepage path (L). When the original exit point of the potential pipe is behind the structure, it is likely that the exit point will move to the place where the piles perforate the clay layer, if the hydraulic pressure in the aquifer is sufficient for an upward flow. Considering the major influence of the seepage path length on the probability of piping, it is therefore likely that this displacement will cause a significant change in the overall probability of failure, in relation to a situation in which the NWO is ignored. The perforation of the clay layer will also increase the local vertical permeability. This may affect the 0.3 x thickest layer rule that is currently applied to account for the vertical permeability of the layers. However, this effect has further not been considered in this study, because it could not be incorporated in the existing parameters within the piping calculations of the HHNK.

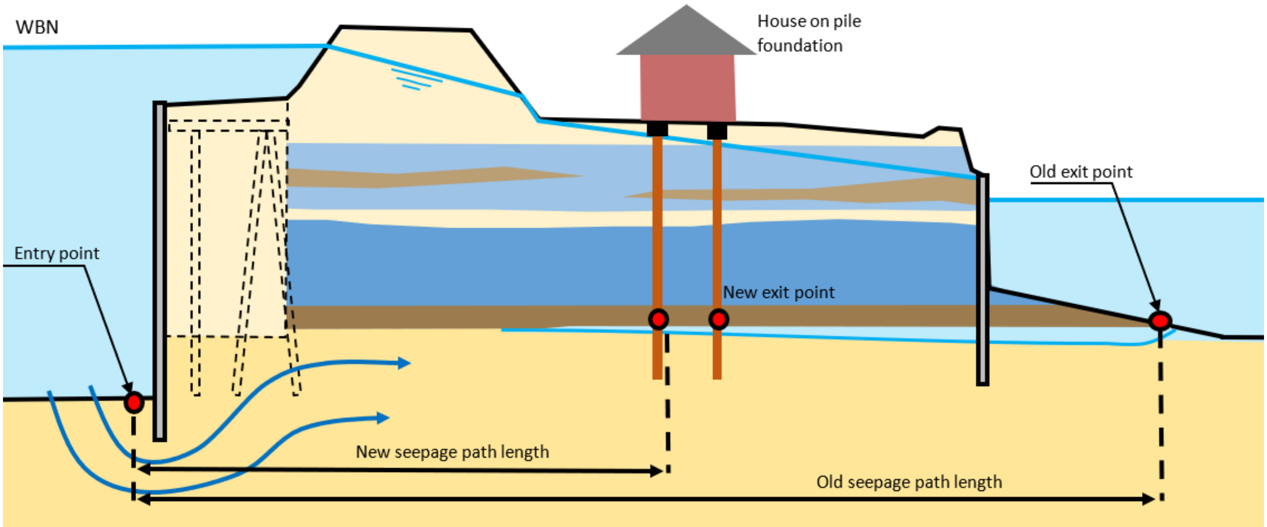

Figure 12: Old seepage path without NWO and new seepage path with NWO

In addition to the effects caused by the piles, the creation of hollow spaces under the structure was also examined. Settlement can cause such hollow spaces to appear under the floor. The effects of such a hollow space are comparable to those of a crawl space. This is described in section 4.1.2.2.

Shallow foundation

With regard to a shallow foundation, there is a different effect on the failure mechanism. The weight of a structure on the footing presses directly on the top layer of soil. This is in contrast to a structure on a pile foundation where the weight of the structure is transferred to the deeper load bearing sand layers, through the piles. When building a structure, a foundation is always located below ground level, both for a shallow foundation and for a pile foundation. This excavation naturally reduces the overburden pressure of the subsoil on the water pressure acting on the upper boundary of the aquifer. However, in the case of a house on a shallow foundation, the weight of the structure is transferred directly on the upper layers, which ultimately increases the overburden pressure. The increase in soil pressure can vary greatly from one type of building to another, as it is strongly related to the materials used and the dimensions of the structure. The dimensions of a shallow foundation are generally designed meeting certain settlement and bearing capacity criteria. Figure 13 shows how a shallow foundation increases the soil pressure on the subsoil below the structure. Looking at how the piping process works, it is to be expected that the probability of hydraulic fracturing, heaving and backward erosion will decrease when the soil pressure increases

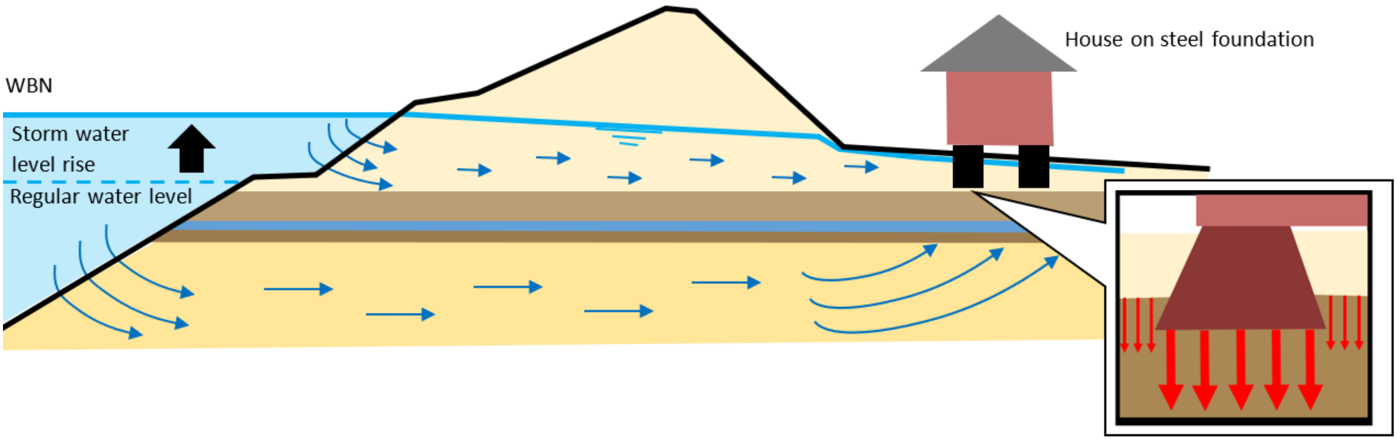

Figure 13: Increase pressure on subsoil due to a shallow foundation

4.1.2.2. Substructures

Crawl space

A crawl space, from which the least impact is expected due to its smaller dimensions, was considered first. Especially in older homes, there is often still a crawl space under the structure through which gas, water, electricity and sewage pipes run. As described earlier, in the case of a structure on pile foundations, it is likely that erosion will occur along the piles when the pile diameter/ clay layer diameter is 1 or higher. When this happens, it is likely that the pipe starts in the crawl space, as the piles also end here under the foundation. When such a pipe ends in the crawlspace and there is enough hydraulic pressure for material transport, the material will be expected to start to accumulate in the crawlspace. As a result, the crawl space will slowly fill up with sand. Figure 14 shows a schematic representation of what this might look like.

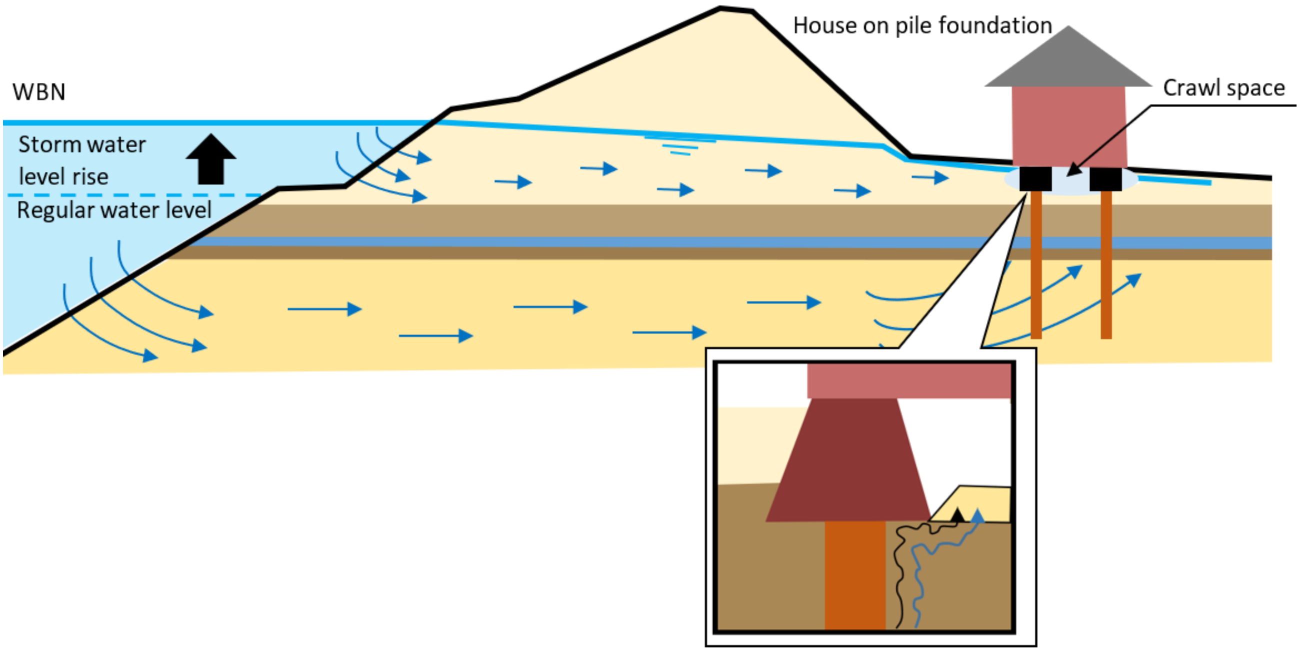

Figure 14: Material accumulation in crawl space

The accumulation of material in the crawl space can basically continue until the space is full. However, it is also possible that the process stops earlier due to a decrease in hydraulic head in the crawl space. When the crawlspace is full or the pressure has increased to such an extent that the piping process in the crawlspace stops, it is possible that the pipe will seek another route. The possible ways in which the entire hydraulic fracturing process takes place in the crawl space have been discussed with experts from HHNK and Deltares. From the expert sessions it seems possible that when the piping process in the crawl space stops, the pipe works its way along the foundation of the structure to the surface. However, the chance of this happening is very small as the pipe would have to move up both horizontally and vertically. The chance that the pressure in the pipe is big enough for this is expected to be very small and is expected to depend on the hydraulic conditions in and around the crawl space.

Basement or cellar

The next types of substructure to be studied are a basement or cellar. These types of substructure are discussed together, as it is assumed that the effect is generally the same. However, in the case of a cellar, the effect is expected to be stronger than for a basement, as a cellar requires a deeper excavation of the subsoil. Naturally the construction of a cellar or basement requires considerable excavation of the subsoil. An excavation of almost 4 metres is commonly required to build a habitable cellar. This depth includes the foundation. The excavation considerably reduces the overburden pressure in the area below the base of the structure. Of course, this only applies if the structure is built on pile foundations. If the structure has a shallow foundation, the decrease due to excavation and the increase due to pressure from the structure will likely cancel each other out. The sharp decrease in overburden pressure with a structure with a foundation on pile foundations increases the risk of piping.

In addition to a reduction in the overburden pressure, there is another effect that plays a part in the construction of a cellar or basement: the so-called barrier effect. This barrier effect is described in several reports from geodata expert (Fugro, 2010). This is not the only report that discusses this effect; there are several reports that study this effect at buildings with a substructure. Barrier effect is the phenomenon whereby the groundwater level (or head) is influenced by an underground watertight or low permeable construction. An underground construction can for example be a basement, cellar or a sheet piling.

Groundwater flows on both local and larger scales. On a local scale, this is, for example, rainwater sinking into the ground where it then drains towards the surrounding watercourses. On a larger scale, this is the flow of rainwater after infiltration in deeper ground layers tens of kilometres towards the sea. By placing a watertight underground construction, this flow can be hindered in a certain zone. A basement or cellar can act as an obstruction to the flow pattern. Obstructing the groundwater flow leads to higher groundwater levels on the upstream side (left side in Figure 15) and lower groundwater levels on the downstream side (right side in Figure 15) (Fugro, 2010).

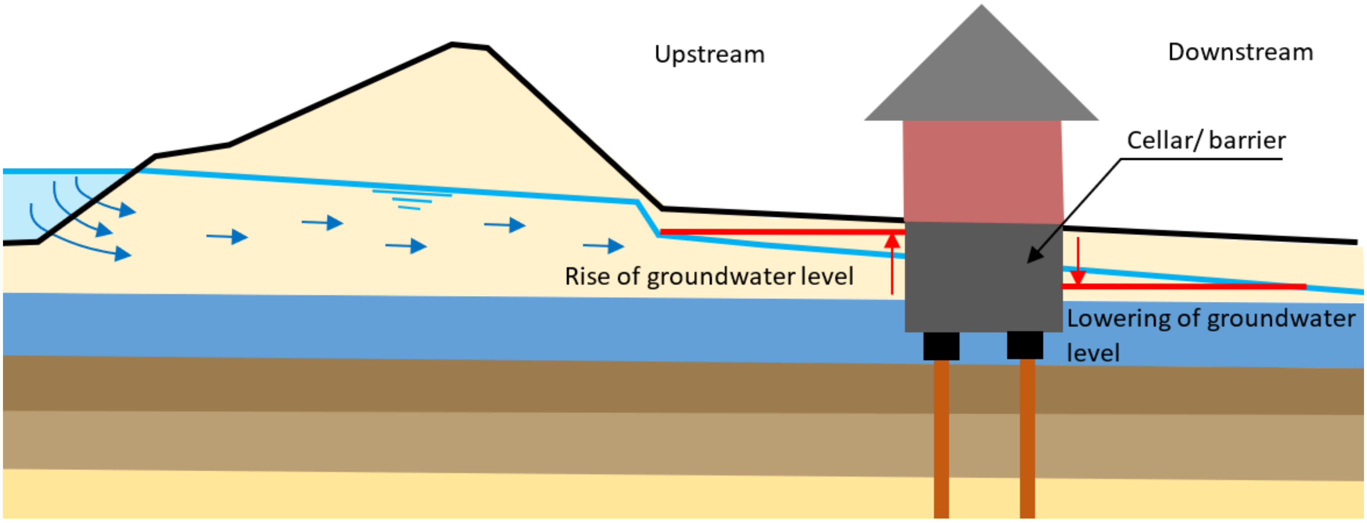

Figure 15: Principle of barrier effect

The degree to which the barrier effect comes into effect depends on four factors (Fugro, 2010):

1. The size of the barrier realised in relation to the direction of flow of the groundwater;
2. The depth of the barrier in relation to the soil conditions and the extent to which the underground construction elements cut through aquifers;
3. The soil conditions (vertical permeability) of the layers below the barrier;
4. The degree of horizontal groundwater flow;

If all four factors are unfavourable, significant uplift and subsidence of the groundwater level will occur in the vicinity of the underground construction. The four factors are elaborated on because the effect only occurs at a structure when these four factors are met. First of all, the dimensions of the barrier. Small cellars under a structure measuring, for example, 5 x 10 metres do not in themselves have a significant influence on groundwater flow. The water can easily flow around the barrier here. Large basements or small basements located close to each other can have a barrier effect. In addition to the size of the cellar, the orientation with regard to the direction of groundwater flow is also important. If a narrow, long cellar is located parallel to the direction of groundwater flow it will have only limited influence. Figure 16 shows how cellar dimensions and orientation influence the extent of water impoundment (Fugro, 2010).

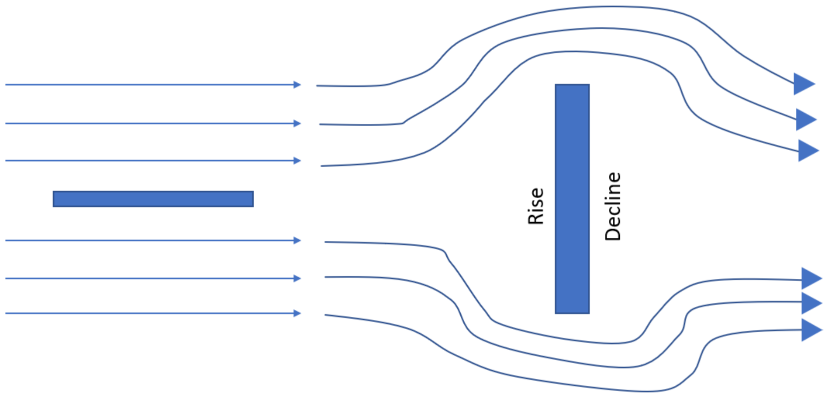

Figure 16: Top view of the barrier; The orientation with respect to the groundwater flow direction determines the obstruction, and thus the impoundment, of the groundwater.

Secondly, the depth of the barrier. The barrier depth is related to the depth of the basement in combination with the local soil structure. Calculations made by Fugro show that an underground construction will only really hinder the flow of groundwater if it blocks a large part (approximately 70%) of an aquifer (Fugro, 2010).

Figure 17 and Figure 18 show two examples of this. In Figure 17, a cellar completely seals off a sandy top layer, preventing phreatic groundwater from flowing under the cellar. Figure 18 shows a cellar that only partly seals off the sand layer. In the latter situation, groundwater can flow under the cellar via a relatively short diversion and does not cause any inconvenience (Fugro, 2010).

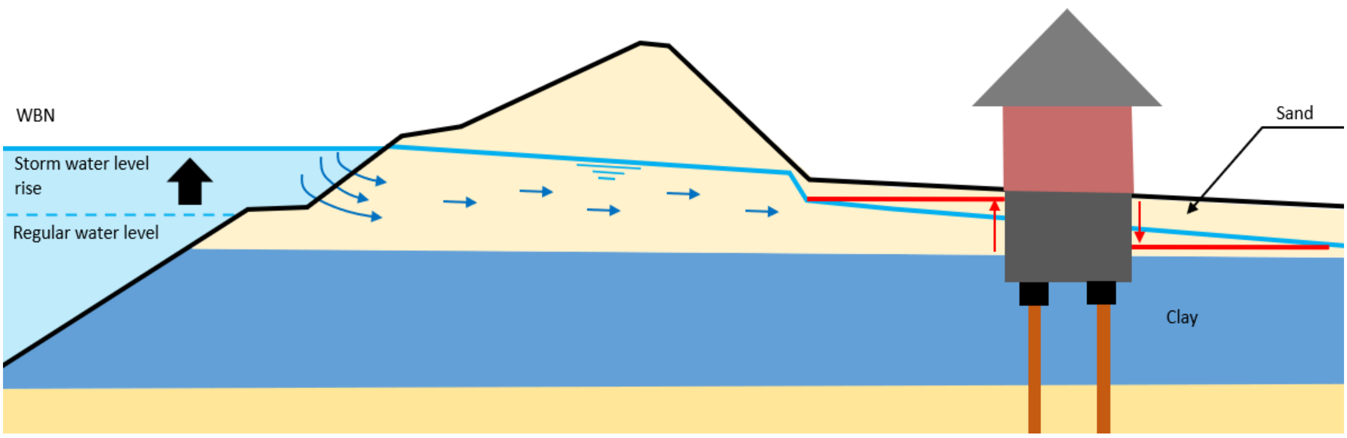

Figure 17: Noticeable impoundment can only occur when the cellar closes 70% of an aquifer or more.

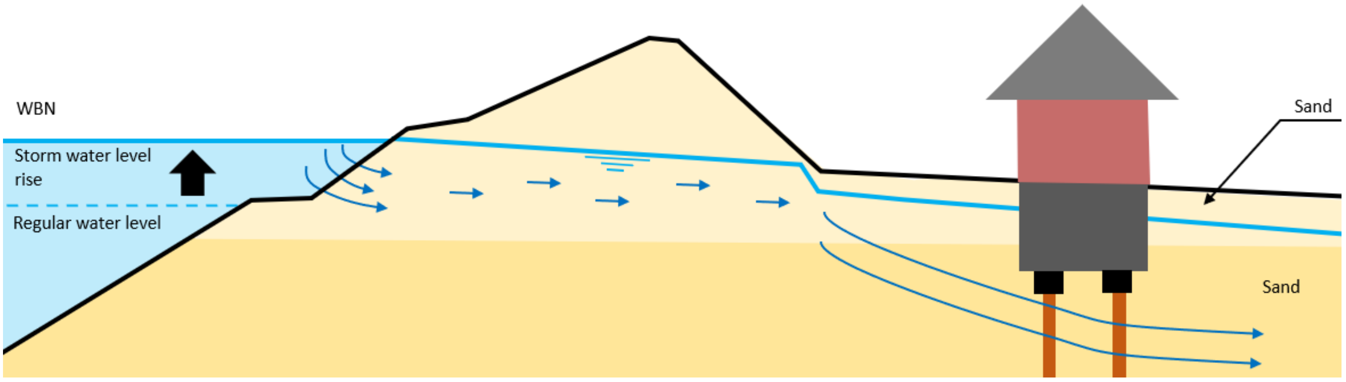

Figure 18: Water can move freely underneath the cellar in the sand layer

The next factor is the thickness of shallow clay/peat layers. When a basement seals off a large part of the sandy aquifer, the degree of barrier effect is related to the thickness of the underlying water-retaining soil layers. Clay and peat layers impede vertical flow, making it more difficult for groundwater to pass under the structure. Thicker layers cause a greater obstruction of vertical flow and therefore a greater risk of barrier effect (Fugro, 2010). This is schematically displayed in Figure 19.

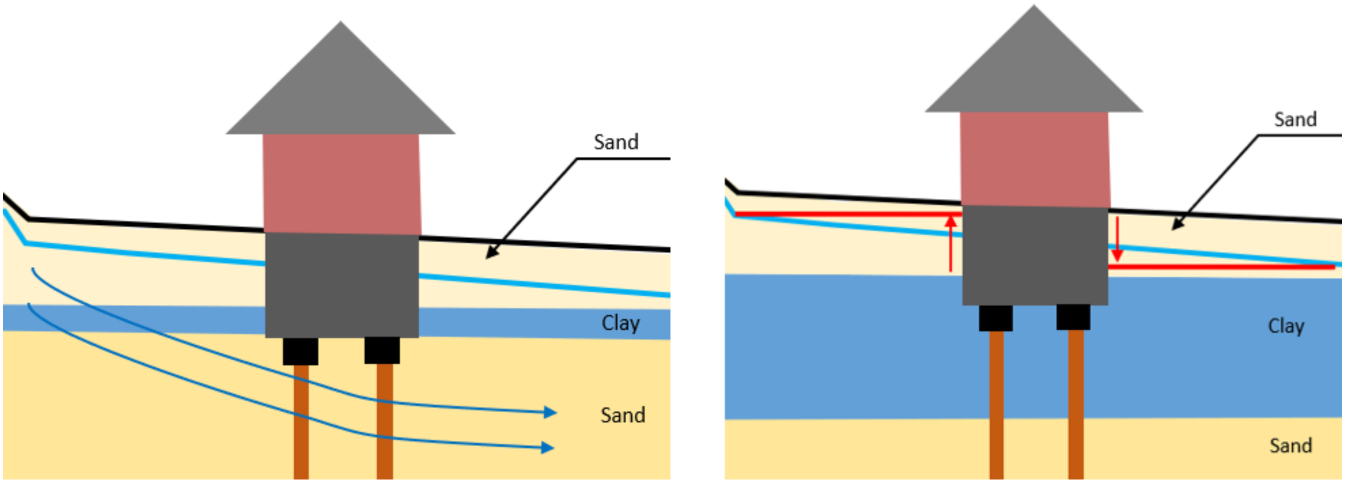

Figure 19: The degree of barrier effect depends on the thickness of underlying clay/peat layers.

Lastly, the factor of groundwater flow. The barrier effect is the obstruction of the natural groundwater flow. A stronger horizontal groundwater flow therefore causes more upwelling at the upstream side and a lower groundwater level at the downstream side. Horizontal groundwater flow is caused by differences in the groundwater level in the vicinity of the building. Water flows from a high water level to a lower water level. When the groundwater level differences in the vicinity are minimal, no obstruction occurs (Fugro, 2010). In Figure 20 can be seen how the groundwater level differences cause the barrier effect.

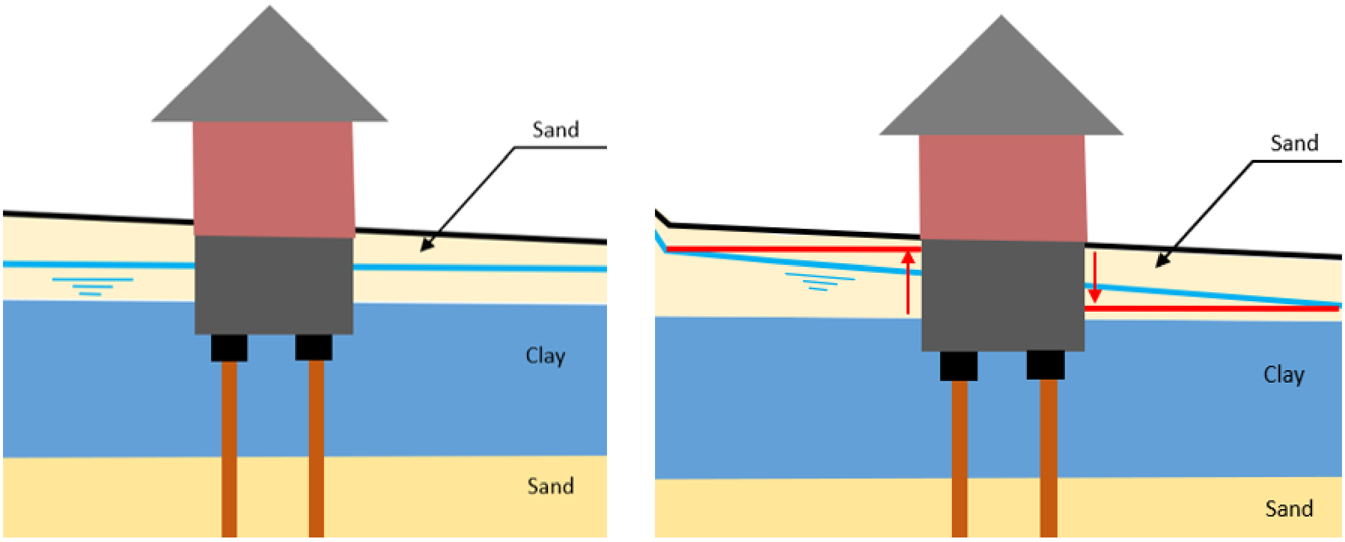

Figure 20: Impoundment depends on the horizontal groundwater level differences.

When the four factors act unfavourably on a dwelling behind a dike, the barrier effect can therefore occur. This may affect the probability of the occurrence of piping. However, the barrier effect can have an effect in both a negative and a positive way. The barrier effect influences the hydraulic gradient over the water barrier. When looking at the left (upstream) side of the structure on the house on the right (downstream) in Figure 20, the present hydraulic gradient decreases because the water level on the left increases due to the barrier effect.

However, when the other side of the structure is examined, where the groundwater level is lowered, there is an increase in the present hydraulic gradient. A bigger hydraulic gradient at the downstream side of a structure leads to an increased risk of piping. It is important to note that the barrier effect, in the described situations only affects the phreatic groundwater level. This means that the effects of the barrier only apply when the clay layer undergoes a hydraulic fracture. Depending on the composition of the top layer, it may be difficult for a pipe to deposit sand in the top layer. In case of a sandy top layer, it is likely that it is difficult for the pipe to deposit sediment. In Figure 21 it can be seen how the barrier effect causes differences in the hydraulic gradients on both sides of the building.

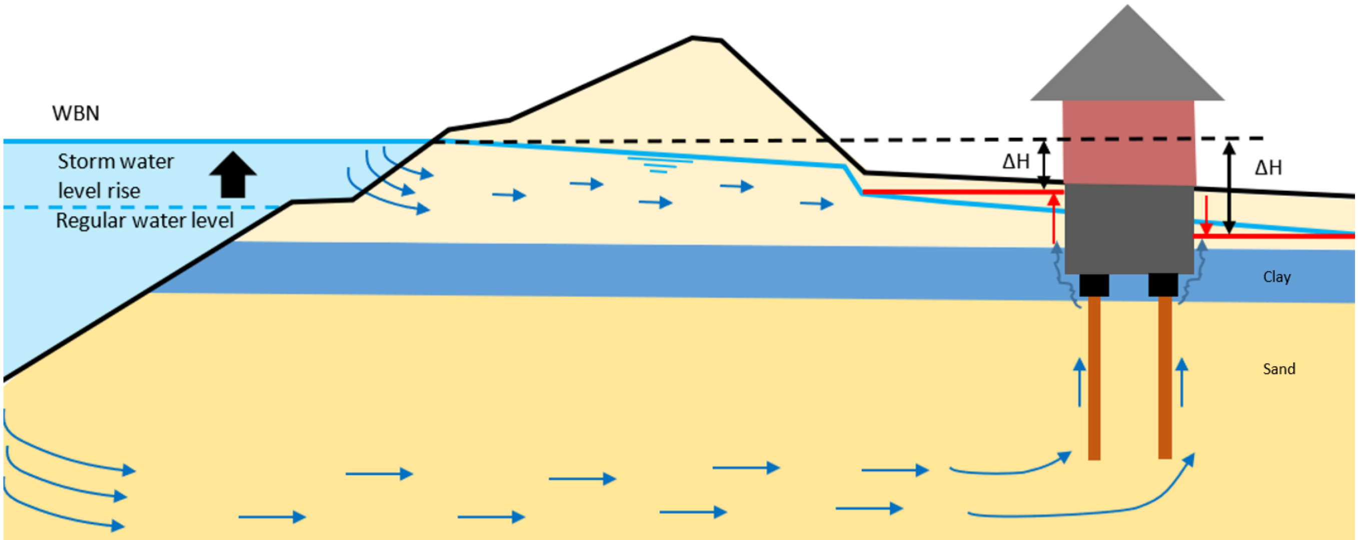

Figure 21: Difference in hydraulic gradients on both sides of building due to barrier effect.

4.2. Flow chart describing the approach to the safety assessment of a levee including a structure

The translation of the knowledge about influential characteristics in a structure on piping is described here. The same categories of structure characteristics are used for the criteria: foundations and substructure. For each influential characteristic of the structure, a link is also made to a sub mechanism of the piping mechanism.

4.2.1. Criteria

For each influential characteristic of a structure, the criteria are described successively.

Substructure

When a substructure is present there are two possible effects that can influence the piping failure mechanism. First of all it can cause a decrease in the soil pressure. The criterion that has to be met is that the structure has to be on a pile foundation. When this criterion is met, the decrease in soil pressure should be taken into account.

The other possible effect that can occur at a substructure is the barrier effect. The first criterion that has to be met is that the substructure should cover at least 70% or more of the area between the sand an clay layer. The second criterion is that a hydraulic head of 0.1 m or more should be present across the location of the property. The last criterion is that the orientation of the substructure should be perpendicular to the flow direction of the water. Or in other words, the substructure should be blocking of a big enough area for the water not to be able to flow around it. When these three criteria are met, it should be taken into account that the barrier effect occurs.

Pile foundation

When a pile foundation is present at a structure, it can have an influence on the length of the seepage path. The first criterion that has to be met is that the piles go deep enough to reach the aquifer. When this this criterion is met, the second criterion is that the pile perforates a clay layer as thick as the diameter of the pile, or thinner. Because it is a relatively large amount of work to find out the pile diameter for all buildings during a test, an average was sought and a safety margin was applied to this average diameter. The maximum average pile diameter is 350 mm (Betonhuis , 2022). The maximum pile diameter that can be delivered prefabricated is 500 mm (IJb groep, 2022). With a safety margin of 0.3 on the maximum average pile diameter of 350 mm, the maximum layer thickness that can be perforated is 450 mm. From this follows the second criteria that for a clay layer thickness of 450 mm or less, it is assumed that erosion along the piles poses a risk. Only when these two criteria are met the piles can have an impact on the seepage path.

Crawl space

The precise effect of a crawl space below a structure remains uncertain. However, criteria have been established to provide a basis for further research and to give an improved risk assessment, based on the current state of knowledge. The first criterion is that the structure has to have a pile foundation. When the structure has a shallow foundation it is assumed that hydraulic fracture will not occur below the structure. If the first criterion is met, the following criteria are the same as these for a pile foundation. When all criteria are met it is assumed that hydraulic fracture occurs in the crawl space.

4.2.2. Flow chart

Once all the criteria had been established, the flow chart could be developed. Due to the size of the flow chart, it is not possible to place it here in the report. However, a snapshot of part of the flow chart can be seen in Figure 22. This snapshot will be used to explain how the flow chart is structured. The complete flow chart can be found in Appendix D: Flow chart.

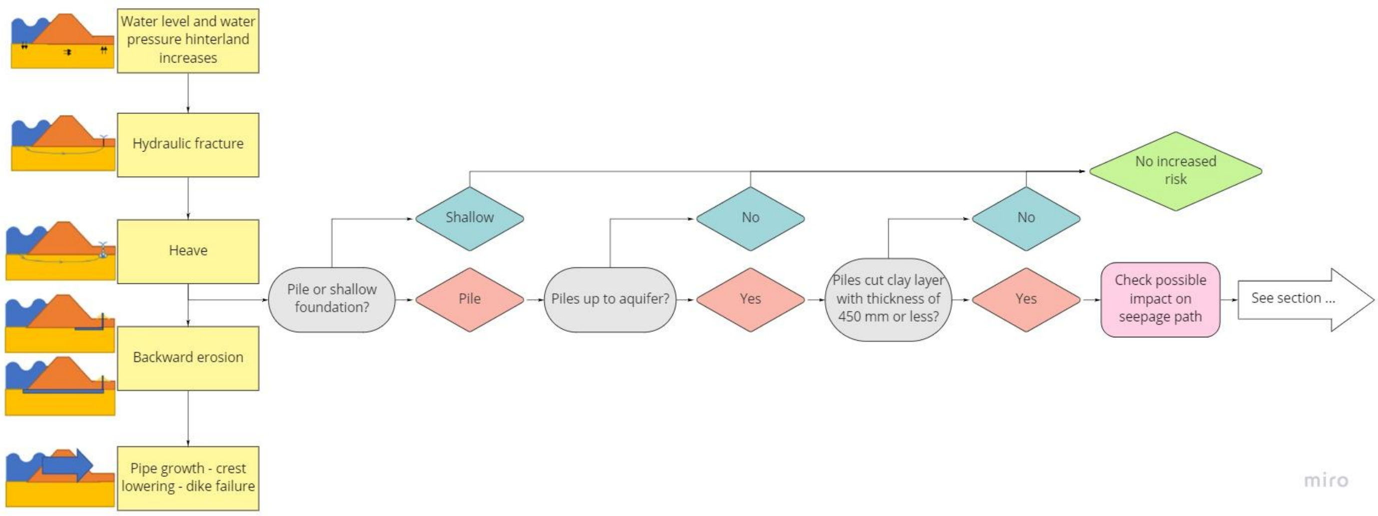

Figure 22: Snapshot of branch for foundation type assessment

The flow chart starts with the event chain of the 5 steps after which piping leads to a dike failure. Subsequently, for each sub-mechanism, characteristics of a structure are linked to the event chain which have to be considered in case they apply to the specific structure. In the snapshot in Figure 22 an example of branch for the foundation type has been given. When the water pressure in the aquifer increases, the type of foundation can influence the probability of backward erosion. Before making the backward erosion calculations it is therefore advised to consider the possible effect of the foundation. This is also the reason why this branch is located between the heave and backward erosion blocks. First of all it is determined whether it is a pile foundation or shallow foundation. Then the criteria as explained in the previous section are linked to the applicable foundation. Finally, the flow chart leads to two possible outcomes. If the criteria for an increased risk are not met, the conclusion is 'no increased risk'. This means that the assessment can continue without taking into account the influence of the structure. If all criteria for increased risk are met, an explanation is provided as to how the influence of the structure can be included in the calculations in a simplified manner. In order to make this as easy as possible for the assessor, schematics are provided showing the influence of the structure on the parameters. These schematizations are provided in Appendix E: Schematizations for additional calculations, together with the complete flow chart. In case of a pile foundation the assessor can determine the impact on the seepage path with the help of the schematization and include it in the calculations.

5. Case Study

This section answers the third research question: “How can the developed flow chart be applied in a case study for the assessment of a levee including a structure?”. First, the importance of a better assessment of the NWOs along the flood defence system around Den Helder is discussed. Subsequently, the choice of the case study location is explained. Then the additional calculations that were made are elaborated on. Finally, the developed flow chart is applied to the structures along the Nieuwe Diep to see if the flow chart leads to a logical assessment.

5.1. Dike section 13-4

The considered dike section for the case study is located north of the province of North Holland, bordering the North Sea and Wadden Sea. In Figure 23 this section (named “Normtraject 13-4”) is marked in black. In 2020-2021 this section was assessed by the HHNK. The dike section has a total length of 10.16 km, and runs from the part of the levee that transitions from a dune profile to an embankment profile at the Verlengde Helderse Zeewering at Huisduinen, to the Spuisluis Oostoever at the end of the Koegraszeekdijk. The dike section crosses the urban area of Den Helder and protects the area against floods from the North Sea and the Wadden Sea (Hoogheemraadschap Hollands Noorderkwartier, 2021).

The dike trajectory is divided historically into 4 main sections. The first section is the Verlengde Helderse Zeewering and forms the transition between the dunes and the hard barrier. The second section is the Helderse Zeewering, which is a 3.6 km hard flood defence against the North Sea. The dike is categorized as a traditional green dike with a sand core. At the location of the ferry port to Texel, the Helderse Zeewering merges into the third section: the Havendijk. This section is located along the Nieuwe Diep. This water is in open connection with the Wadden Sea and connects the Nieuwe Haven and the Noord-Hollands Kanaal. The slope revetment on the Harbour dike is not uniform. Depending on the location, the dike is covered with asphalt, clinker, basalt and grass cover. There is also a road over part of the dike and a strip of buildings inside the dike, some of which are located at the toe of the inner slope. The final section of the dike is formed by the Koegraszeedijk. The dike runs for approximately 1.9 km to the Oostoever drainage sluice and borders the Noord-Hollands Kanaal on the inside (Hoogheemraadschap Hollands Noorderkwartier, 2021).

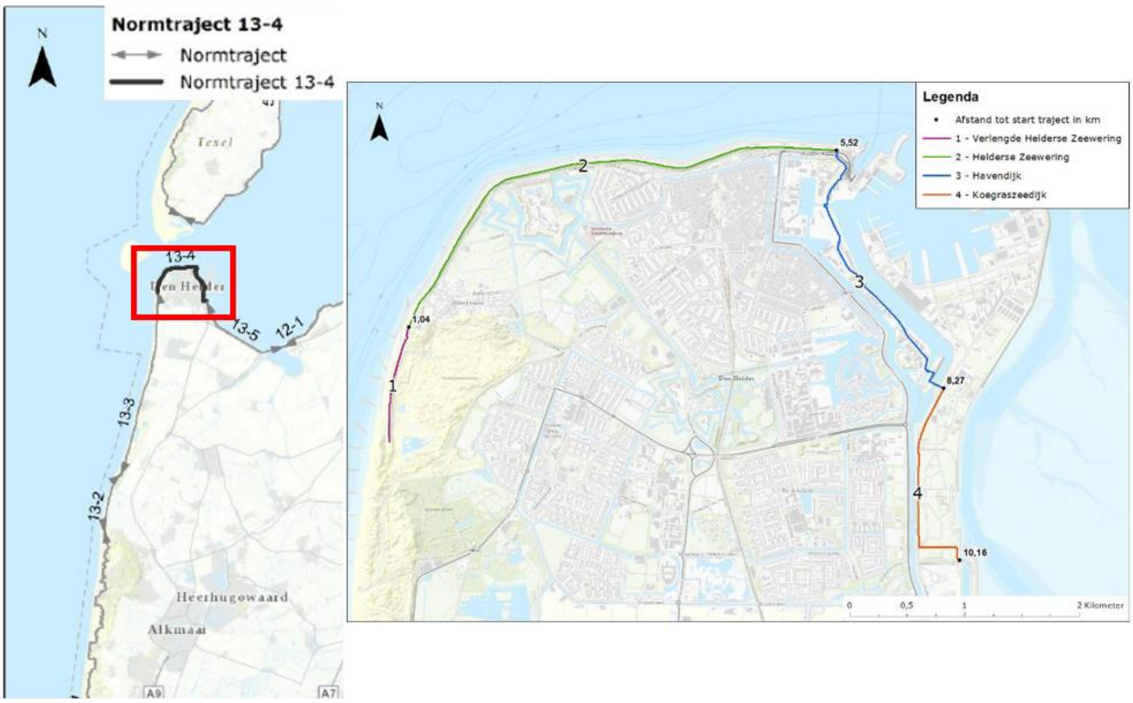

Figure 23: Zooming in on dike section 13-4 (Source: (Hoogheemraadschap Hollands Noorderkwartier, 2021))

The soil of the area consists of Holocene and Pleistocene deposits. The Holocene package is composed of sand, young sea clay, holland peat, old blue sea clay and basic peat from the Naaldwijk and Nieuwkoop formations. Below the Holocene package lie mainly sandy deposits of the Boxtel formation. The deeper lying Pleistocene layers consist of sand, clay and loam layers of the Eem Formation. In general, the soil in the diked area consists of a varying composition of sea clay and sandy layers. Holocene deposits can be found between NAP -5.5 m and NAP +1.5 m near the barrier. The general composition of the dike therefore consists of sand and clay (Hoogheemraadschap Hollands Noorderkwartier, 2021).

5.2. Risk according current assessment methodology

The HHNK's assessment report states that the test assessment for piping and heave for the entire section of dike falls into categories Iv and IIv. With the exception of part of the flood defence along the Nieuwe Diep. This section has been assessed for erosion progression and is in category IVv. The results of the assessment for piping are shown in Figure 24 (Hoogheemraadschap Hollands Noorderkwartier, 2021).

Figure 24: Assesement for piping and heave (STPH) on section level (Source: (Hoogheemraadschap Hollands Noorderkwartier, 2021))

The inspection of the vast majority of the area along the Nieuwe Diep was carried out by consultancy firm HKV Consultancy on behalf of the HHNK. This report has been studied in order to get a better idea of how the assessment was carried out. The failure probability of the Nieuwe Diep flood defence for the piping failure mechanism has been assessed in detail. (Large) outflow openings are present in the structural elements of the retaining wall along the dike and are corroded. As a result, there is no interruption to the seepage path, there is no water safety function with regard to the piping failure mechanism, and these elements have therefore been ignored in the assessment track (Zethof, et al., 2018).

With regard to NWOs, this report only discusses the presence of the brickwork shed at address number 34. This shed is considered a NWO and it is assumed that an excavation of 2 metres will be necessary when the NWO is removed. The analysis therefore includes a scenario in which the NWO is removed illegally (without informing the HHNK) and this is not noticed while normative hydraulic conditions occur. The probability of this scenario is assumed to be 1/2,400 (per year). The geometry has been conservatively assumed at the shed's location, where the ground level is somewhat lower than elsewhere along the dike section. In practice, the shed also transfers its own weight and other loads on the ground there, which means that the safety against erosion will be high enough. At other places the safety against subsidence will also be sufficient due to the higher ground level (Zethof, et al., 2018).

Finally, the report concludes that the assessed stretch is in a tidal zone. In general, these areas are less sensitive to time-dependent mechanisms such as piping. The trajectory does not meet the requirements of the WBI to exclude piping on the basis of time dependence. It is recommended that the time-dependent aspects be investigated further and that the insights on this be improved (Zethof, et al., 2018).

5.3. High Water Protection Programme

The influences of the NWOs present along section 13-4 were not included in the current assessment by the HHNK. The decision not to include them in the assessment was made in accordance with the NWOs fact sheet (Ministerie van Infrastructuur en Waterstaat, 2021). Given the assessment category score of the diked section (category C) it was not necessary to assess the NWOs along this section. However, since the diked section is assessed category C, it will have to be reinforced to meet the safety criteria again. In order to receive financial support from the HWBP for reinforcing the flood defence system, the HHNK must do an “ingangstoets”, or in English: entrance examination. The purpose of this entrance examination is to increase the stability and predictability of this high water protection programme. The programme management of the HWBP assesses new applications and programmes the subvention requests on this basis. This entrance test includes the statutory safety assessment from the WBI assessment report, including the action perspective, combined with the spatial challenges and opportunities in the surrounding area. As NWOs also influence the urgency of the flood risk management, it is important that they are included in the report of the entrance test. A better understanding of the influences of NWOs along dike section 13-4 thus contributes to the completeness of the application to the HWBP.

5.4. Case study location Nieuwe Diep

The determination of the suitable location for the case study was carried out by studying in detail all the properties of all locations within the vicinity of the primary water barrier. For a suitable location, it is important that enough information is available about the foundation and soil structure. This mainly involves looking at the depth of the aquifer and the depth of the foundation. In addition, it is important that there is a reasonable hydraulic head present to cause piping. The first step was to spatially map all NWOs that might have an impact on the safety of the flood defence system.

All buildings located on the flood defence or just at the upstream or downstream side of the flood defence were initially included. Figure 25 shows the NWOs that meet this requirement. At the beginning of section 13-4 beach restaurants Nogal Wiedus & Storm aan zee are located on top of the dike. A few hundred metres further east from there is the Lange Jaap lighthouse. This has been considered because it is in danger of toppling over. In consultation with the HHNK it was therefore decided that the possible consequences of a crater caused by the falling over could be interesting to study. At the most north-eastern end of section 13-4 is Hotel Landsend, which is built halfway into the dike. Finally, the entire section of the dike that runs along the Nieuwe Diep was studied. Several potentially interesting NWObe’s are located along this section.

Figure 25: Overview of potential case study locations (Source: Google Maps)

To determine which of the sites was the most suitable for the case study, a number of criteria were first drawn up. These criteria are shown in Table 4.

Table 4: Criteria to evaluate potential case study locations

Criteria	Expressed in terms of
Location of NWO in relation to the water barrier	foreshore; outer side; crest; inner side; hinterland
Depth of foundation	Meters
Calculated probability of piping	1 / year
Length of seepage path	Meters
Hydraulic head at location	Meters
Depth of piping layer	Meters
Availability foundation data	Good; reasonable; moderate; poor

These criteria were chosen because they determine the relevance of the location for this case study. The location of the NWObe in relation to the water barrier determines whether the structure is located within the zone of influence of the water barrier. The depth of the foundation and with that the availability of that information are important criteria, because the aquifer for piping is located deep below ground level. The currently calculated probability of piping is included to determine whether the potentially negative influence of a NWO makes a difference in an approval or rejection of the dike. The length of the seepage path and the hydraulic head at the NWObe are included, as these are influential parameters within the failure probability calculations. Finally, the depth of the aquifer where piping occurs is included to check whether the foundation of the structure reaches into this layer.

First the existing foundations at the potential locations were studied. For the buildings Nogal Wiedus, Storm aan Zee, Lange Jaap and Hotel Landsend, the available archive documents were requested at the building record office of Alkmaar, which is part of the Alkmaar regional archive. For the trajectory along the Nieuwe Diep, the archive documents for 8 different buildings were requested and studied. The archive documents studied consist of permits and permit modifications, construction drawings, soil investigations and any additional documents. The focus was on studying the construction drawings and soil investigations at the potential locations. For all locations, the depth of the foundations, the original piping calculations and, where available, the soil surveys were studied. In Appendix F: Data Summary Possible Case Study Locations Nieuwe Diep all relevant information of these locations is summarized.

Table 5: Data matrix possible case study locations

Criteria		Nogal Wiedus; Storm aan Zee	Vuurtoren Lange Jaap	Hotel Landsend	Paleiskade 100
Location of NWO in relation to water barrier	[Location]	Crest	Hinterland	Foreshore	Outer side
Depth of foundation	[w.r.t. NAP]	15		-12	-12
Calculated probability of piping	[1/year]	Neglegible	1,25E-10	4,93034E-06	1,66E-07
Length of seepage path	[m]	288,1	213,2	37,2	33,1
Hydraulic head at location	[m]	-2,94	4,94	3,28	2,42
Depth of piping layer	[w.r.t. NAP]	-4,7	-4,44	-4,5	-0,53
Availability foundation data	[Rank]	Reasonable	Poor	Good	Good
		Zwarte Pad SC1	Zwarte Pad SC2	Nieuwe Diep 34	Nieuwe Werk 85
Location of NWO in relation to water barrier	[Location]	Inner side	Inner side	Inner side	Outer side
Depth of foundation	[w.r.t. NAP]	-12	-12		-12
Calculated probability of piping	[1/year]	5,25891E-06	2,61214E-07	5,41143E-05	7,47785E-08
Length of seepage path	[m]	116,3	37,4	67	52,9
Hydraulic head at location	[m]	5,01	3,28	5,01	3,17
Depth of piping layer	[w.r.t. NAP]	-4,8	-4,8	-4,45	-4,1
Availability foundation data	[Rank]	Moderate	Moderate	Poor	Good

In Table 5 the criteria scores for all possible case study locations is summarized in a matrix. After studying the foundation data and construction drawings, Nogal Wiedus, Storm aan Zee and the Lange Jaap lighthouse were rejected as potential case study sites. Based on the data for these locations, it could be assumed that the foundation would not have any interesting influences for this particular study. Subsequently, the locations along the Nieuwe Diep and for Hotel Landsend the subsoil schematics and piping calculations of the HHNK were studied. The piping calculations showed that Hotel Landsend is in fact on the foreshore of the primary flood defence. The location of the entry point would ensure that piping at that location would most likely not occur at the height of the hotel. Therefore, Hotel Landsend is considered not to be an interesting case study location. The archive documents of buildings along the Nieuwe Diep showed that the foundations there reach into the aquifer. Also, most structures are located just in front of or behind the dike and seepage paths are relatively short. In addition, the original calculations by the HHNK already indicated a relatively high probability of piping at the Nieuwe Diep. This combination led to the choice of the Nieuwe Diep as the case study location.

5.5. Piping calculations including NWOs along Nieuwe Diep

For the case study along the Nieuwe Diep, detailed calculations were made for some of the structures present. The knowledge acquired with regard to the influences of the characteristics of a structure have been incorporated in the calculation models of the HHNK. The consequences for the probability of piping at the location of the structures were then examined and this probability was compared with the original probability as calculated by the HHNK. This provided a clear overview of the increase or decrease in the probability of piping near the NWOs along the Nieuwe Diep. The results of the calculations performed in this section can then be compared with the outcome of the flow chart. In section 5.6. Assessment NWOs along Nieuwe Diep with flow chart, the steps according to the event tree leading up to failure will be followed and compared with the results of this section.

5.5.1. Locations

Within the HHNK's calculations, the section along the Nieuwe Diep is divided into small pieces for which separate calculations are made. For the case study, a number of properties were chosen for which relevant information regarding the foundation was available. Subsequently the HHNK calculations for those sections of the dike were used as a basis for the case study. Figure 26 shows the used locations along the Nieuwe diep.

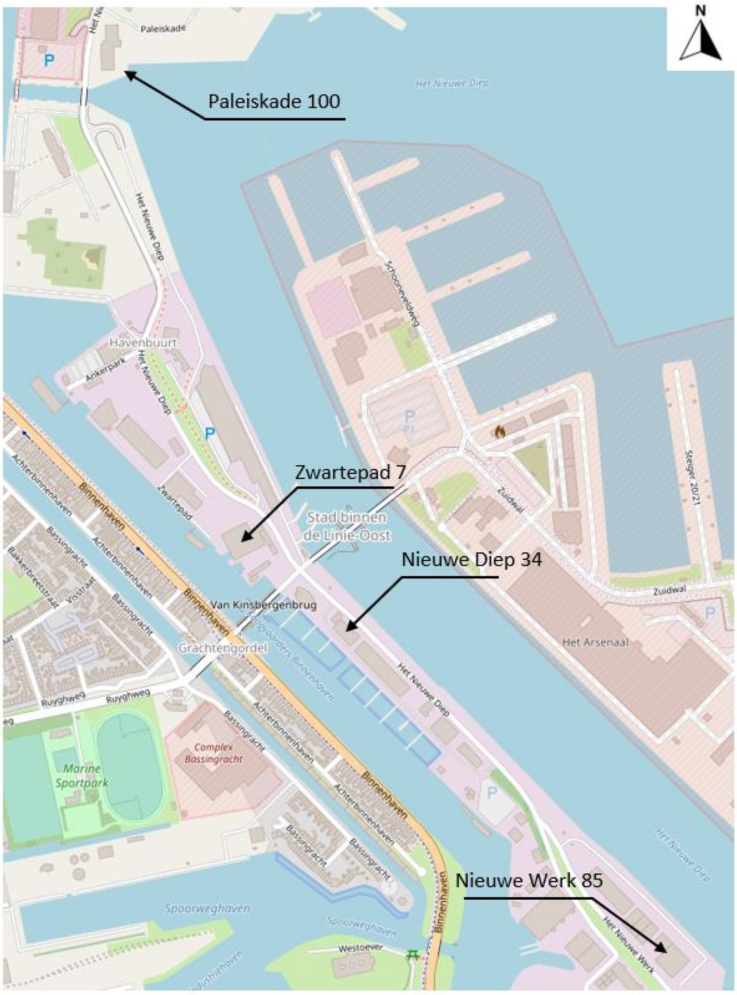

Figure 26: Used locations along Nieuwe Diep for case study calculations (Source: openstreetmap.org)

5.5.2. Hydraulic boundary conditions

In the calculations of the HHNK, the database *WBI2017_Waddenzee_West_13-4_v03* advised by Helpdesk Water has been used to determine the hydraulic loads. Typical values for wave heights and water levels at the signalling value (category boundary A) and lower limit (category boundary B) for the North Sea and Wadden Sea based on marginal statistics are given in Appendix G: Hydraulic Boundary Conditions Section 13-4. However, the database does not give conditions for locations in the ports. Therefore, in order to be able to determine hydraulic loads for the primary barrier along the Nieuwe Diep, the HHNK generated new output points. Using the HB Havens software, output points have been added to the existing database. In order to determine the hydraulic loads, the harbour is schematized in terms of harbour areas, harbour dams and water depths. The hydraulic boundary conditions as stated in the calculations of the HHNK are summarised in Table 6.

Table 6: Hydraulic boundary conditions for locations along the Nieuwe Diep (Source: (Hoogheemraadschap Hollands Noorderkwartier, 2021))

	Paleiskade	**Zwarte Pad**	**Nieuwe Diep**	**Nieuwe Werk**
Water level [NAP+m]	4,33	4,33	4,33	4,33
Hydraulic head [m]	2,42	3,28	5,01	3,17
Head in 1st aquifer [NAP+m]	3,92	4,08	4,39	2,91
Water level inland [NAP+m]	1,91	1,05	-0,68	1,16
Damping factor [-]	0,83	0,92	1,01	0,55

5.5.3. Scenarios

Various possible combinations of influential characteristics at a structure were devised for the calculations. These combinations (hereafter referred to as scenarios) are divided into so-called *generic* scenarios and *site-specific* scenarios. The generic scenarios are applied in the same way for all 4 locations within the calculations. The location-specific scenarios depend on unique characteristics per location. For each scenario used, a short motivation and explanation of how it is applied within the calculations will be given.

Generic scenarios

The generic scenarios are intended to give a clear picture of the general influence per characteristic feature of a structure. In the scenarios, a distinction is therefore made between the characteristics as shown in Table 7.

Table 7: Summary generic scenarios Nieuwe Diep

Scenario	**Characteristic**	**Effect**	**Incorporation in calculations**
0	Original HHNK calculation	-	-
1	Pile foundation	Hydraulic fracture and flow along piles	Shorter seepage path
2	Basement/ cellar	Barrier effect	Increase of hydraulic head
3	Excavation of foundation	Excavation	Decrease of soil pressure
4	Digging foundation & pile foundation	Excavation and hydraulic fracture at piles	Decrease of soil pressure and shorter seepage path
5	Digging foundation & cellar & pile foundation	Excavation, barrier effect and hydraulic fracture at piles	Lower soil pressure, bigger hydraulic head and shorter seepage path
6	Shallow foundation	Structures weight acting on ground level	Higher soil pressure

Site-specific scenarios

For the site-specific scenarios, the original seepage path, the exit point and the present cover layers were carefully considered. For each location a plausible new exit point with a foundation on piles was considered. This new exit point was then included in the calculations and three scenarios were calculated per location. Table 8 shows a summary of the location specific scenarios. It is important to note that for the site-specific scenarios only the exit point of the seepage path differs. The incorporation of the other effects in the parameters is the same as for the general scenarios. It is important to note that for the site-specific scenarios only the exit point of the seepage path differs. The incorporation of the other effects in the parameters is the same as for the generic scenarios.

Table 8: Summary location specific scenarios Nieuwe Diep

Scenario	**Characteristic**	**Effect**	**Incorporation in calculations**
7	Pile foundation	Hydraulic fracture at piles below structure	Removal exit point seepage path to approximate location structure
8	Digging foundation & pile foundation	Excavation and hydraulic fracture at piles below structure	Lower soil pressure and removal exit point seepage path to approximate location structure
9	Digging foundation & cellar & pile foundation	Excavation, barrier effect and hydraulic fracture at piles below structure	Lower soil pressure, bigger hydraulic head and removal exit point seepage path to approximate location structure

5.5.4. Calculations

All described scenarios have been incorporated in the original calculations of the HHNK by adjusting the parameters. It is briefly described how all effects from the scenarios have led to adjustments of the parameters of the calculations. The incorporation of the effects in the parameters of the piping calculation is summarized in Table 9.

Table 9: Summary incorporation of effects in parameters of piping calculations

General scenarios		
Effect	**Parameter**	**Parameter change**
Hydraulic fracture at piles(horizontal shortening over seepage length)	Exit point pipe	- 5 metres
Barrier effect (increase hydraulic head difference)	Water level inland	- 0.1 metres
Foundation excavation	Thickness covering layer	- 0.8 metres
Cellar / basement excavation	Thickness covering layer	-4 metres
Shallow foundation	Pressure covering layer	+ 14.7 kN/m^2
Location specific scenarios		
Hydraulic fracture at piles of structure	Exit point pipe	Move exit point to location of structure

In order to determine how a difference in seepage path length due to piling affects the probability of piping, the seepage path in the general scenarios has been shortened by 5 metres. Obviously this is very location dependent and the seepage path can also be longer in case the original exit point is in front of the building. In Figure 27 is shown how moving the exit point of the pipe to the pile location can influence the seepage path length. The seepage path may become either longer or shorter, depending on the original location of the exit point. In the site-specific calculations, the exit points are moved to the location of the building in question. In most cases this results in an extended seepage path, but at one of the locations it results in a shorter seepage path.

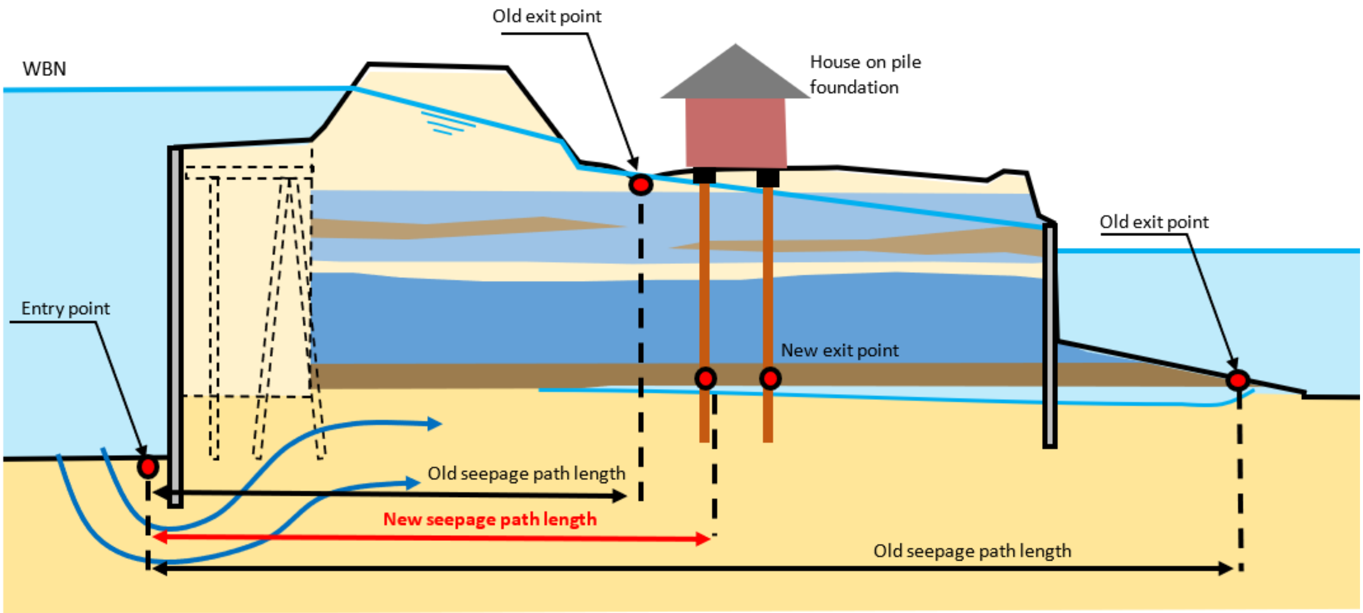

Figure 27: Influence of moving of exit point pipe to pile location on seepage path length

The barrier effect that can occur in the presence of a basement or cellar has been taken into account by lowering the water level inside the dike by 0.1 m (Fugro, 2010). This quantity was deemed reasonable and assumed based on various reports by Fugro regarding the barrier effect. It could be assumed that a barrier effect may occur at the location of the Nieuwe Diep, as available groundwater monitoring data showed a significant hydraulic decline at high water levels. A schematization of the barrier effect on the piping calculation parameters can be seen in Figure 28 (a). For the excavation of a foundation, part of the covering layer at the exit point was omitted from the calculations. It is assumed that an excavation of 0.8 metres (Vree, 2022) is carried out for the foundation. When a cellar or basement is excavated the thickness of the covering layer at the exit point is decreased by 4 meters. The influence of foundation and basement/ cellar excavation on the piping calculation parameters can be seen in Figure 28 (b). In case of a shallow foundation, an extra covering layer representing the pressure of the structure, transferred through the footing of the shallow foundation is added in the calculations. According to (redactie Exaktueel, 2021) an average house is 7 m long, 7 m wide and 6 m high, and glass, floors and internal walls of such a house have a combined mass of 60,000 kg. This means that a house weighs about 1225 kg per square metre. A small margin has been kept for the conversion to kN and 1500 kg per m^2 has been assumed. This covering layer exerts a pressure of 14.7 kN/m^2 at the location of the exit point. The influence of a shallow foundation on the piping calculation parameters can be seen in Figure 28 (c). In Appendix H: Piping Scenarios Calculations, an overview of the performed calculations per location can be found.

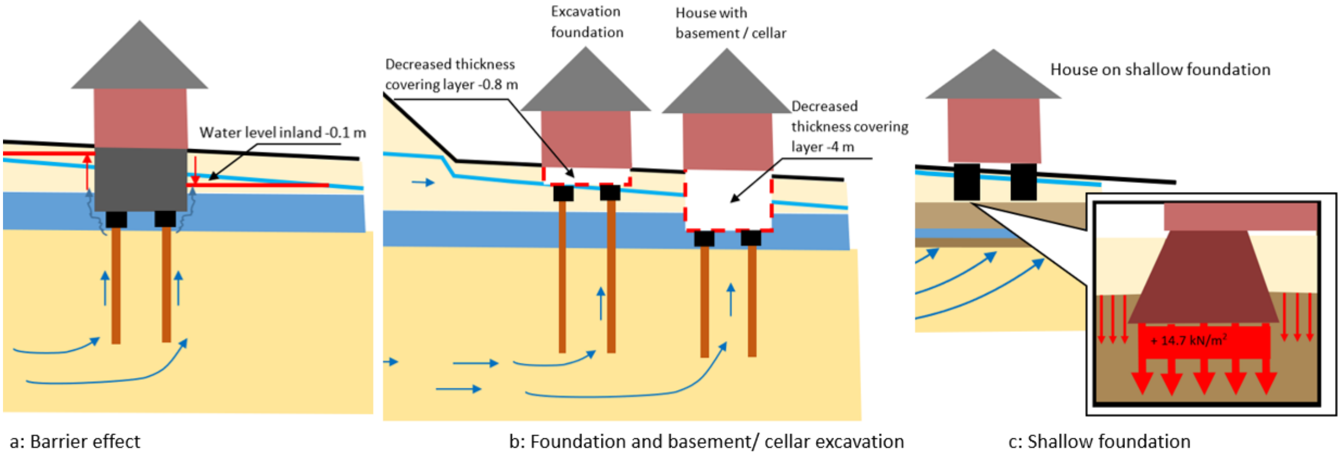

Figure 28: Influence barrier effect (a); foundation and basement/ cellar excavation (b) and shallow foundation (c) on piping parameters

5.5.5. Results

After calculating all scenarios for the various locations, an overview could be made of the effects on the probabilities of the various failure mechanisms and thus the overall probability of piping. An overview of all results per location and for all scenarios can be found in Appendix G: Piping Scenarios Results.

In Table 10 one can see an overview of the results at Nieuwe Diep 34. A conditional formatting has been added to the cells in excel to indicate whether the probability is greater or less than the mean. The 0 scenario indicates the probabilities as they are in the calculations of the HHNK. It is striking that the probability of the sub mechanisms hydraulic fracture and heave are in all cases considerably higher than the probability of backward erosion. In the first scenario, it becomes clear that the shortened seepage path only affects the probability of backward erosion and has a major impact. Subsequently, scenarios 2 to 4 show that the various combinations with excavation of the foundation, a basement and pile foundation influence all sub mechanisms. Scenario 5 stands out because the probability of hydraulic fracture and heave have increased significantly here, but also the probability of backward erosion has increased strongly due to the combination of factors. In the location-specific scenarios, the significant increase in the probability of backward erosion is also striking. This is caused by the shorter seepage path at the Nieuwe Diep. In the worst case scenario 9 with a basement and pile foundation, the probability of piping is even increased by 4.29^{-3}.

Table 10: Scenario results summary Nieuwe Diep 34

Scenario summary Nieuwe Diep 34	Scenarios				
Sub mechanism	**0**	**1**	**2**	**3**	**4**
Hydraulic fracture	1,57E-02	1,57E-02	1,77E-02	7,25E-02	7,25E-02
Heave	7,56E-02	7,56E-02	8,25E-02	1,51E-01	1,51E-01
Backward erosion	5,41E-05	1,14E-04	7,23E-05	1,07E-04	2,19E-04
Piping overall	5,41E-05	1,14E-04	7,23E-05	1,07E-04	2,19E-04
Scenarios	**5**	**6**	**7**	**8**	**9**
Hydraulic fracture	1,00E+00	4,75E-03	1,57E-02	7,25E-02	1,00E+00
Heave	9,97E-01	6,36E-02	7,56E-02	1,51E-01	9,97E-01
Backward erosion	1,94E-03	4,51E-05	1,42E-03	2,45E-03	1,26E-02
Piping overall	1,94E-03	4,51E-05	1,42E-03	2,45E-03	1,26E-02

5.6. Assessment NWOs along Nieuwe Diep with flow chart

In this section, the developed flow chart will be applied to the different properties along the Nieuwe Diep to see if it leads to logical decisions with regard to the testing of the NWO.

5.6.1. Influence zone NWO on STPH

Before proceeding to the assessment of the structures along the New Diep, it is important to look at the zone of influence of the NWOs. In Figure 29 the zones of influence of a NWO per direct failure mechanism are summarised. From this it can be concluded that the zones of influence for piping (STPH) are on the foreshore and just behind the dike.

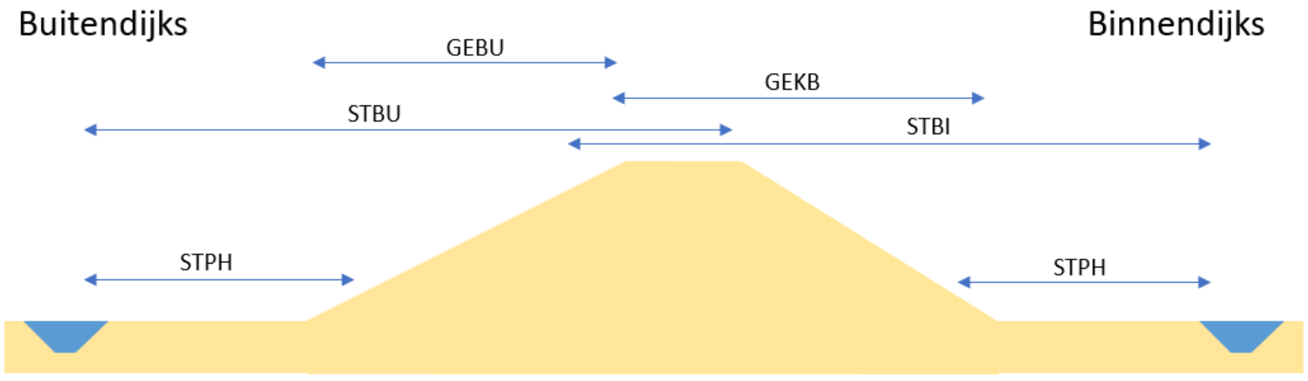

Figure 29: Overview of NWO influence zones per direct failure mechanism (Source: (Leau & Kames))

For the properties in the case study, Zwarte Pad 7 and Nieuwe Diep 34 are both located behind the dike. Paleiskade 100 and Nieuwe Werk 85 are both on the foreshore of the dike. All buildings are therefore within the piping influence zone. For the development of the flow chart, the focus in this study was only on the effects at a structure behind the dike. It is not known if the effects are applicable to a building on the upstream side of the dike. Since no archive data were available for the buildings near Paleiskade and Nieuwe Werk just behind the dike, the data for the buildings on the upstream side of the dike were searched. But for the application of the flow chart, it is assumed that there is a comparable building behind the dike.

5.6.2. Assessment with flow chart

Paleiskade

The building on Paleiskade whose archive documents were requested is located just in front of the dike. For the case study, the focus was on a building just behind the dike, of which no archive documents could be obtained. For the building on the Paleiskade, the archive documents show that there is no substructure below the building. Therefore, the effects related to the substructure do not pose a risk and do not need to be included in the calculations. With regard to the foundation, it was found that the building rests on a pile foundation. The piles have a length of 15 metres and the piles are driven till approximately 12 metres below NAP. Since the aquifer is located already at 2 metres below NAP, the piles therefore cross the aquifer. The geotechnical profile shows that in some places there is no clay layer thicker than 450 mm behind the dike. It can therefore be assumed that the pile foundations could influence the length of the seepage path. However, the exit point in the original calculations by the HHNK is located directly behind the dike. The property, on the other hand, is located further behind the dike than the original exit point and this would mean that the seepage path becomes longer when the exit point moves up to the pile foundation. This will not have a negative effect on the likelihood of backflow erosion and therefore does not constitute a risk. Lastly, there is no crawl space under the building, the concrete floor is poured directly on the piles. Possible influences of a crawl space are therefore not considered.

Zwarte Pad

The building on the Zwarte Pad is an industrial hangar. No indications of the presence of a substructure were found in the archive documents. The hangar stands on a pile foundation which, based on a cone penetration test and the foundations of surrounding buildings, is assumed to go to approximately 12 metres below NAP. The aquifer is located at approximately 5 metres below NAP and is therefore penetrated by the piles. However, the geotechnical profile shows that the clay layer is on average more than 1 metre thick and therefore it can be assumed that it will not be perforated by the piles. In the original calculations by the HHNK, two scenarios were also included in which the exit point first lies in the Koopvaarders Binnenhaven and in the second scenario just behind the dike. Therefore, should the exit point be at the property, this scenario already falls within the scenario calculations of the HHNK. The archive documents indicated that the concrete floor was poured on the piles, which means that there is no crawl space.

Nieuwe Diep

The building at Nieuwe Diep 34 is a wide industrial hangar. No traces of a possible substructure have been found in the archive documents for this building. It is interesting to note, that if a cellar had been present, this building might have caused the barrier effect because of its perpendicular orientation to the direction of the waterflow. This in combination with the substantial width of the building makes it an interesting location. However, there is no substructure and therefore no need to go through this branch of the flow chart. There is, however, a pile foundation under the building which is driven in the ground up to approximately 12 metres below NAP. This aquifer is located here at about 5 metres below NAP and is therefore crossed by the piles. In most places there is a clay layer of at least 2 metres, but in a small section this clay layer is interrupted and is thinner than 450 mm. Because there are many piles under the building, it is likely that some of the piles here perforate the clay layer. The original exit point in the calculations of the HHNK is located in the Koopvaarders Binnenhaven. When the exit point is moved to the pile foundation a shorter seepage path will be created, resulting in a higher probability of backward erosion. It is therefore recommended that additional calculations are made with a shorter seepage path. The archival documents have not shown any indications of the presence of a crawl space, and this branch of the flow chart will therefore not be followed.

Nieuwe Werk

The Nieuwe Werk building is located just before the dike on the quay. For the buildings just behind the dike, no foundation data was found in the archives. It is assumed that the foundation of the Nieuwe Werk 85 is comparable to the buildings just behind the dike. For the use of the flow chart, it is assumed that the building is located just behind the dike. No indications have been found that the building has any kind of substructure. Therefore, this branch of the flow chart is not considered. The archive documents show that the buildings here stand on 15.5 m long piles which are driven in the soil up to 12 m below NAP. These piles therefore cut through the piping layer located at -5 metres NAP. The geotechnical profile shows that the clay layer behind the dike is very thin. In some places less than 450 mm, so it is assumed that it is perforated by the pile foundation. The exit point in the original calculations by the HHNK is located just behind the dike. The property is located further downstream, so the seepage path would increase in this case if the exit point were moved to the building. It is therefore assumed that the pile foundation has no further negative impact. Finally, no indications of a crawl space were found under the property and therefore this branch was not continued.

5.6.3. Flow chart outcome comparison with scenario calculations

The purpose of the flow chart is to contribute to the completeness of the assessment of the dike by including the possible influence of a structure. It is therefore important to check now whether the flow chart has led to the right decisions for the NWOs along the Nieuwe Diep. To check this, the scenario calculations for all locations were looked at again. The results of these calculations were compared with the flood probability standards for route 13-4 as shown in Table 11.

Table 11: Flood probability standards section 13-4 (Source: (Hoogheemraadschap Hollands Noorderkwartier, 2021))

Norm section	Signalling value	Lower limit
13-4	1/3,000 [per year]	1/1,000 [per year]

Paleiskade

According to the flow chart, no additional calculations are required for the building on the Paleiskade. In the results of the scenarios, the structure does not cause the signalling value to be exceeded in any case. Even in the worst case scenario, the probability does not exceed 1/4854 per year. The conclusion of the flow chart is therefore in line with the results of the scenarios.

Zwarte Pad

According to the flow chart, no additional calculations are necessary for the hangar along the Zwarte Pad. The scenario results do not show a probability greater than 1/1441 per year for this structure. This means that the signalling value is exceeded and this is close to the lower limit. Nevertheless, it can be concluded that the flow chart has led to the right decision. The probability of 1/1441 per year is in the worst case scenario where there is a basement under the building and the exit point moves further upstream. For Zwarte Pad however, both of these are not the applicable and if one looks at the scenario that comes closest to the true situation, the probability is only $1/3.5^8$ per year. Therefore, the conclusion of the flow chart that no additional calculations are necessary is correct.

Nieuwe Diep

At the Nieuwe Diep 34 property, the flow chart concludes that additional calculations are required to determine the influence of a shortened seepage path due to the presence of the property. The scenario in the calculations that most closely matches the true conditions is scenario 8. In this scenario the exit point is moved to the estimated location of the building in relation to the dike. In addition, the decrease in soil pressure due to digging the foundation is included in this scenario because the building is on piles. The probability of failure in this scenario is 1/408 per year, which is well above the lower limit. It can therefore be concluded that the flow chart in this case has led to the right choice of additional calculations. Based on this result, it can be decided to perform an expert assessment for this building.

Nieuwe Werk

In the case of the premises at Nieuwe Werk 85, the flow chart judges that no additional calculations are necessary. The scenario results show that in the worst case scenario the probability of piping is equal to 1/5650 per year. This means that even in the worst case scenario with a basement and shortened seepage path the structure will still not exceed the warning value. The flow chart therefore leads to a logical conclusion for this location.

6. Discussion

The case study showed that the flow chart generally works well for assessing the influences of structures and when they should or should not be included in the assessment. However, there are still limitations that need to be mentioned.

It is difficult to determine exactly when piles cause perforation of the existing clay layer. Perforation is currently assumed when the diameter of the pile / thickness of the clay layer is 1:1 or when the diameter of the pile is greater than the thickness of the clay layer. This is based on the scale tests in (Hird, Emmett, & Davies, 2006). However, the soil displacement as a result of driving in or driving out piles strongly depends on local factors. Because the exact deformation depends on the strength of the different layers, this is difficult to predict in practice. In addition, the scale tests also showed that the different ways in which the piles are installed influence the magnitude of the impact. Depending on the method used, the clay layer may or may not be perforated.

With regard to the perforation of the clay layers, it is now only assumed that this will influence the length of the seepage path. Currently, the piles therefore only increase the risk of backward erosion at the NWO. Earlier calculations considered increasing the permeability coefficient of the aquifer. However, during the expert session with the experts of Deltares it emerged that the perforation only affects the vertical permeability of the clay layer and not the permeability of the aquifer. Within the Sellmeijer formula, the only resistance from vertical seepage now comes from the 0.3 x thickest layer rule. It is likely that the piles have an influence here. Within the current calculation model of the HHNK an increase of the vertical permeability of the clay layer could not easily be taken into account. Therefore, this effect was not further investigated in this study. However, it can be assumed that a pile foundation also affects the vertical permeability and thus the likelihood of a pipe growing vertically.

At the moment, it is not really clear what exactly happens when a pipe ends up in the crawl space of a structure. This effect has been extensively discussed in expert sessions with the experts of both Deltares and the HHNK. However, it is very difficult to predict with any certainty how this process will develop. Several scenarios have been considered. When the pipe along a pile ends in the crawl space, it is likely that the sediment will settle here for a considerable time. When the crawl space is subsequently filled with the sediment, several things can occur. Firstly, the pipe can now no longer deposit its sediment in the crawl space and must follow a new path. This means that the pipe must work its way both horizontally and vertically along the foundation of the building to a place next to the structure. This requires a lot of pressure in the pipe and the question is whether this will occur. In addition, the hydraulic head in the crawl space decreases as more water and sediment fill the crawl space. It is therefore also possible that the backward erosion process will be stopped sooner.

It is difficult to determine whether or when the barrier effect that can be caused by the presence of a basement or cellar will occur in the vicinity of a NWO. It is often difficult to know whether all conditions for the occurrence of this effect are met. This requires extensive knowledge about the soil structure and the flow of the groundwater along the structure. In the management reports of Fugro concerning the barrier effect, it often appeared that the barrier effect did not occur at the substructure under investigation. The main reason for this was often that there was insufficient local hydraulic head of the phreatic groundwater level at the location of the object. In the event of the case study, there was sufficient groundwater monitoring data available to determine whether there was sufficient difference in the phreatic groundwater level for the barrier effect to occur along the Nieuwe Diep. However, even though it could be assumed that sufficient hydraulic head can occur along the Nieuwe Diep, it is uncertain whether the hydraulic head lasts long enough for the barrier effect to occur.

Another point of discussion is what exactly happens when a pipe reaches a sandy top layer. It often happens that there is a sandy layer underneath the structure, which is then separated from the aquifer by a clay layer. This is also the case at several locations in the case study. When the clay layer fractures, two scenarios can occur. Firstly, it is possible that the pipe works its way vertically upwards, for instance along the pile under a structure. The pipe then ends under the building in the crawl space or right next to the building. However, it is also conceivable that the pipe will try to deposit sediment in the sand package when it has burst through the clay layer. When the pressure in the pipe is not great enough to continue growing vertically, it is likely that the process will then also stop in the top layer of sand.

Finally, the focus of this study was mainly on identifying negative influences of a structure on piping. The only effect that contributes to the safety of the water barrier, that was found in this study, is the effect of a shallow foundation. It is therefore important to mention that there are probably also positive influences, that could make the negative influences identified in this study have less impact on the probability of failure.

7. Conclusion

This research was conducted to answer the main research question:

How can the influential characteristics of a structure on piping be translated into a flow chart describing how to approach the safety assessment for piping of a levee including a structure?

This question was split up in three sub-questions:

7.1. Which characteristics of a structure have an influence on piping and how do these characteristics influence the piping failure mechanism?

It has been found that a structure can exert influences on the piping failure mechanism both positive and negative due to conditions within the subsoil and the different possible configurations of the structure and foundation. The most important characteristics of a structure that emerged from this study were the foundation and any substructure. Within these characteristics a distinction is made between a foundation on piles and a shallow foundation, because both influence the subsoil in different ways. In addition, for the substructure a distinction is made between no substructure, a crawl space, and a basement or cellar.

After determining which characteristic features of a structure influence the geohydrologic conditions in the subsurface and thus the sub-mechanisms of the piping failure mechanism, it was investigated how these characteristics influence the different sub-mechanisms. Literature research has shown that the driven piles of a structures foundation can cause perforation of the clay layer that separates the overlying sand layers from the aquifer sand layer. Such a perforation depends on the local soil composition, but when there is a pile diameter/clay layer thickness ratio of 1:1 or greater, it is likely that the clay layer no longer separates the aquifer from the top sand layer at the location of the piles. This may cause the exit point of the seepage path to move towards the location of the structure and the pipe to develop vertically more easily. The effect of a hydraulic fracture at a shallow foundation is limited due to an increase in overburden pressure due to the weight of the structure that rests on the foundation. Because this type of foundation commonly lies on the top bearing sand layer, the overburden pressure increases which reduces the risk of piping. With regard to the substructure, it has been found that a cellar or basement can cause the so-called barrier effect. This effect can occur when groundwater can no longer flow unhindered along or under the structure. When the barrier effect occurs, the phreatic groundwater level is raised on the downstream side of the structure. On the upstream side of the structure, the groundwater level drops, resulting in an increase of hydraulic head when the hydraulic fracture occurs on this side of the structure. Finally, the influence of a crawl space under the structure was examined. When the erosion takes place in the crawlspace and the backward erosion causes sediment to be deposited underneath the structure, it is difficult to predict how this process will continue or end. Several options are possible, but it remains likely that the piping process will eventually stop below the structure. The exact impact remains dependent on how much sediment can be deposited in the crawlspace in the meantime. It is important to note that the influence of the structure is in all cases highly dependent on local conditions.

7.2. How can the influential characteristics of a structure be translated into criteria, that together form a flow chart, describing the approach to the safety assessment for piping of a levee including a structure?

Once the effects of the characteristics of a structure were mapped, criteria were drawn up for these characteristics to be able to decide when the effect takes on and should be included in the calculations for the piping failure mechanism. These criteria are also split up per characteristic part of a structure. The effects of a pile foundation will have an influence if the pile foundation reaches the depth of the aquifer and perforates a clay layer as thick as the diameter of the pile or if the clay layer is thinner than this. The effects of substructure are twofold. First, the barrier effect can occur if all three criteria for this effect are met. This means that the substructure must cover at least 70% of the area of the sand layer above the sealing clay layer. If this criterion is met, there must also be a hydraulic head in the phreatic groundwater of at least 0.1 m. And the last criterion is that the orientation of the substructure must be perpendicular to the direction of the groundwater flow. Second, there is the possibility that the substructure causes a decrease in the soil pressure when the building is built on a pile foundation. Lastly, there is the possible influence of a crawl space. Although the effects of a crawl space are still relatively uncertain, it is important that they are included in the risk assessment. If the structure stands on a pile foundation that meets the previously mentioned conditions for hydraulic fracture in a pile foundation, it can be assumed that hydraulic fracture will occur in the crawl space. It is important to note that the criteria listed do not represent a complete solution, additional criteria may be conceivable.

The criteria were then linked to the three sub-mechanisms of the piping process. The existing event chain for piping is used as a basis for the flow chart. By first linking influential characteristics to this event chain and then linking the earlier developed criteria with these characteristics, the flow chart was developed.

7.3. How can the developed flow chart be applied in a case study for the assessment of a levee including a structure?

The developed flow chart was applied to a case study for dike section 13-4 and specifically for buildings along the Nieuwe Diep. By performing additional calculations for a number of structures along the Nieuwe Diep it was determined how the probability of piping is influenced by certain characteristics of a structure. Based on the results of these additional calculations it was investigated whether the flow chart would lead to logical results when applied in an assessment of these structures. By examining whether the calculated failure probability per scenario exceeded the flood probability standards and comparing this with the outcome of the flow chart, it became clear that the flow chart gave a logical conclusion for all structures in the case study. In three of the four cases, the flow chart showed that no additional calculations were necessary. In these cases, the flood probability standard was not exceeded in the results of the scenarios. In the single case where the flow chart indicated that additional calculations were required, the scenario result showed that the failure requirement was exceeded by a wide margin.

In summary, the influential characteristics of a structure on piping can make a positive contribution to the assessment of a dike, by incorporating them into a flow chart. By determining the influential characteristics of a structure on the failure mechanism and translating them into criteria, such a flow chart can be developed. Combining existing event chains for failure mechanisms with the developed criteria leads to a well-structured flow chart. By means of a case study, it can then be verified whether the flow chart actually leads to logical judgements or whether additional criteria are necessary. When the flow chart is provided with recommendations for additional calculations when the structure is found to have a negative influence, the chance that possible negative influences of structures are not included in the assessment process is reduced. This ensures a more complete and reliable safety assessment of the dike.

8. Recommendations

This study was carried out with the aim of providing a first step towards a stepwise plan for the assessment of a levee including a structure. Several assumptions have been made in this study. To get an improved overview of the influences of a structure on the probability of piping it is recommended to do further research on a number of things.

In this study, the focus has been on the general influences of a structure on the piping failure mechanism and whether or not the assessor should consider further calculations because of this influence. However, there are several types of NWOs and failure mechanisms that can occur in various combinations at a primary water barrier. It is therefore recommended that further research is conducted into the impact of other types of NWOs in order to arrive at a complete test assessment. This means considering the influences of NWOs such as trees, roads, cables and pipes. In addition, for the completeness of the test assessment it is important to investigate the influences of all these NWOs on the other relevant failure mechanisms. However, it will always be difficult to include a generic flow chart for all forms and combinations in one diagram. This will therefore always remain dependent on a certain degree of customisation.

Second, it is recommended to further consider how the location of a structure relative to the dike affects the probability of the development of a pipe. Think of the foreshore, crest, inside and behind the dike. The focus of the scenarios in the case study along the Nieuwe diep was on structures located behind the dike. Because the exit point of the pipe is in most cases just behind the dike, the effect of the structure was often not of great negative influence. However, it is conceivable that when the structure is located on the crest or just behind it that this influence is stronger.

With respect to the effect of driving in piles, much additional research can be done and is highly recommended. It would be very valuable to do a study on a true scale with different types of piles. The main distinction is now only made between soil-driven and non-soil-driven piles. However, there are many types of piles and techniques by which these piles can be precisely applied in the subsurface. By conducting further research into the various effects of different piles and techniques, it can quickly be determined for each pile type to what extent it affects the subsurface and thus the piping failure mechanism. In addition, by carrying out the research at full scale it can be determined whether the effects described in (Hird, Emmett, & Davies, 2006) also occur on true scale and whether the disturbance area of the subsurface around the piles then grows or shrinks. It is also recommended to measure any increase in groundwater flow along the piles. With this knowledge it can be determined whether the flow increases enough for the growth of a pipe through which the sand can be transported. In addition, additional research can determine whether seepage along the piles possibly affects the previously mentioned 0.3 x thickest layer rule used in Sellmeijer.

It is recommended to further expand the flow chart and associated criteria. The current flow chart has the occurring sub-mechanisms as a basis and the sub-mechanisms are linked to common characteristics of the structure that influence that sub-mechanism. However, the criteria that must be met to assume that a particular occurs are currently quite general and subject to multiple assumptions. If it is now decided that it is plausible that a certain effect occurs as a result of the structure, data must be collected in order to then perform additional calculations. After this, based on the possible increase or decrease in the probability of piping, it must be determined whether an expert assessment should be done at the structure. If the criteria in the flow chart are better elaborated and subject to fewer assumptions, this ensures a more reliable outcome of the safety assessment of a levee including a structure.

Additional research for the development of an improved flow chart, and/or flow charts for other failure mechanisms and NWOs, should not be carried out by one water board. It is recommended that experts from water boards across the country work together so that all local dilemmas are considered in the process. A cost-benefit analysis could show whether such a full-scale study would provide enough information to possibly prevent numerous dike reinforcements. Dike reinforcements in built-up areas are often much more expensive than reinforcements in rural areas. A cost-benefit analysis would therefore be very valuable.

Lastly, it is recommended that additional geohydrological research is carried out with regard to the exit point in the crawl space. It is currently unclear what exactly happens when the exit point of the pipe is in the crawl space. Possible scenarios were discussed earlier in this report. Additional research can clarify whether one or more of these scenarios are possible, and whether they are still dependent on more factors at the location of a structure. If there is a better picture of the possible (positive and/or negative) influence of a crawl space, this can also be incorporated in the flow chart.

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Appendix A: Schematical overview of assessment process

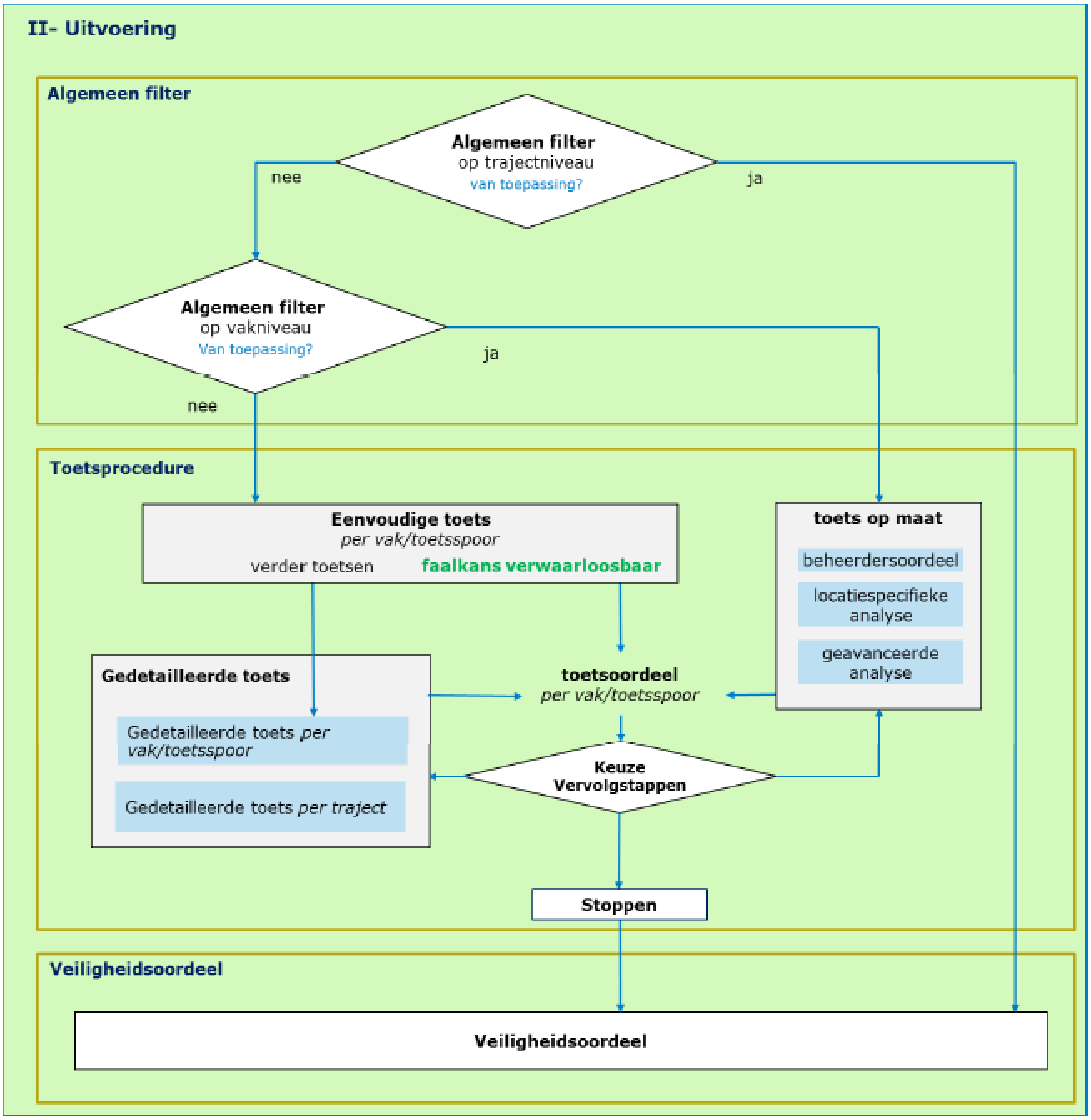


Figure 30: Schematization of assesment process (Source: (Rijkswaterstaat, 2017))

Appendix B: Stability Factors Piping Sub Mechanisms

Appendix B.1. Stability factor hydraulic fracture

The stability factor for hydraulic fracture is given by (Rijkswaterstaat, 2017):

$$F_u = \frac{\Delta\phi_{c,u}}{\Delta\phi} \tag{B.1}$$

Where:

- F_u [-] : Calculated stability factor for sub mechanism hydraulic fracture
- $\Delta\phi_{c,u}$ [m] : Critical head difference over the top layer (at exit point)
- $\Delta\phi$ [m] : Head difference occurring over the top layer (at exit point)

The critical head difference over the top layer is given by (Rijkswaterstaat, 2017):

$$\Delta\phi_{c,u} = \frac{D_{deklaag}(\gamma_{sat}-\gamma_{water})}{\gamma_{water}} \tag{B.2}$$

Where:

- $D_{deklaag}$ [m] : thickness of the cohesive layer
- γ_{water} [kN/m^3]: volumetric weight of water
- γ_{sat} [kN/m^3]: saturated volumetric weight of cohesive layer

The head difference occurring over the top layer is given by (Rijkswaterstaat, 2017):

$$\Delta\phi = \phi_{exit} - h_{exit} = (h - h_{exit})r_{exit} \tag{B.3}$$

Where:

- ϕ_{exit} [m] : Head in aquifer at exit point compared to NAP
- h_{exit} [m] : Phreatic level, or height of ground level, at exit point relative to NAP
- r_{exit} [-] : Damping or response factor at exit point
- h [m] : Level of the outside water level in relation to NAP, with a probability of occurrence equal to the norm

Appendix B.2. Stability factor heave

The stability factor for heave is given by (Rijkswaterstaat, 2017):

$$F_h = \frac{i_{c,h}}{i} \tag{B.4}$$

Where:

- F_h [-] : Calculated stability factor for sub mechanism heave
- $i_{c,h}$ [-] : Critical heave gradient
- i [-] : Calculated heave gradient

The heave gradient, the load component, is given by (Rijkswaterstaat, 2017):

$$i = \frac{(h-h_{exit})r_{exit}}{D_{deklaag}} \tag{B.5}$$

Where:

- h_{exit} [m] : Phreatic level, or height of ground level, at exit point relative to NAP
- r_{exit} [-] : Damping or response factor at exit point
- $D_{deklaag}$ [m] : thickness of the cohesive layer
- h [m] : Level of the outside water level in relation to NAP, with a probability of occurrence equal to the norm

Appendix B.3. Stability factor backward erosion

The stability factor for backward erosion is given by (Rijkswaterstaat, 2017):

$$F_p = \frac{\Delta H_c}{(h - h_{exit} - r_c D_{deklaag})} \quad \text{(B.6)}$$

Where:

- ΔH_c [m] : Critical head difference of flood defence
- F_p [-] : Stability factor for backward erosion
- h_{exit} [m] : Phreatic level, or height of ground level, at exit point relative to NAP
- $D_{deklaag}$ [m] : thickness of the cohesive layer
- r_c [-] : Reduction factor for resistance at the exit point = 0.3
- h [m] : Level of the outside water level in relation to NAP, with a probability of occurrence equal to the norm

Appendix C: Groundwater Flow Results

Values of measured clay layer permeability, calculated as described above, are given in Table 12. It is immediately clear that some significant increases of permeability occurred as a result of driven pile construction. However, the apparent permeabilities calculated after pile construction cannot simply be assumed to apply at full scale as they depend on the ratio of pile diameter (or width for an H-section pile) to model diameter. In order to interpret the results at full scale it is better to think of either the increase of flow caused by the construction of a single pile, under a given hydraulic gradient across the clay layer, or the dimensions of an equivalent column of overlying soil adjacent to the pile in the clay. In each case it is assumed that individual piles do not interfere with one another, which is likely to be true if the spacing exceeds about three pile diameters (Hird, Emmett, & Davies, 2006).

Table 12: Summary of groundwater flow results (Source: (Hird, Emmett, & Davies, 2006))

Test no.	Pile type*	T/D or T/L**	Clay layer permeability (m/s)		Increase of full-scale flow under unit hydraulic gradient (litres/day)	Normalised area of equivalent column of overlying soil (A_s/A_p)
			Pre-pile	Post-pile (apparent)		
A4	C	2	1.55×10^{-9}	1.46×10^{-9}	Negligible	Negligible
A5	C	1	1.40×10^{-9}	3.32×10^{-6}	1451	0.118
A6	H	2	1.21×10^{-9}	2.08×10^{-6}	911	0.044
A7	H	4	2.09×10^{-9}	1.29×10^{-6}	563	0.030
A8	C	4	1.81×10^{-9}	1.70×10^{-9}	Negligible	Negligible
A9	H	8	1.65×10^{-9}	1.08×10^{-7}	47	0.002
A10	H	2	2.08×10^{-9}	5.83×10^{-7}	254	0.051
A11	C	1	2.05×10^{-9}	1.41×10^{-6}	618	0.157
A13	CFA	1	1.10×10^{-6}	Flow results unreliable – see text		
A14	CFA	2	1.48×10^{-9}	2.34×10^{-9}	Negligible	Negligible
A15	CFA	2	1.42×10^{-9}	1.68×10^{-9}	Negligible	Negligible
A16	H	2	1.49×10^{-9}	2.19×10^{-6}	957	0.051

*C = cylindrical, H = H-section, CFA = continuous flight auger

**T = clay layer thickness, D = pile diameter, L = pile side-length (for H-section)

Appendix D: Flow chart

The figure below shows the complete flow chart. At the end of the flow chart, reference is made to a number of figures with schematizations in which it is made clear how the influence can be taken into account in the calculations. These figures can be found in Appendix E: Schematizations for additional calculations.

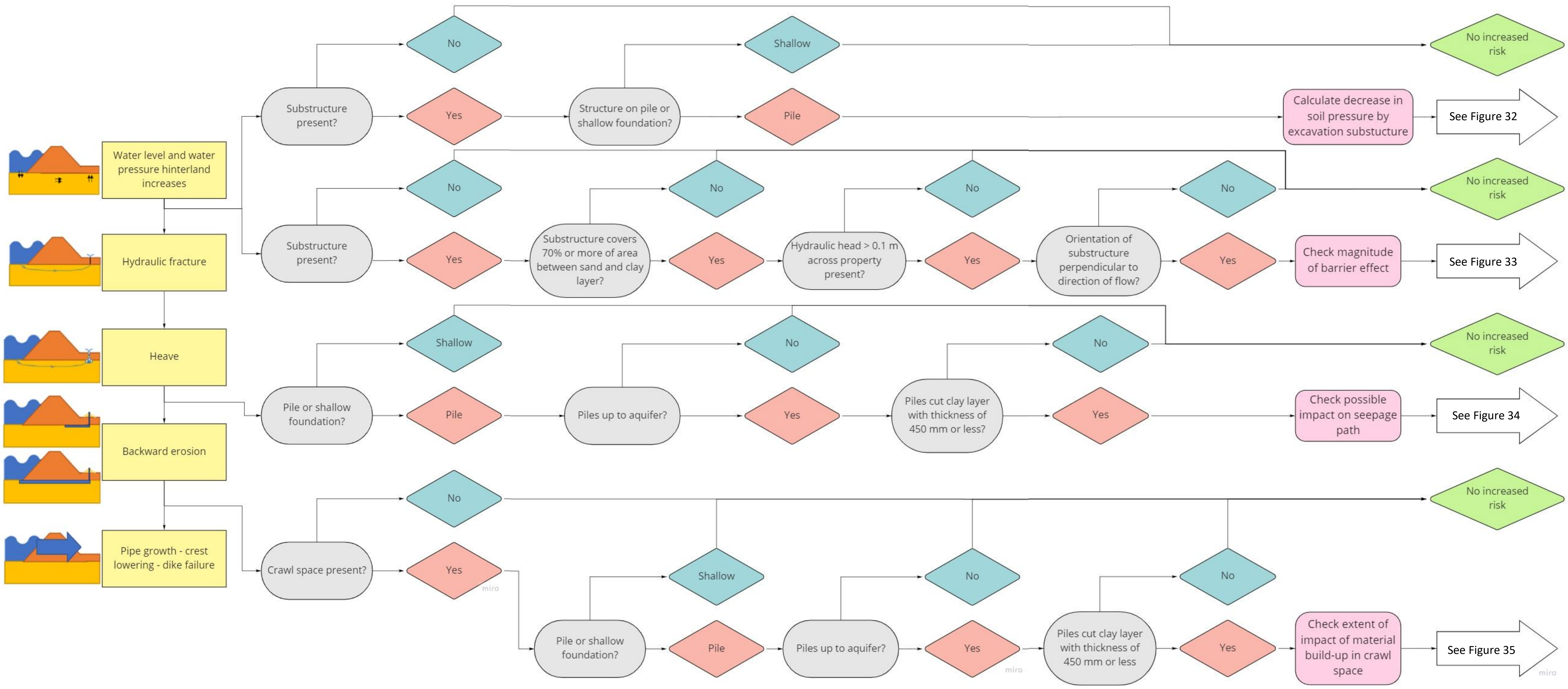


Figure 31: Flow chart

Appendix E: Schematizations for additional calculations

This appendix contains the schematizations that go with the recommendations for additional calculations in the flow chart.

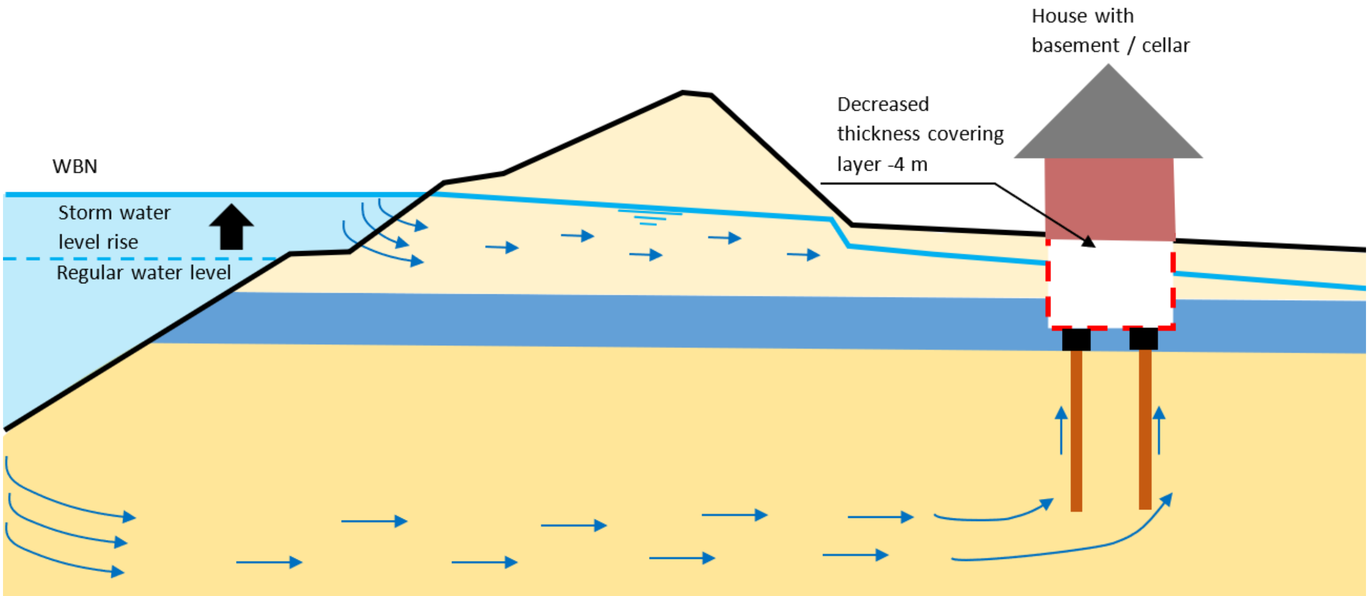


Figure 32: Schematization of influence caused by excavation substructure

In the case of substructure at a property, it is recommended that a decrease in the thickness of the overburden of 4 metres be included in the calculations. This is schematically shown in Figure 32.

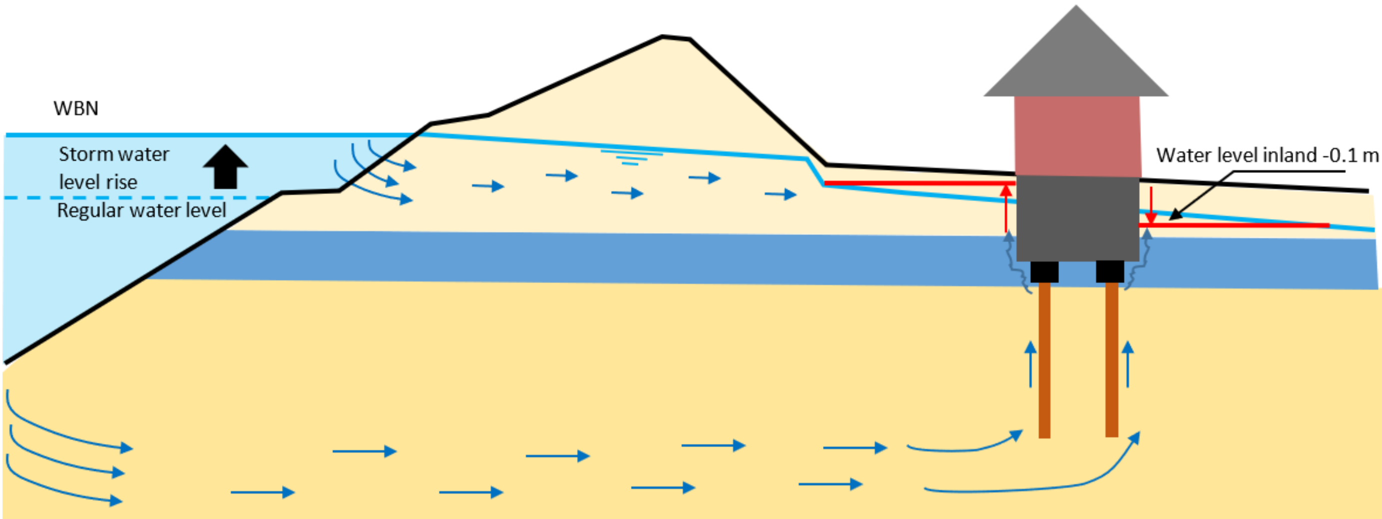


Figure 33: Schematization of influence caused by barrier effect

When the barrier effect is expected to occur at a structure, it is recommended to include a decrease in the water level inside the dike in the calculations. This is equivalent to an increase in hydraulic head. In Figure 33 this is shown schematically.

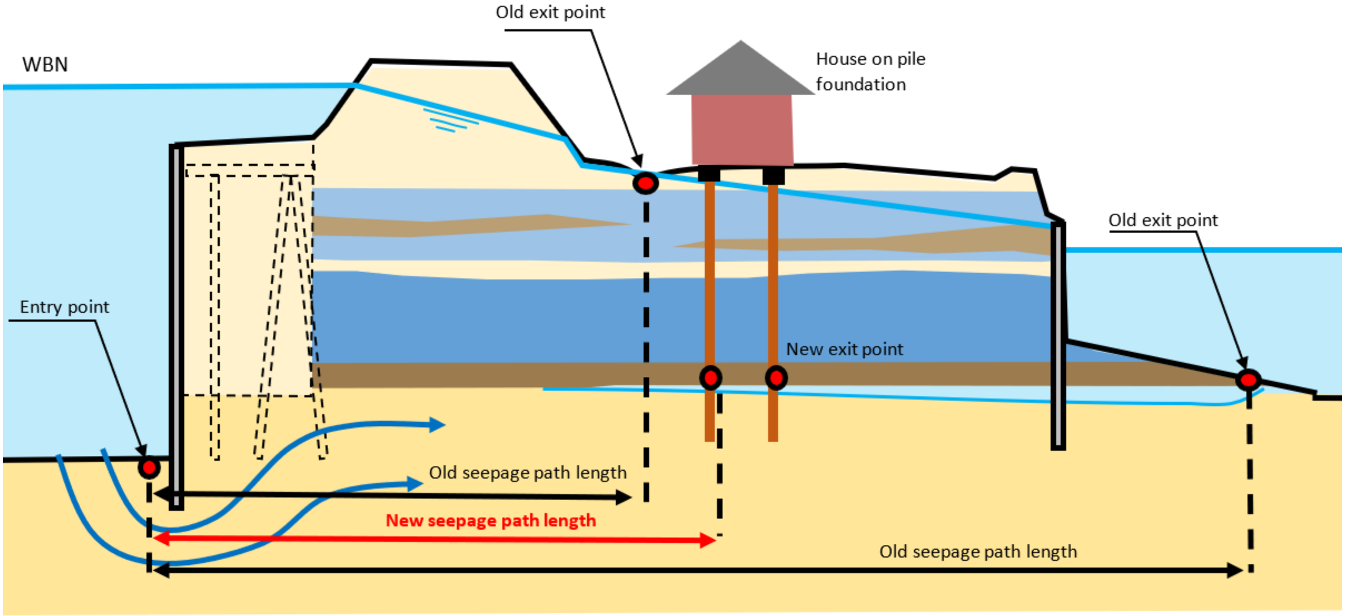


Figure 34: Schematization of influence caused by piles perforating clay layer

If the piles cause perforation of the clay layer, this will increase or decrease the seepage path length. This can be included in the calculations by moving the exit point to the location of the building. This is schematically shown in Figure 34.

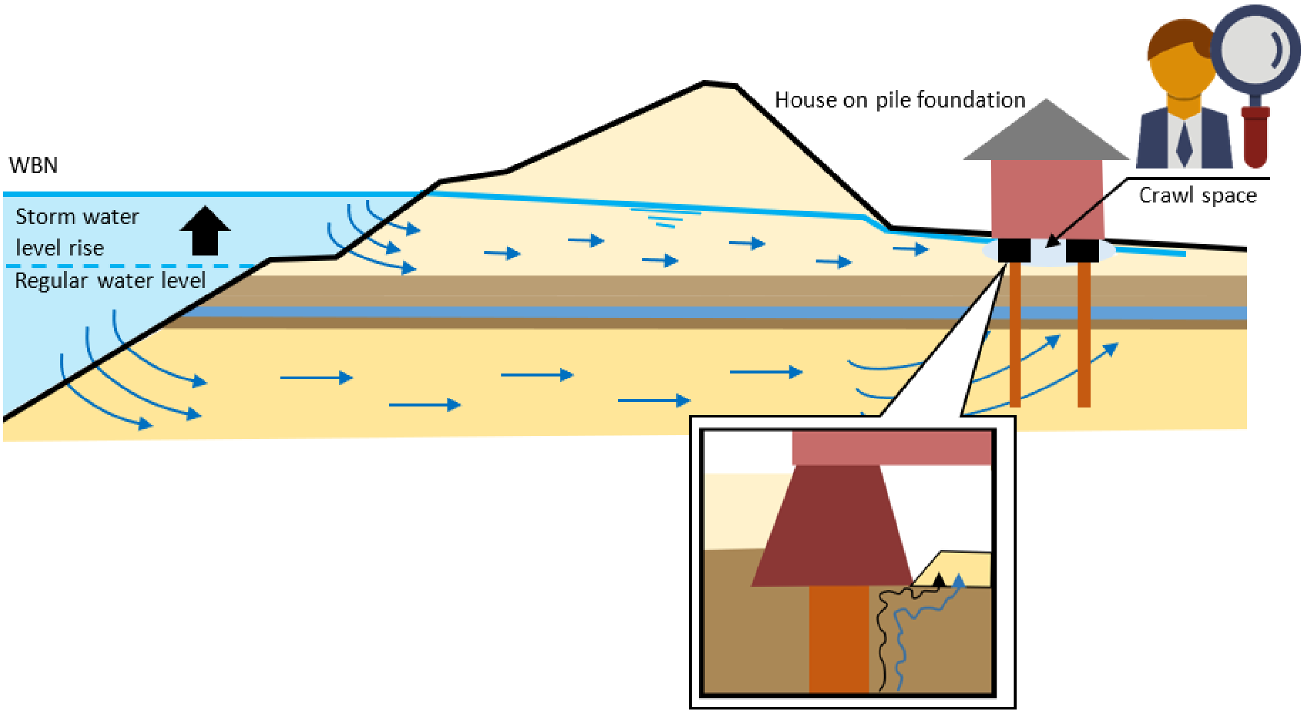


Figure 35: Schematization of influence caused by crawl space

When the flow chart recommends further consideration of the potential impact of a crawl space, it is recommended to determine how much sediment can be deposited in the crawl space. If this is a large amount, it could mean that a lot of material is washed away from under the dike, which could cause an increased risk.

Appendix F: Data Summary Possible Case Study Locations Nieuwe Diep

Legend

For the assessment of section 13-4, the HHNK used geotechnical length profiles along the flood defence. These profiles were drawn up along the entire flood defence on the outside, on the crest and at the rear of the dike. This geodata was studied to choose the case study location. In Figure 36 one can see the legend for the geotechnical profiles.

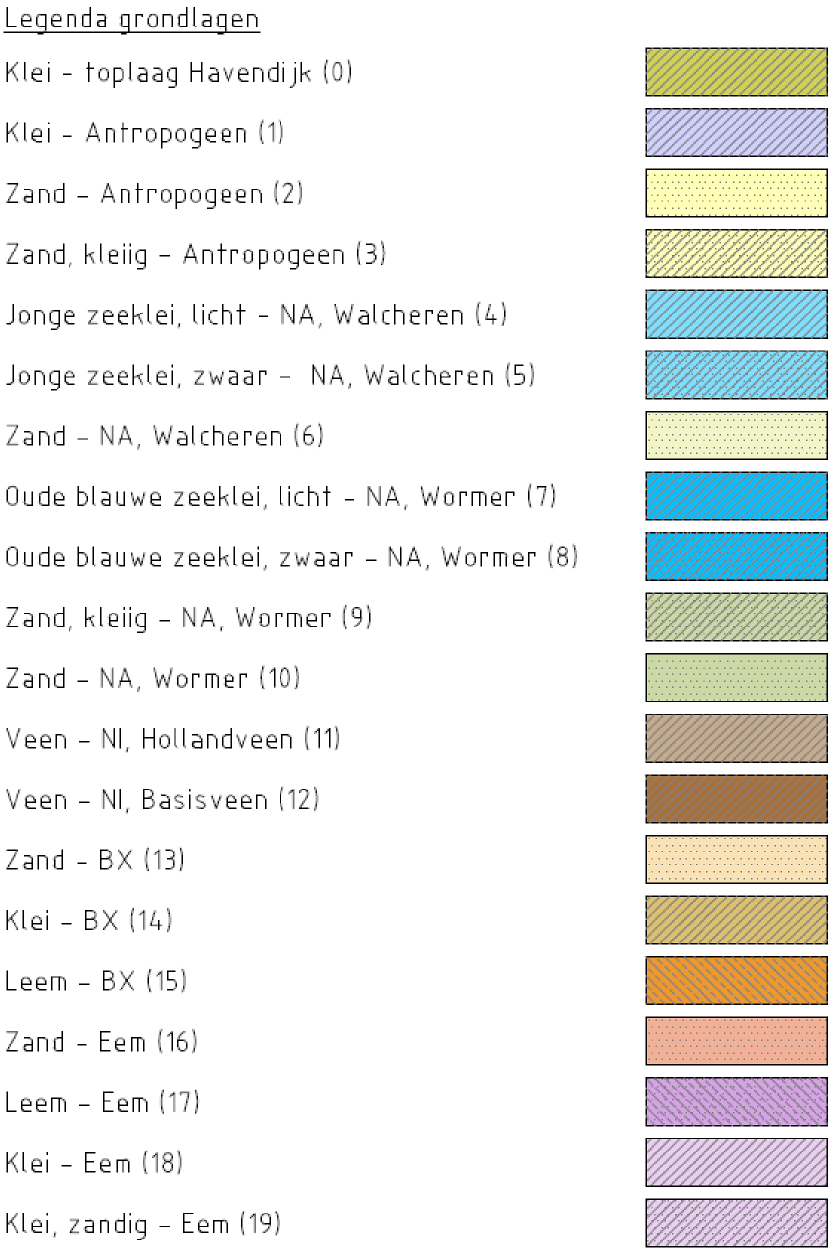


Figure 36: Legend subsoil layers (Source: (Hoogheemraadschap Hollands Noorderkwartier, 2020))

Nogal Wiedus & Storm aan Zee

At the Nogal Wiedus & Storm aan Zee beach pavilions, the geotechnical profile in Figure 38 first shows a thin clay layer and then a thick sand layer at the crest of the dike. The buildings are both on the crest of the dike and the cross-sections of the buildings show that they both have shallow foundations. No pile foundations were found in the building plans shown in Figure 39 and Figure 40 that were retrieved from the archives. This is likely because the dike consists of a solid sand package which is expected to have a good bearing capacity. Because the buildings have little influence deeper in the subsurface, they are unlikely to affect the piping failure mechanism. In Table 13 can be seen that due to the limited hydraulic head, no values for the probability of failure are calculated by excel for this location. This in combination with the knowledge of the foundation makes that this location is not considered interesting for the case study.

Table 13: Piping calculation results Nogal Wiedus & Storm aan Zee (Source: (Hoogheemraadschap Hollands Noorderkwartier, 2021))

Results 13-4_0113		**SF**	**β**	**P_f**	**γ_{eis}**
		[-]	[-]	[1/jaar]	
Hydraulic fracture	F_u	-1,73	#GETAL!	#GETAL!	1,45
Heave	F_h	-0,58	#GETAL!	#GETAL!	1,10
Backward erosion	F_p	-2,18	#GETAL!	#GETAL!	1,22
Piping overall	F_{piping}	#GETAL!	#GETAL!	#GETAL!	

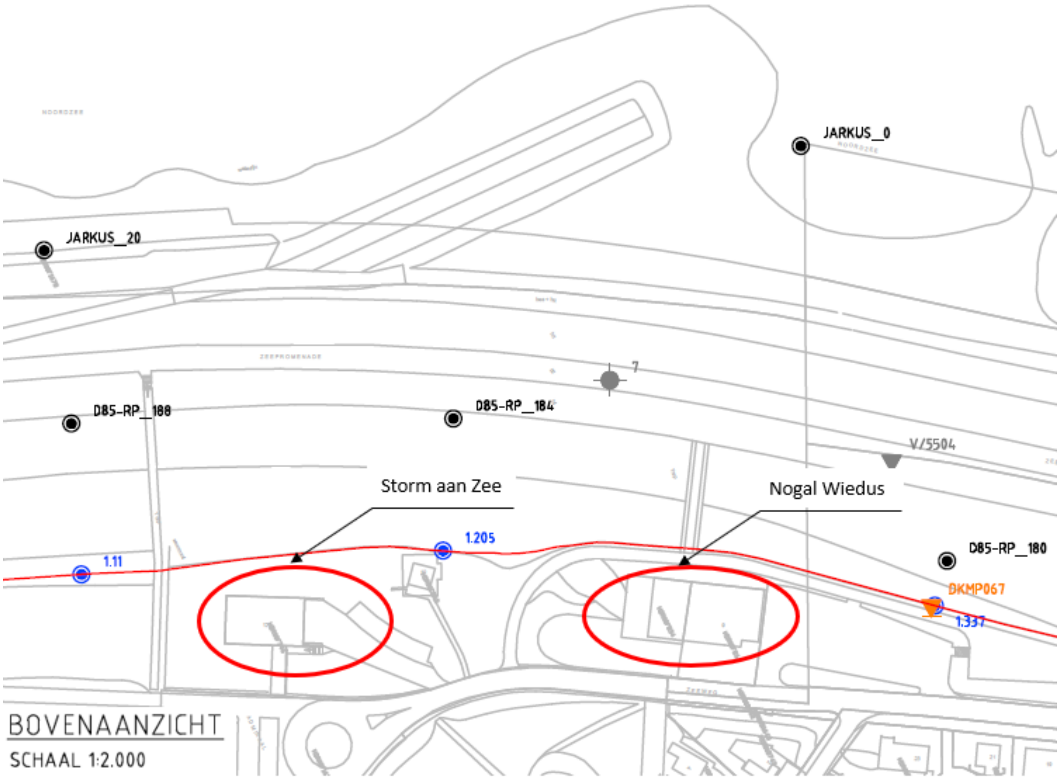


Figure 37: Locations of Storm aan Zee and Nogal Wiedus in relation to the dike (Source: (Hoogheemraadschap Hollands Noorderkwartier, 2020))

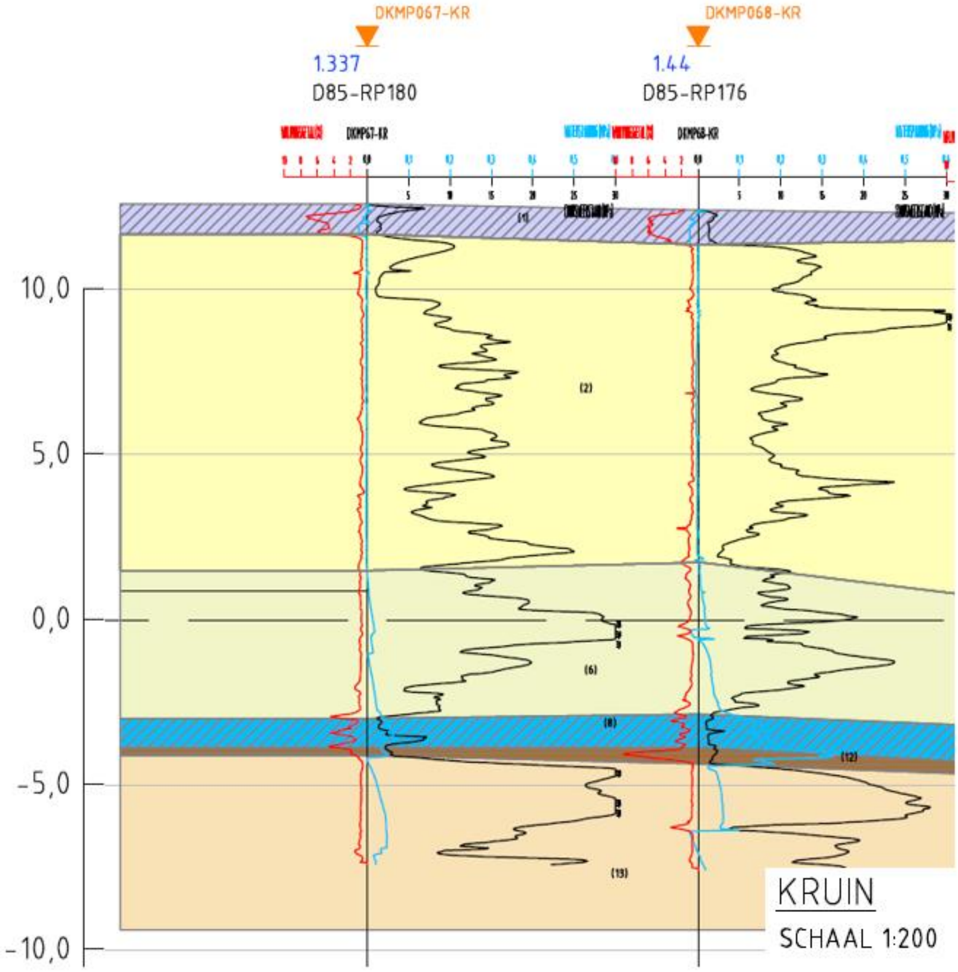


Figure 38: Geotechnical profile at Nogal Wiedus and Storm aan Zee; crest of dike (Source: (Hoogheemraadschap Hollands Noorderkwartier, 2020))

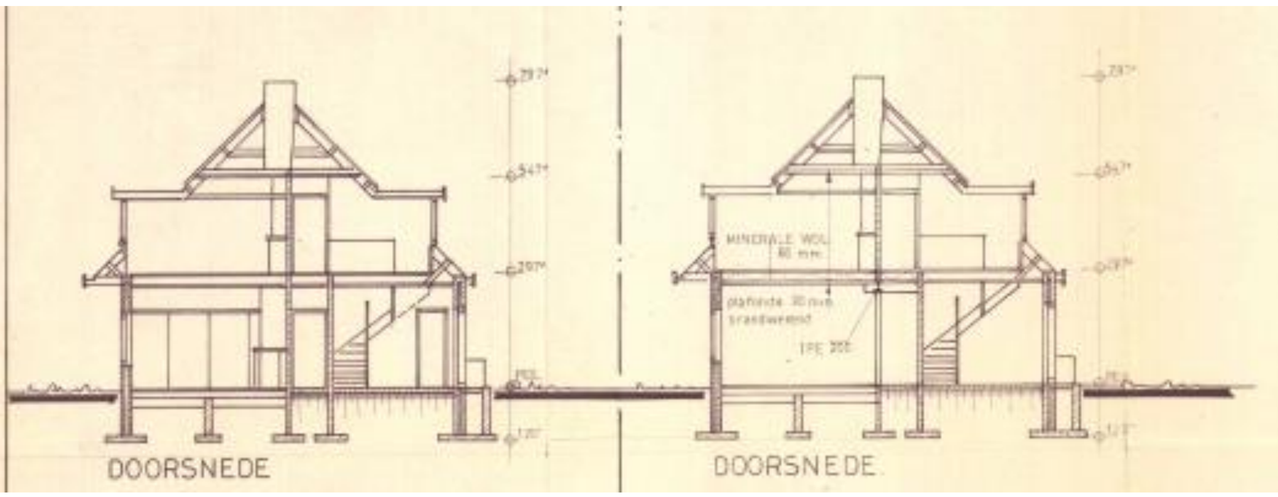


Figure 39: Cross section Nogal Wiedus (Source: (Architektenburo H. v.d. Laan , 1989))

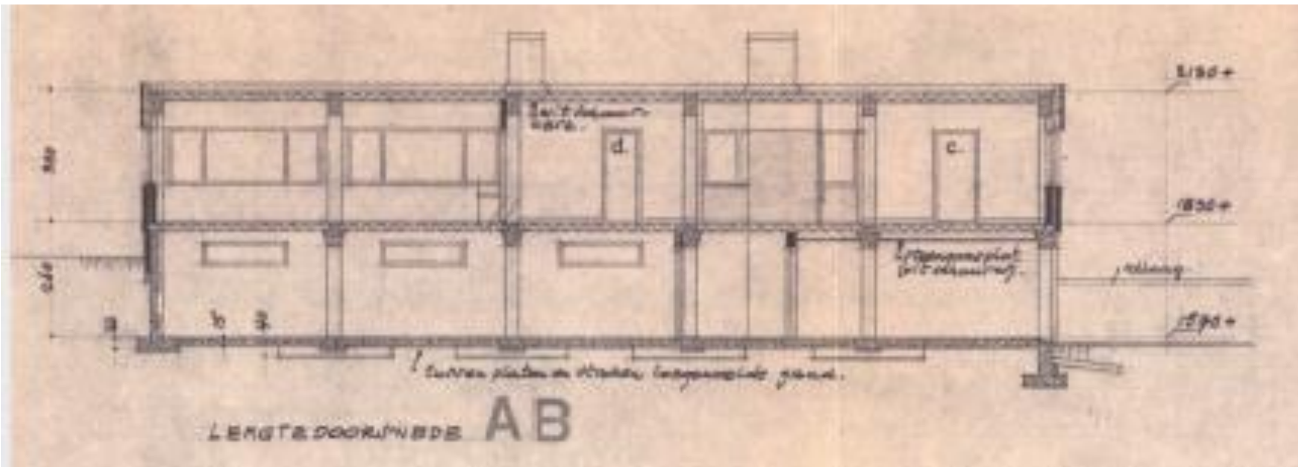


Figure 40: Cross section Storm aan Zee (Source: (H. Th. Oudejans Architect BNA, 1984))

Lighthouse Lange Jaap

No geotechnical profile is available at the Lange Jaap lighthouse. It is therefore assumed that the geotechnical profile at Lange Jaap looks similar to the one at the foot of the dike. In this profile in Figure 42 first a sand layer of several metres can be seen and then a thin clay layer separates this upper sand layer from the aquifer. No drawings of the lighthouse foundation were found in the archives, the only drawings that were found were similar to the one in Figure 43. The only thing that has been found is that the foundation consists out of 249 piles on which a concrete platform is poured, but it is not known how deep the piles go into the ground. Since the lighthouse is in a bad condition and there is a chance that it will fall over (NOS, 2021), it is interesting to know what this would mean for the risk of piping. If it falls over, the foundation may leave a large hole when it is ripped out of the ground, which could increase the risk of hydraulic fracture. However, this scenario contains many assumptions and uncertainties. The calculation results of the HHNK in Table 14 and Table 15 show that the calculated probability of piping is very small. This in combination with the unlikely scenario of the lighthouse falling over and leaving a hole, led to the decision that this was not the most interesting location for the case study.

Table 14: Piping calculation results scenario 1 Lighthouse Lange Jaap (Source: (Hoogheemraadschap Hollands Noorderkwartier, 2021))

Results 13-4_0141 SC1		**SF**	**β**	**P_f**	**γ_{eis}**
		[-]	[-]	[1/jaar]	
Hydraulic fracture	F_u	0,50	2,08	1,89E-02	1,45
Heave	F_h	0,24	1,21	1,13E-01	1,10
Backward erosion	F_p	2,50	6,33	1,25E-10	1,22
Piping overall	F_{piping}	1,44	6,33	1,25E-10	

Table 15: Piping calculation results scenario 2 Lighthouse Lange Jaap (Source: (Hoogheemraadschap Hollands Noorderkwartier, 2021))

Results 13-4_0141 SC2		**SF**	**β**	**P_f**	**γ_{eis}**
		[-]	[-]	[1/jaar]	
Hydraulic fracture	F_u	1,04	3,68	1,17E-04	1,45
Heave	F_h	0,38	2,19	1,43E-02	1,10
Backward erosion	F_p	2,88	6,71	9,87E-12	1,22
Piping overall	F_{piping}	1,53	6,71	9,87E-12	

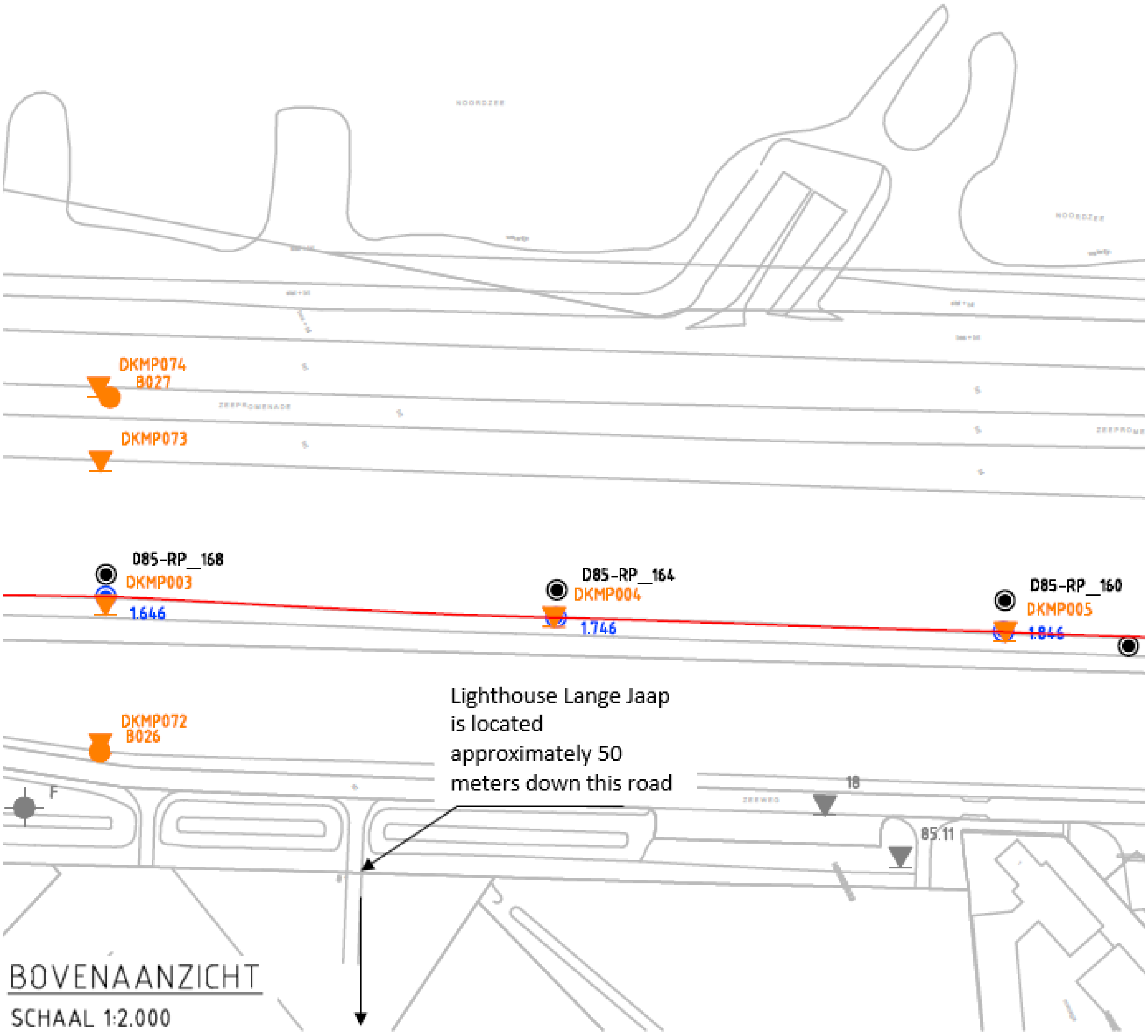


Figure 41: Location of Lighthouse Lange Jaap in relation to the dike (Source: (Hoogheemraadschap Hollands Noorderkwartier, 2020))

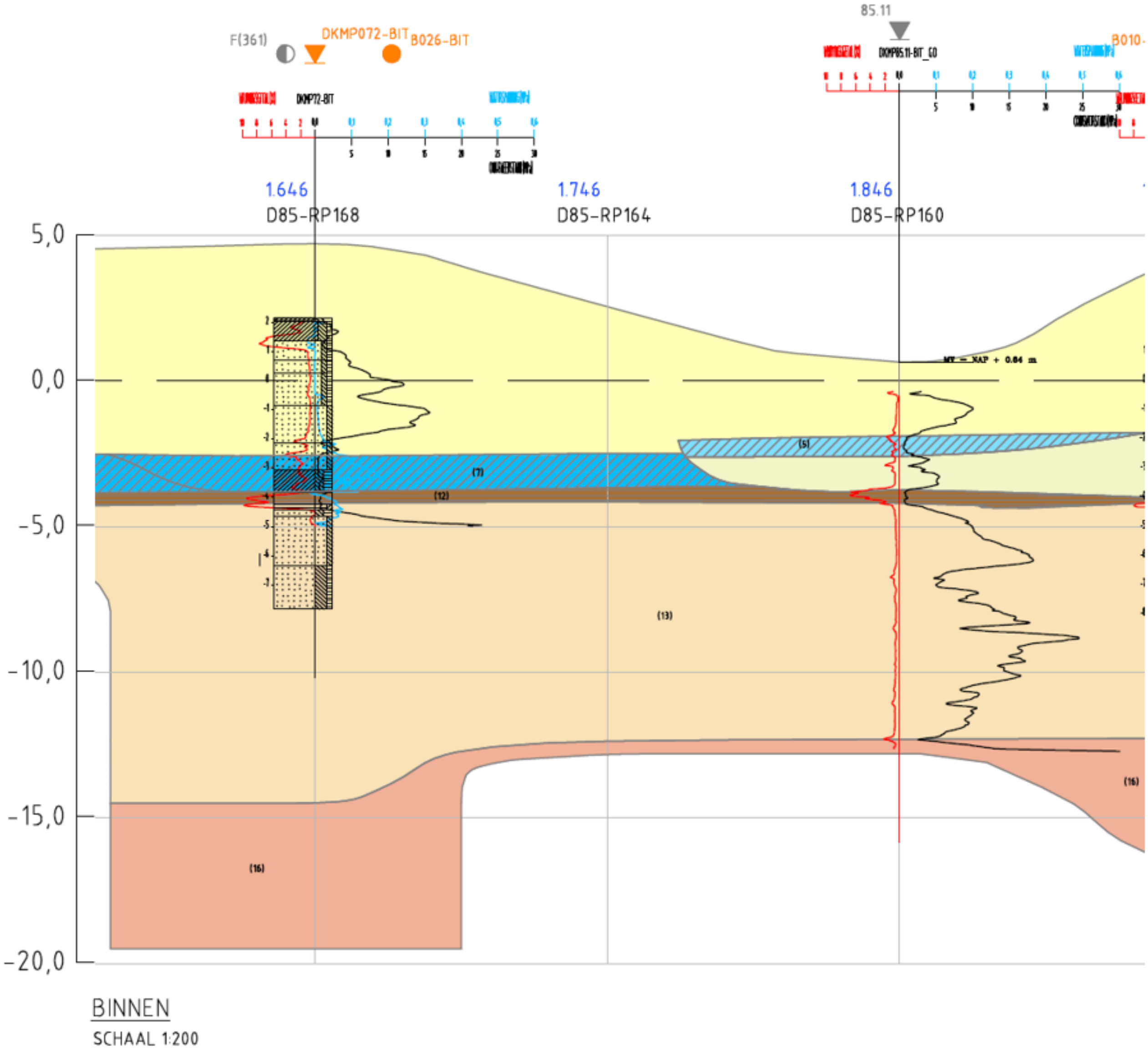


Figure 42: Geotechnical profile at Lighthouse Lange Jaap; inner side of dike (Source: (Hoogheemraadschap Hollands Noorderkwartier, 2020))

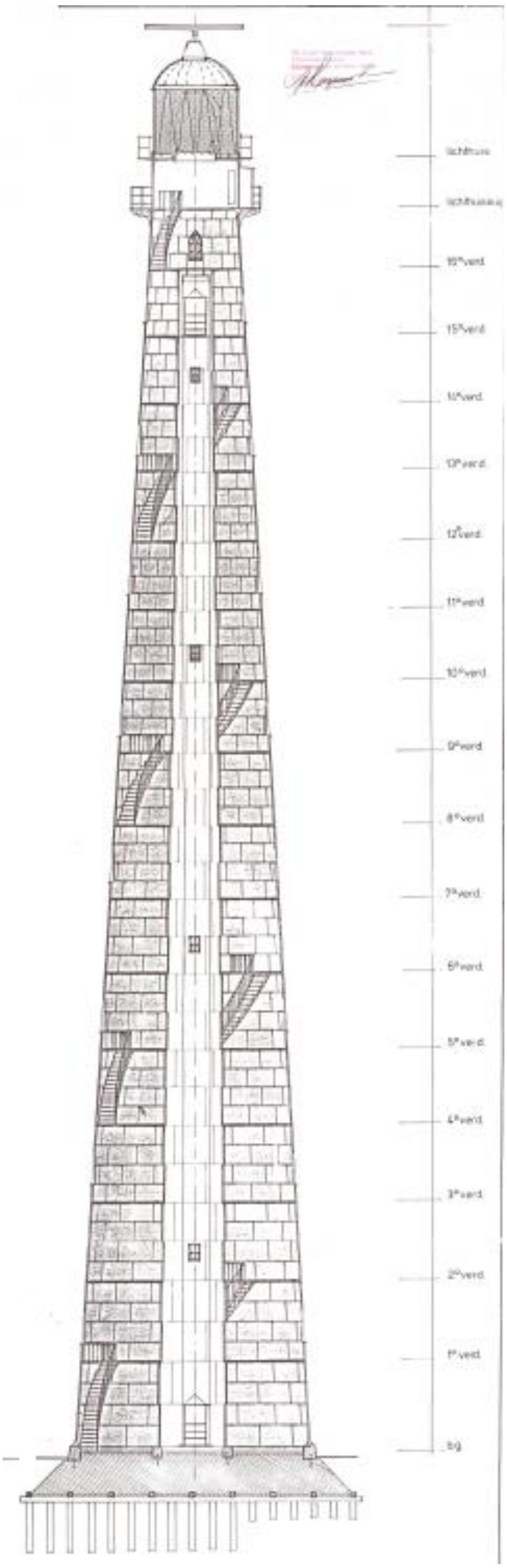


Figure 43: Drawing of lighthouse Lange Jaap (Source: (Gopzeman, 1991))

Hotel Landsend

There is no geotechnical profile available at Hotel Landsend. The property is located on a part that is considered to be the foreland of the dike. This foreland is only assessed in the context of the embankment tracks of the dike. The geotechnical profile in Figure 45 is the geotechnical profile at the crest of the dike just before it curves in front of Hotel Landsend. The location of the hotel in relation to the flood defence is shown in Figure 44. In the geotechnical profile can be seen that there is first a thick sand package and then alternating clay and peat layers separating the aquifer from the upper sand package. The foundations of the hotel consist of pile foundations with piles of 16.5 metres (Figure 47) , which therefore go deep enough to reach the aquifer. However, since the hotel is located so far on the foreshore from the dike, it is not considered interesting for the case study.

Table 16: Piping calculation results Hotel Landsend (Source: (Hoogheemraadschap Hollands Noorderkwartier, 2021))

Results 13-4_0347		**SF**	**β**	**P_f**	**γ_{eis}**
		[-]	[-]	[1/jaar]	
Hydraulic fracture	F_u	1,14	3,87	5,38E-05	1,45
Heave	F_h	0,49	2,71	3,41E-03	1,10
Backward erosion	F_p	1,24	4,42	4,93E-06	1,22
Piping overall	F_{piping}	1,01	4,42	4,93E-06	

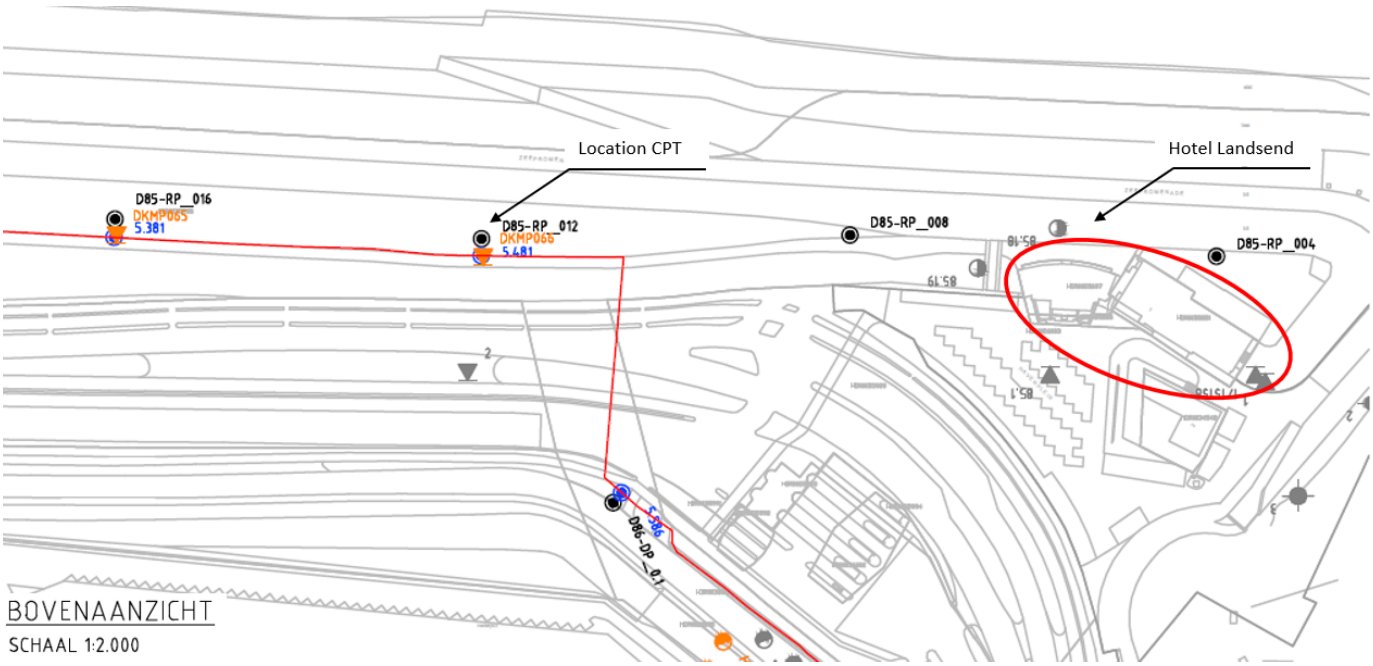


Figure 44: Location cone penetration test dike crest in relation to Hotel Landsend (Source: (Hoogheemraadschap Hollands Noorderkwartier, 2020))

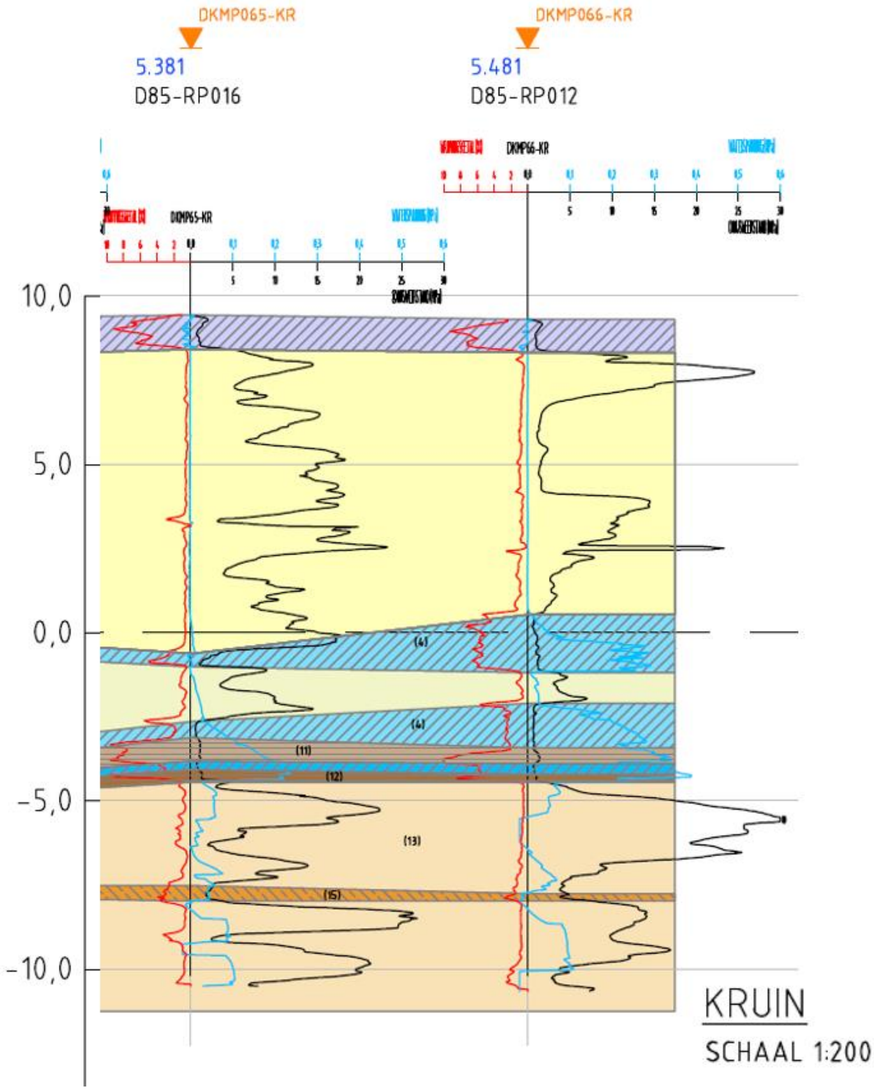


Figure 45: Geotechnical profile Hotel Landsend; crest of dike (Source: (Hoogheemraadschap Hollands Noorderkwartier, 2020))

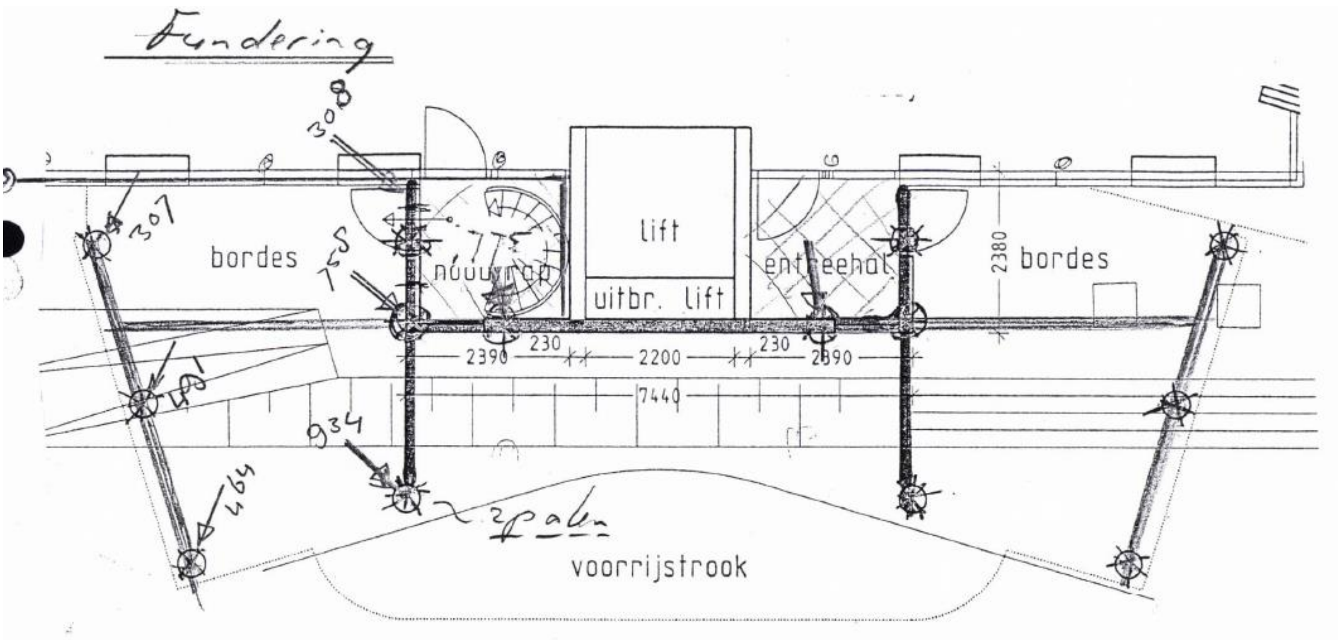


Figure 46: Foundation plan Hotel Landsend (Source: (Wijcon BV))

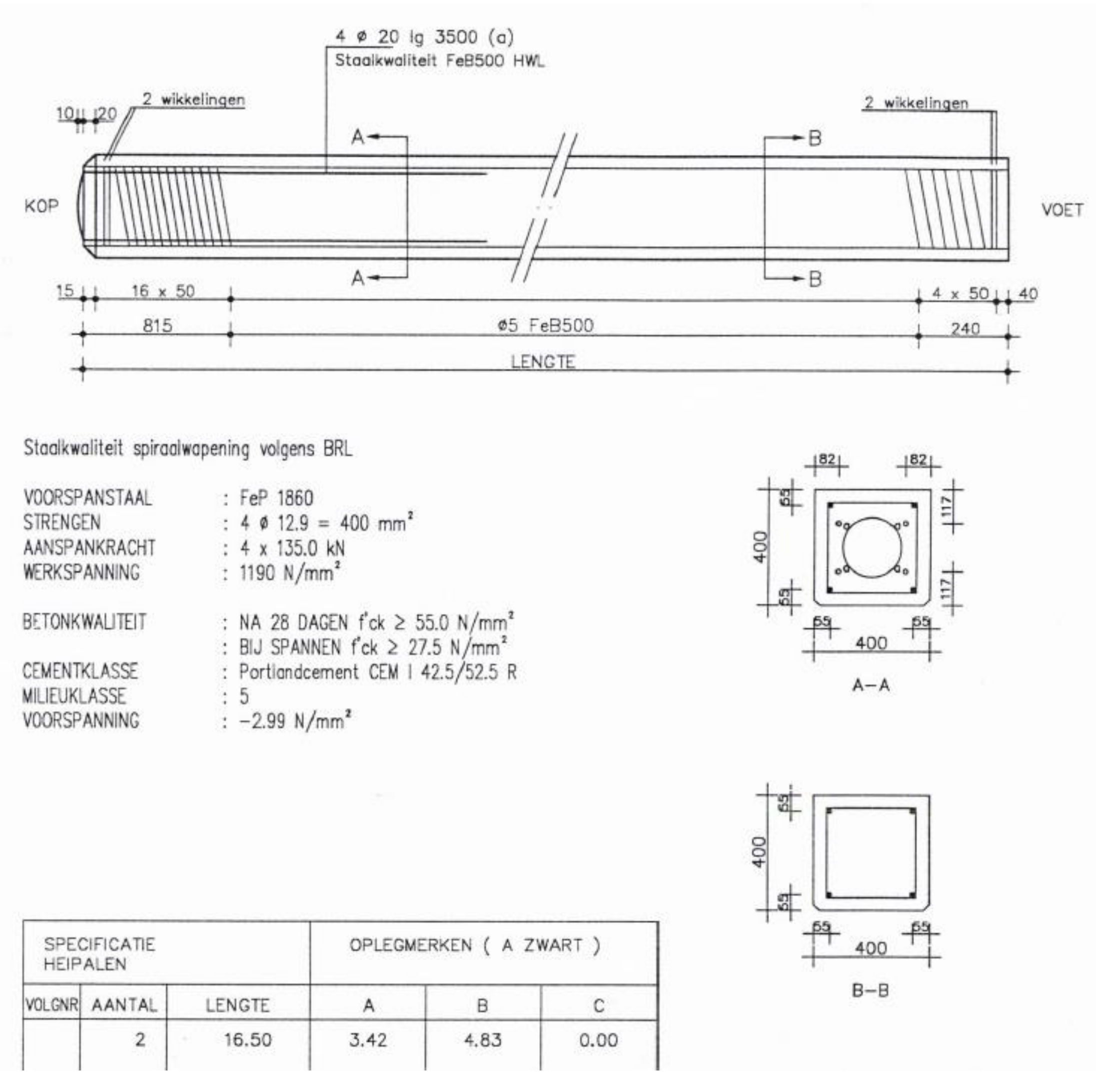


Figure 47: Pile characteristics Hotel Landsend (Source: (Haitsma, 2003))

Paleiskade

In Figure 48 it can be seen that Paleiskade 100 is located outside the dike. This property was chosen because no foundation data could be obtained for a property on the inside of the dike. Since this data was available for Paleiskade 100, it was assumed that the foundations of the buildings behind the dike are similar to those of Paleiskade 100. The geotechnical profile shows many alternating layers. First a relatively thin sand layer, followed by a thin clay layer which is interrupted in some parts by sand and peat. Near the Paleiskade 100 building (on the right in Figure 49), there is even a section where there are no clay layers between the various sand layers. Based on the geotechnical profile in Figure 49 and the depth of the foundations of 12 metres below sea level (Figure 50), it can be assumed that the foundation of the buildings behind the dike reach the aquifer and may therefore have an impact on the piping failure mechanism.

Table 17: Piping calculation results Paleiskade (Source: (Hoogheemraadschap Hollands Noorderkwartier, 2021))

Results 13-4_0374		**SF**	**β**	**P_f**	**γ_{eis}**
		[-]	[-]	[1/jaar]	
Hydraulic fracture	F_u	1,12	3,84	6,24E-05	1,45
Heave	F_h	0,39	2,24	1,26E-02	1,10
Backward erosion	F_p	1,59	5,10	1,66E-07	1,22
Piping overall	F_{piping}	1,16	5,10	1,66E-07	

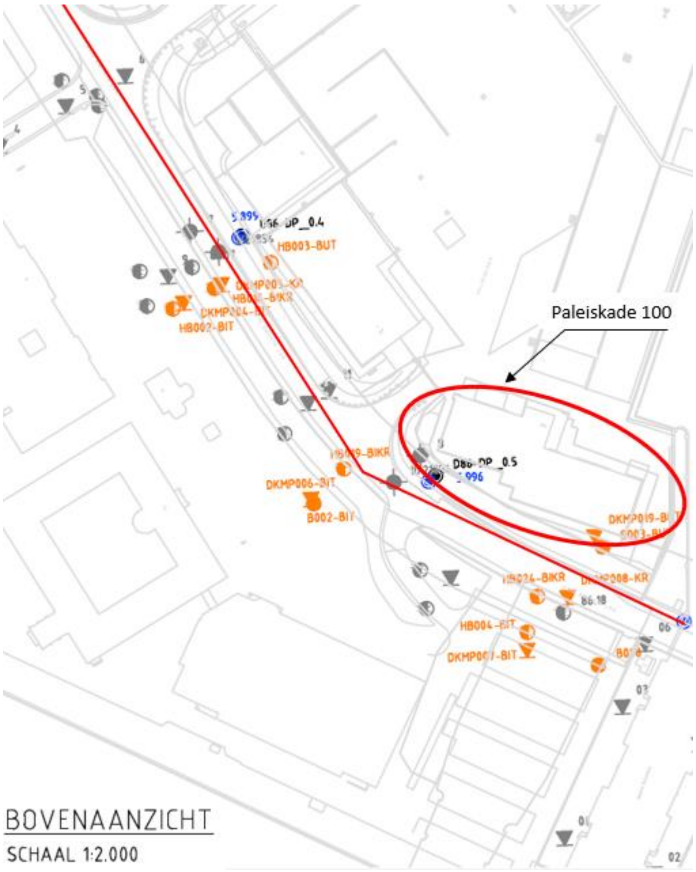


Figure 48: Location Paleiskade 100 in relation to dike (Source: (Hoogheemraadschap Hollands Noorderkwartier, 2020))

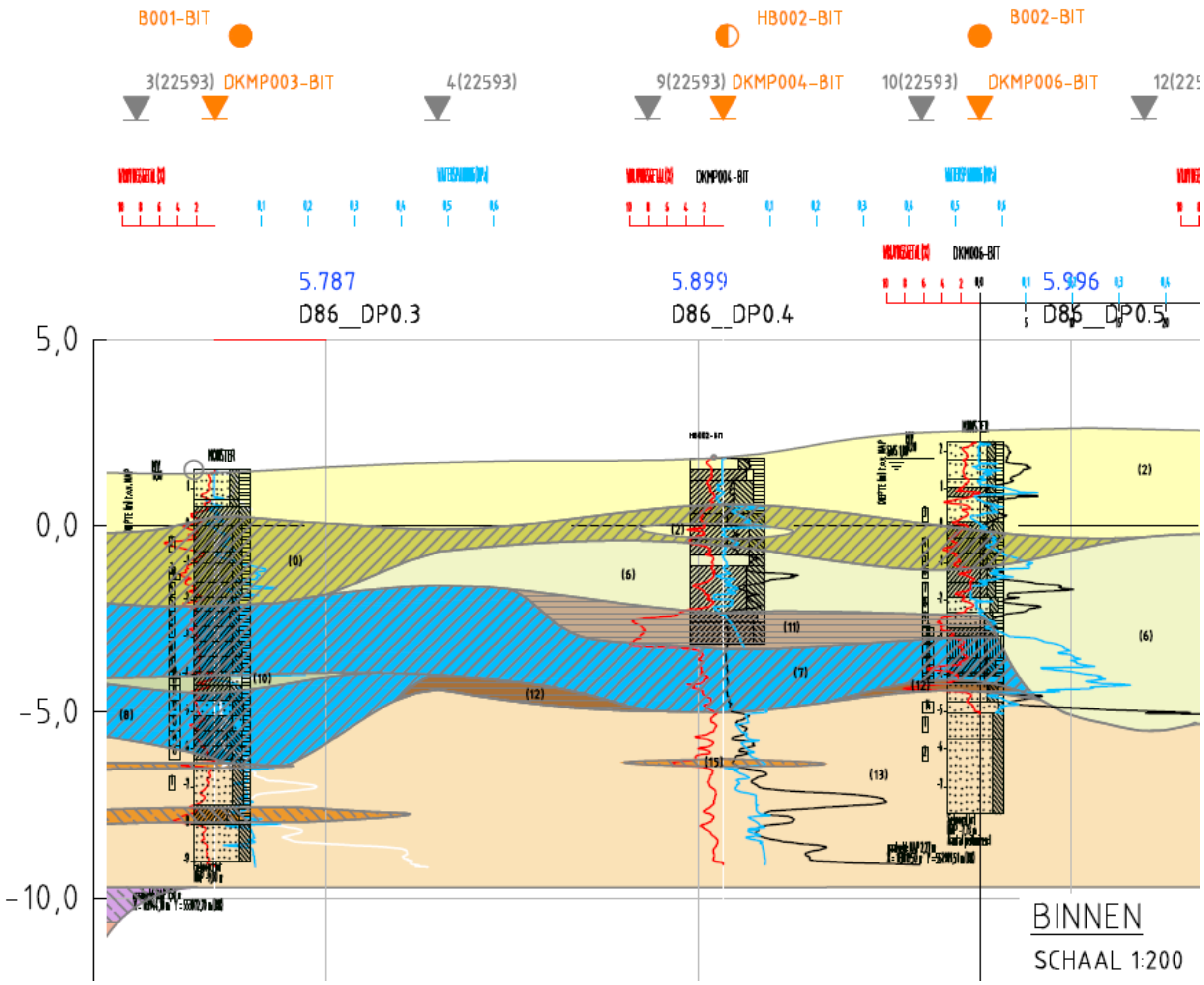


Figure 49: Geotechnical profile Paleiskade; inner side of dike (Source: (Hoogheemraadschap Hollands Noorderkwartier, 2020))

In onderstaande tabel is voor de betreffende sonderingen, naast de hoogte van het maaiveld, het vereiste paalpuntniveau aangegeven.

Sondering nr.	Maaiveld-hoogte m + NAP	Paalpuntniveau m - NAP
DKM 2	3,44	12,0
DKM 5	3,31	11,5
D6	3,30	11,5

Bij berekening van het draagvermogen is, naast het optreden van positieve wrijving, tevens met het optreden van negatieve kleef door het opgespoten zandpakket gerekend.

De palen dienen door het in de ondergrond aanwezige dijklichaam te worden geheid. Plaatselijk bestaat de dijkbekleding uit basaltblokken, die verwijderd dienen te worden. Hiertoe zal een ontgraving van 2,0 a 3,0 m moeten worden uitgevoerd, waarna de blokken verwijderd worden. Na het heien van palen dient de ruimte tussen de paal en de basalt blokken te worden aangevuld met b.v. beton. Het verdient aanbeveling deze aanvulling te dilateren van de paal b.v. door aan brengen van polystyreen naast de paal.

Figure 50: Part geotechnical soil investigation report (Source: (Struyk, 1988))

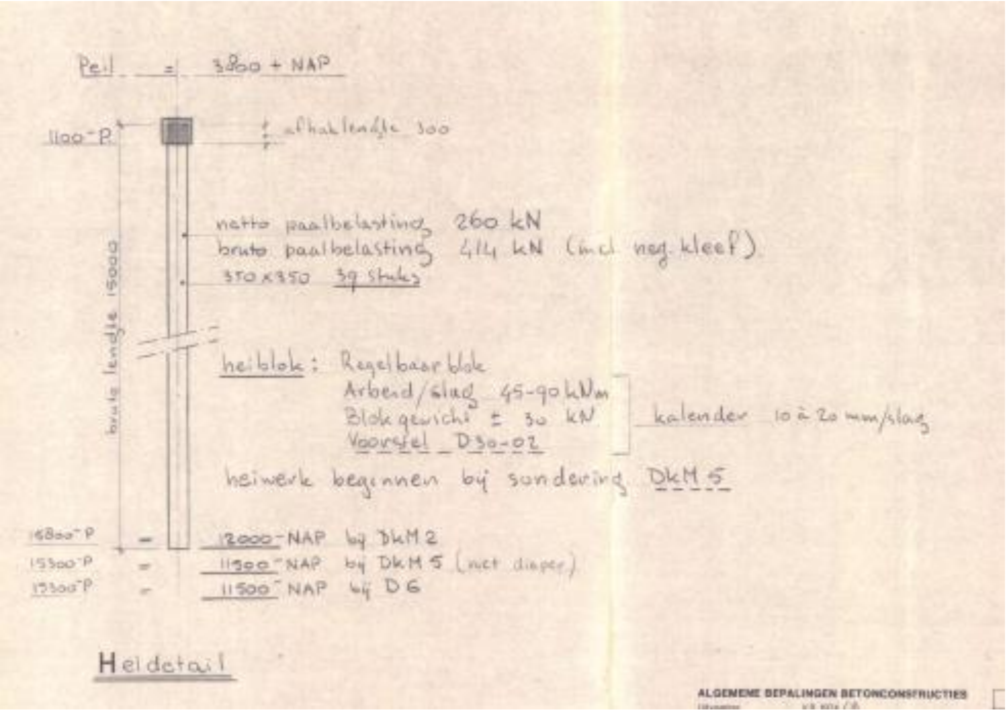


Figure 51: Piles details (Source: (Groen, 1988))

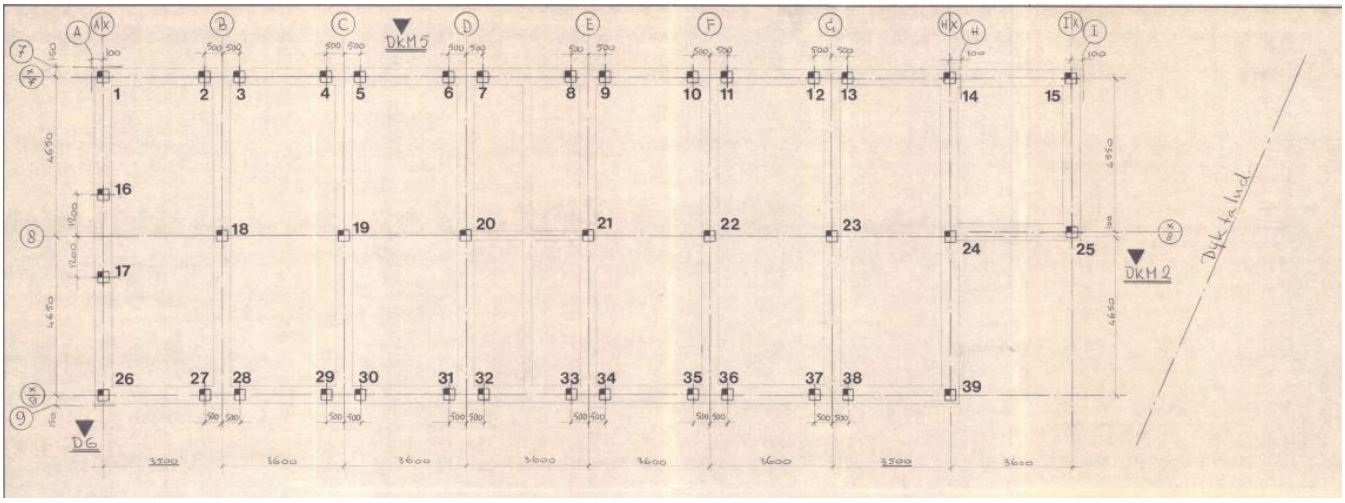


Figure 52: Pile plan with on the right the dike embankment (Source: (Groen, 1988))

Zwarte Pad

At Zwarte Pad, the building is on the inside of the dike. The geotechnical profile behind the dike in Figure 54 first shows a sand pack of varying thickness. Then thin layers of clay, sand and peat alternate. The aquifer is located at -5.0 NAP and a little deeper there are some thin layers of loam in between this package. The exact depth of the foundation was not found in the archives. However, the CPT of Figure 55 shows that the resistance increases at 5 metres below sea level. It is assumed that the piles, like surrounding buildings, are up to approximately 12 metres below NAP. This means that the piles here also reach into the aquifer and may therefore affect piping. The probability of piping calculated by the HHNK is also relatively high at this location (Table 18), making it extra interesting for the scenario calculations of the case study.

Table 18: Piping calculation results Zwarte Pad (Source: (Hoogheemraadschap Hollands Noorderkwartier, 2021))

Results 13-4_0424		**SF**	**β**	**P_f**	**γ_{eis}**
		[-]	[-]	[1/jaar]	
Hydraulic fracture	F_u	0,02	-4,51	1,00E+00	1,45
Heave	F_h	0,01	-4,75	1,00E+00	1,10
Backward erosion	F_p	1,23	4,41	5,26E-06	1,22
Piping overall	F_{piping}	1,00	4,41	5,26E-06	

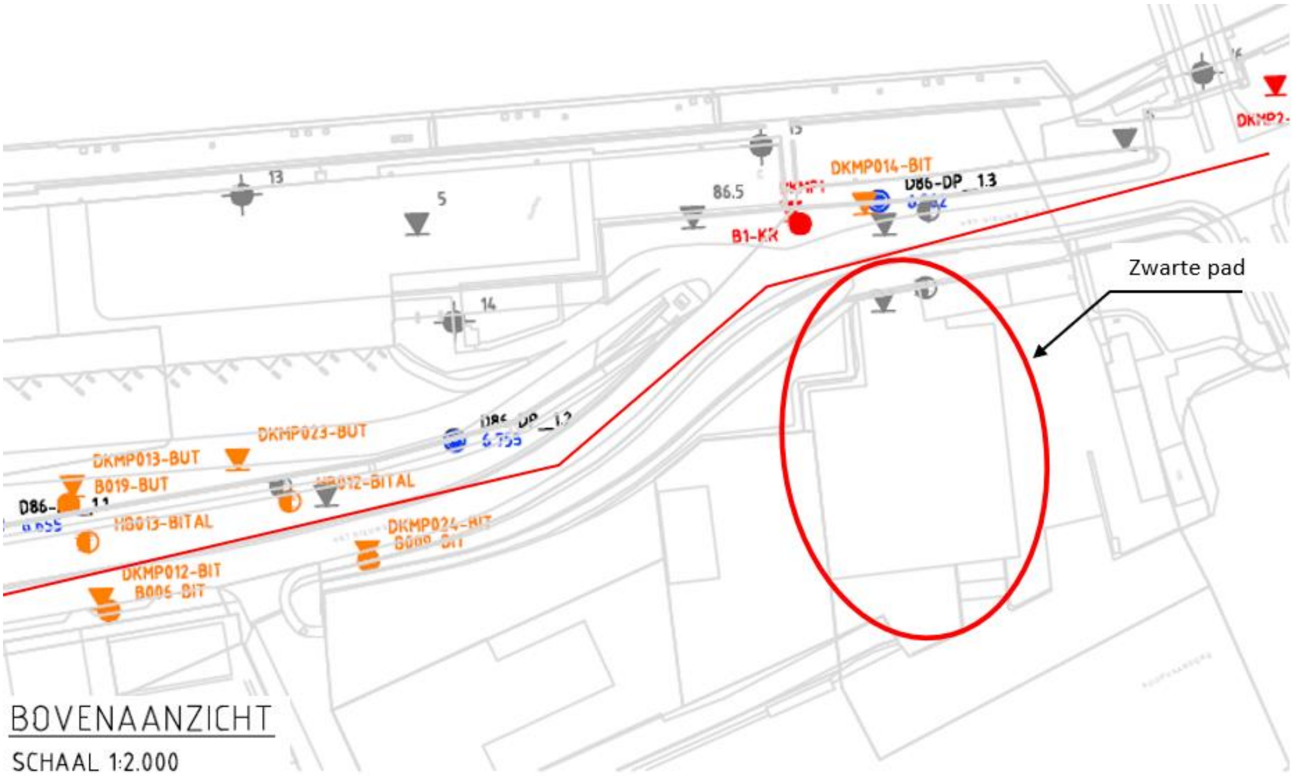


Figure 53: Location Zwarte Pad in relation to dike (Source: (Hoogheemraadschap Hollands Noorderkwartier, 2020))

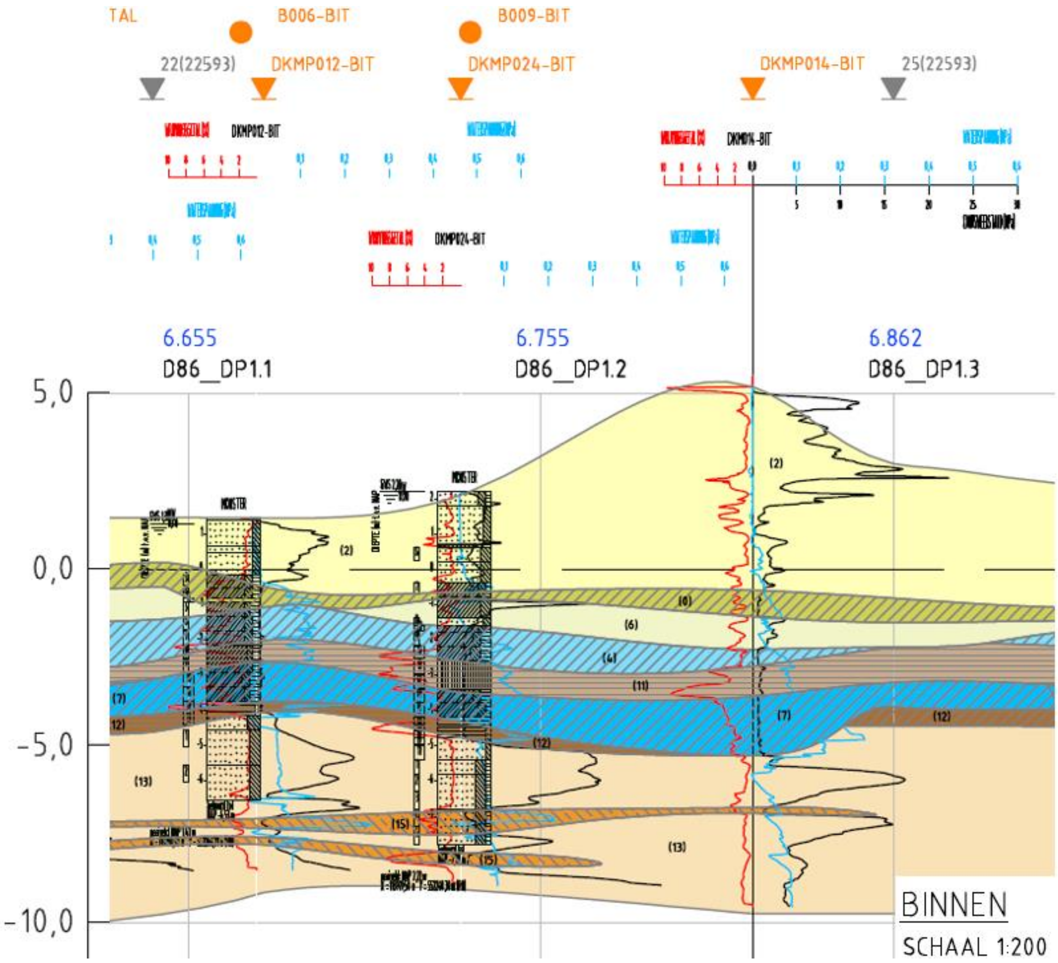


Figure 54: Geotechnical profile Zwarte Pad; inner side of dike (Source: (Hoogheemraadschap Hollands Noorderkwartier, 2020))

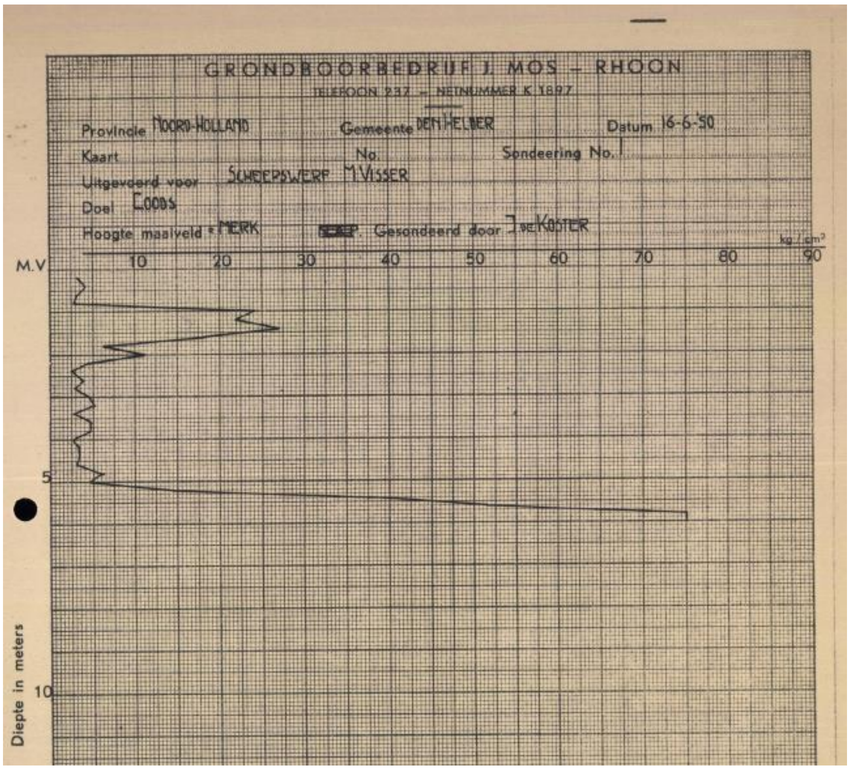


Figure 55: Cone penetration test results at Ankerpark (Source: (Grondboorbedrijf Mos-Rhoon, 1950))

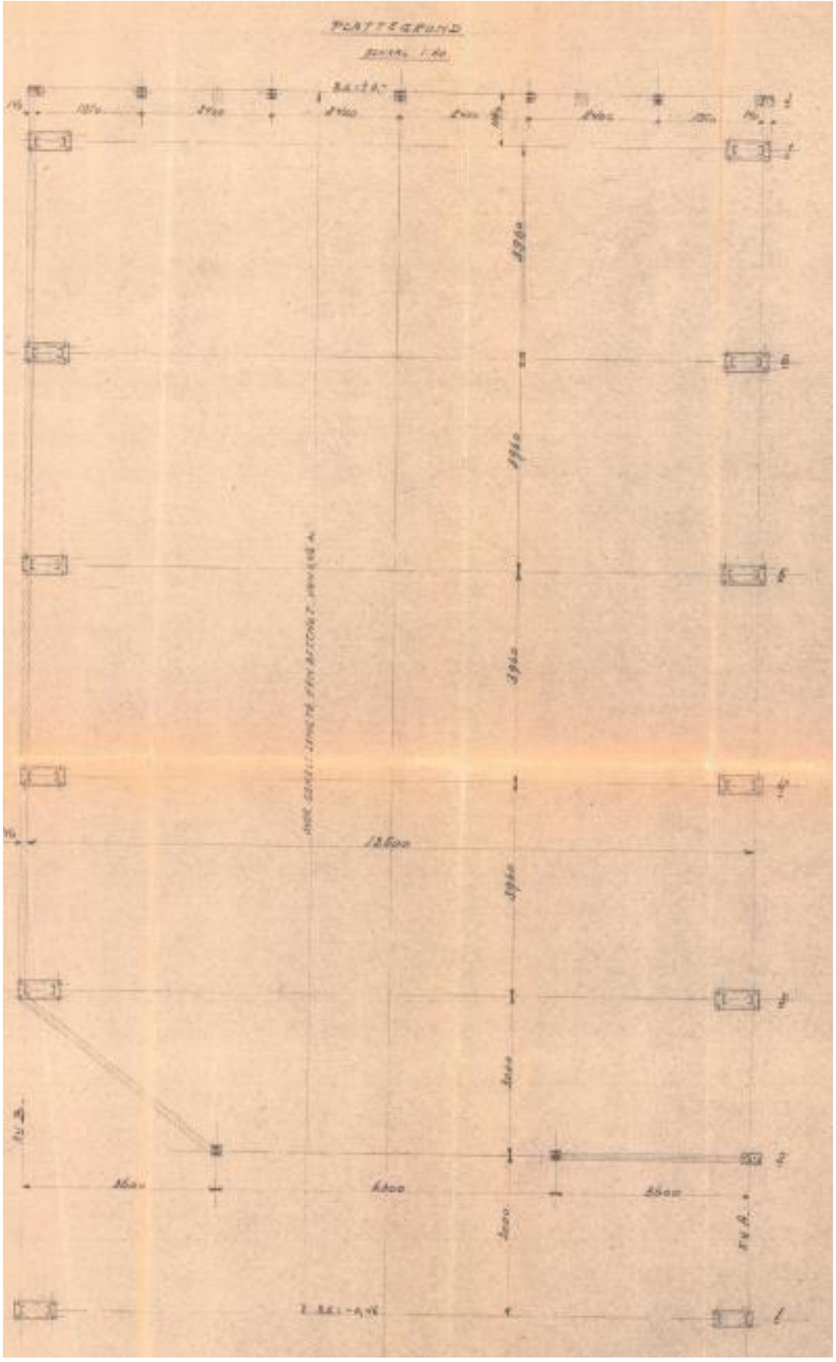


Figure 56: Foundation pile plan Ankerpark 2 (Source: (Baard, 1950))

Nieuwe Diep

The shed at Nieuwe Diep 34 is located just behind the dike on a narrow strip between the Nieuwe Diep and the Koopvaarders Binnenhaven. The geotechnical profile in Figure 58 first shows a relatively thick sand layer with thin clay layers in some places. A clay layer several metres thick then separates the top sand layer from the aquifer. In between, there are also thin layers of peat and loam in some places. No further information about the foundation of this building was found in the construction drawings from the archive. However, for some other buildings along the Nieuwe Diep it was found that the piles go to approximately -12 NAP. It is therefore also assumed that the foundation has an influence on piping in this case. In addition, the piping calculations of the HHNK in Table 19 show a high probability of piping. This location is therefore considered to be interesting for the case study.

Table 19: Piping calculation results Nieuwe Diep (Source: (Hoogheemraadschap Hollands Noorderkwartier, 2021))

Results 13-4_0439		**SF**	**β**	**P_f**	**γ_{eis}**
		[-]	[-]	[1/jaar]	
Hydraulic fracture	F_u	0,52	2,15	1,57E-02	1,45
Heave	F_h	0,27	1,44	7,56E-02	1,10
Backward erosion	F_p	1,01	3,87	5,41E-05	1,22
Piping overall	F_{piping}	0,88	3,87	5,41E-05	

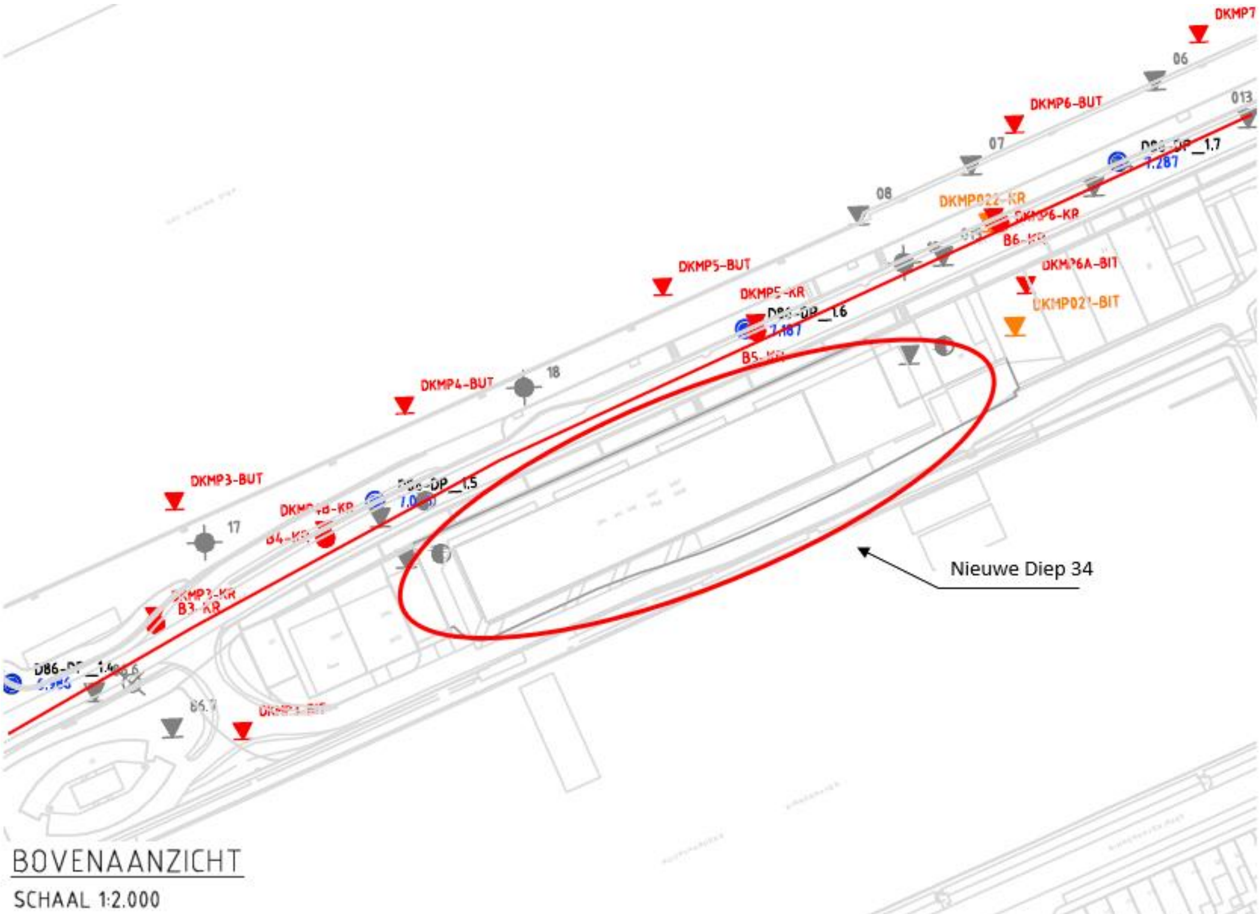


Figure 57: Location of Nieuwe Diep 34 in relation to the dike (Source: (Hoogheemraadschap Hollands Noorderkwartier, 2020))

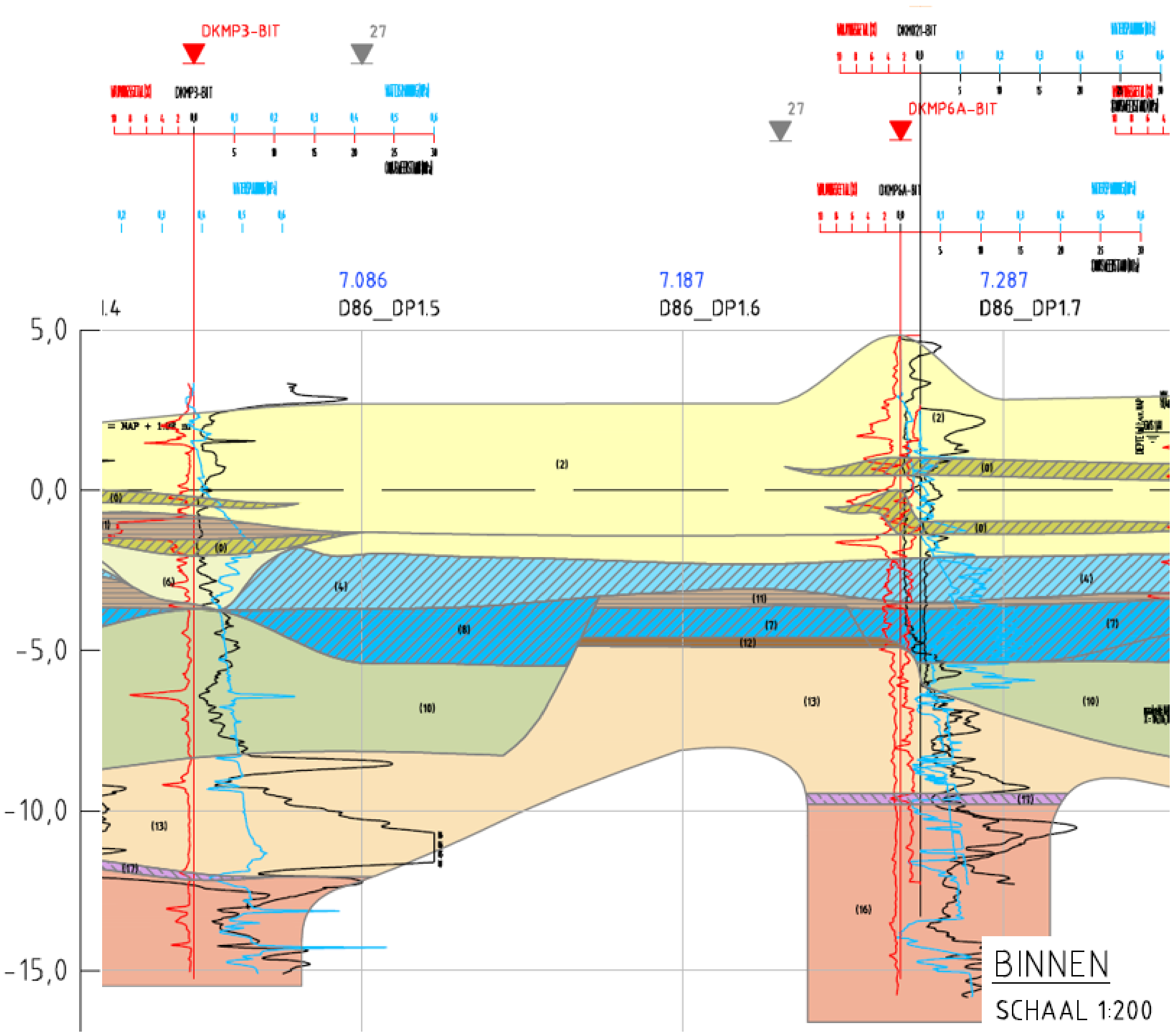


Figure 58: Geotechnical profile Nieuwe Diep; inner side of dike (Source: (Hoogheemraadschap Hollands Noorderkwartier, 2020))

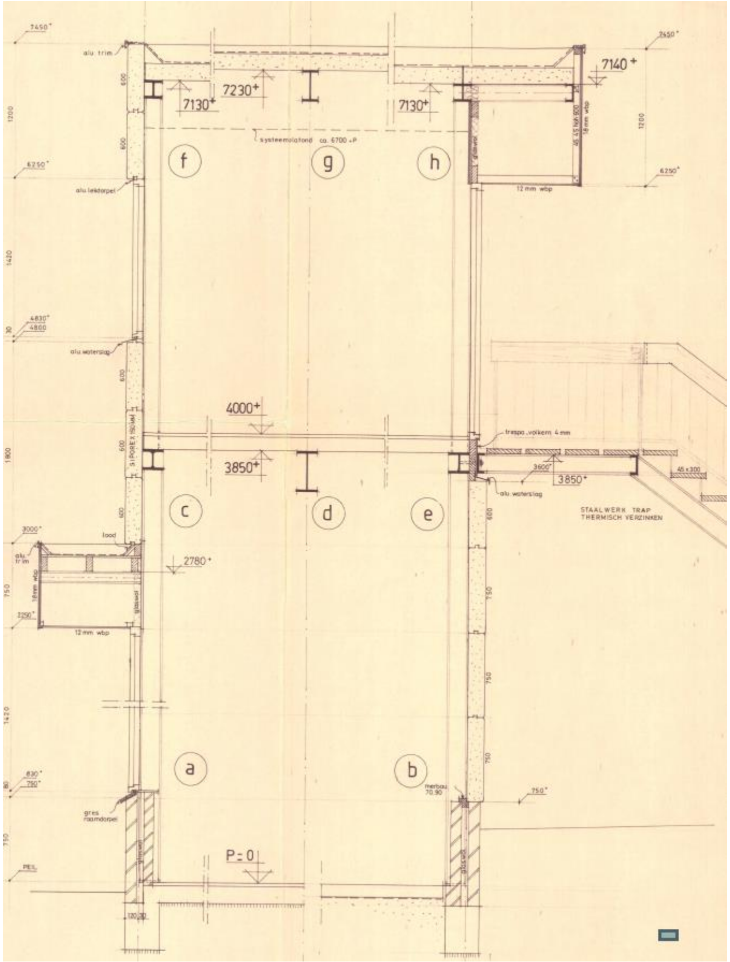


Figure 59: Cross-section Nieuwe Diep 34 (Source: (Architektenburo Fred Groen , 1985))

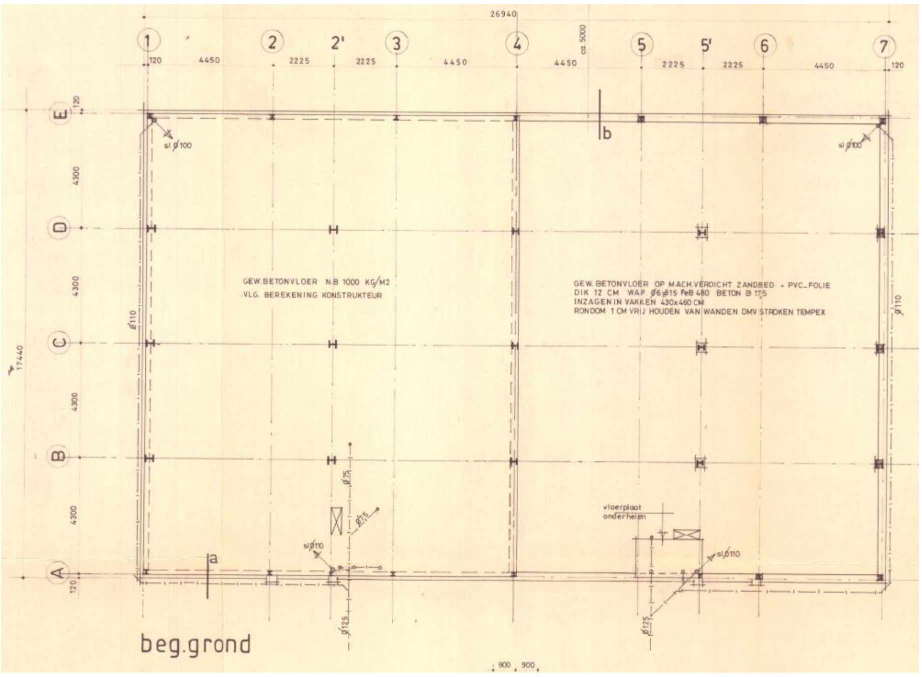


Figure 60: Ground level Nieuwe Diep 34 (Source: (Architektenburo Fred Groen , 1985))

Nieuwe Werk

The building Nieuwe Werk 85 is located outside the dike on the quay. The geotechnical profile in Figure 62 outside the dike shows that a thick sand package is separated from the aquifer by a thick clay layer with some peat mixed in. Based on the foundation information from the archives which can be seen in Figure 64 and Figure 65, it has been determined that the foundation is also located here at approximately -12 NAP. Because no clear foundation data were available for the premises just behind the dike, the focus for the soil investigation was shifted to this premises, although the calculations were based on a premises just behind the dike. It is assumed that it has a similar foundation as the building at Nieuwe Werk 85. The geotechnical profile in Figure 63 shows that just behind the dike there are thick clay and sand layers on the surface. A thin clay layer separates this package from the aquifer sand layer. Since the piles are driven into the ground to a depth of -12 NAP into the aquifer it is assumed that they influence the piping failure mechanism. It is therefore considered to be an interesting location for the case study.

Table 20: Piping calculation results Nieuwe Werk (Source: (Hoogheemraadschap Hollands Noorderkwartier, 2021))

Results 13-4_0484		**SF**	**β**	**P_f**	**γ_{eis}**
		[-]	[-]	[1/jaar]	
Hydraulic fracture	F_u	1,89	4,97	3,31E-07	1,45
Heave	F_h	0,79	3,72	1,01E-04	1,10
Backward erosion	F_p	1,68	5,25	7,48E-08	1,22
Piping overall	F_{piping}	1,19	5,25	7,48E-08	

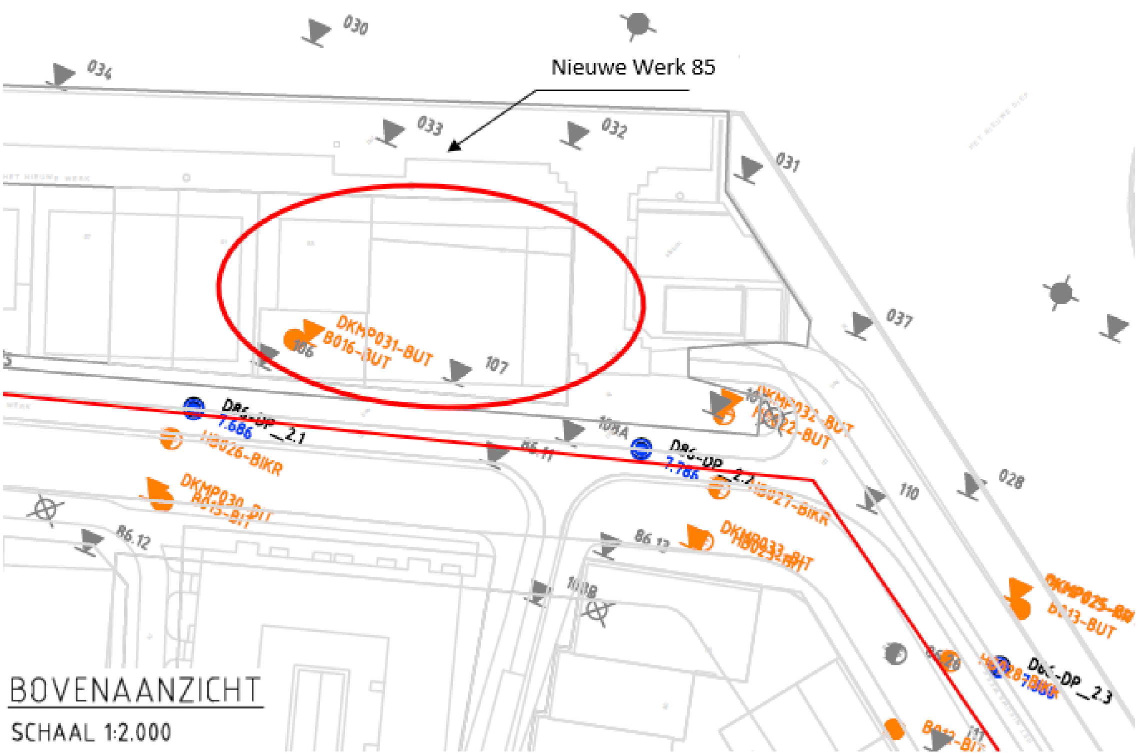


Figure 61: Location of Nieuwe Werk 85 in relation to the dike (Source: (Hoogheemraadschap Hollands Noorderkwartier, 2020))

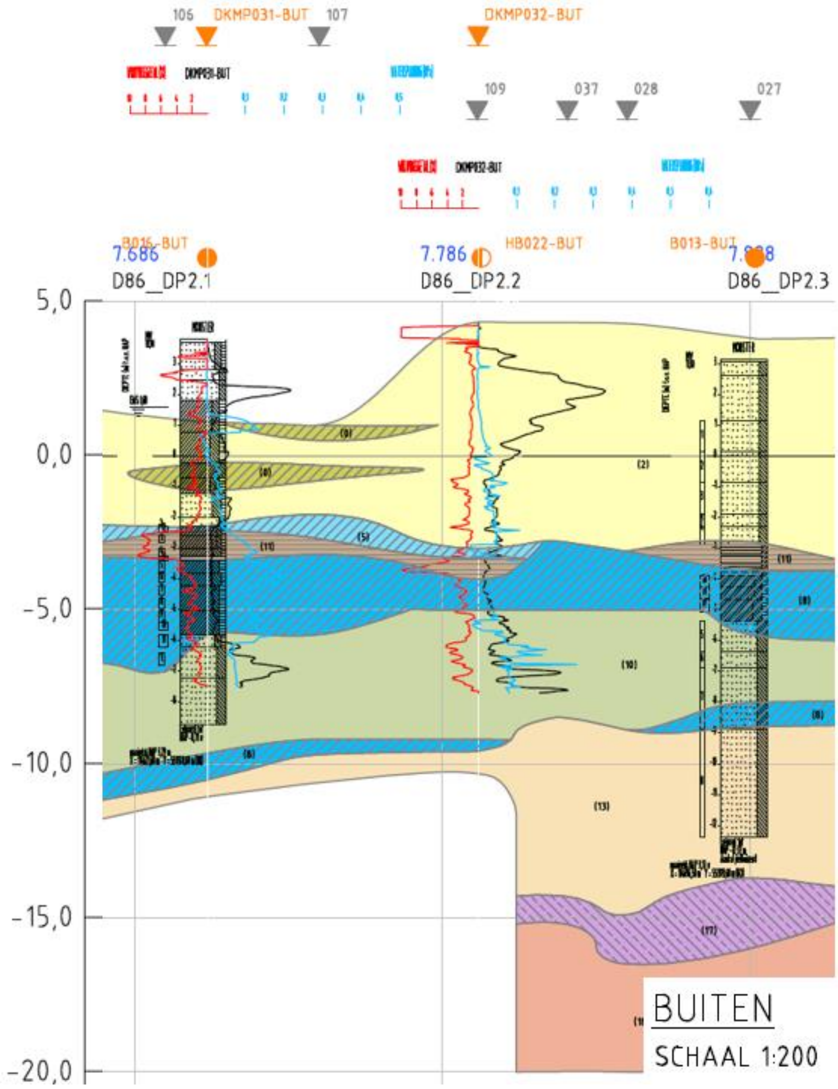


Figure 62: Geotechnical profile Nieuwe Werk; outer side of dike (Source: (Hoogheemraadschap Hollands Noorderkwartier, 2020))

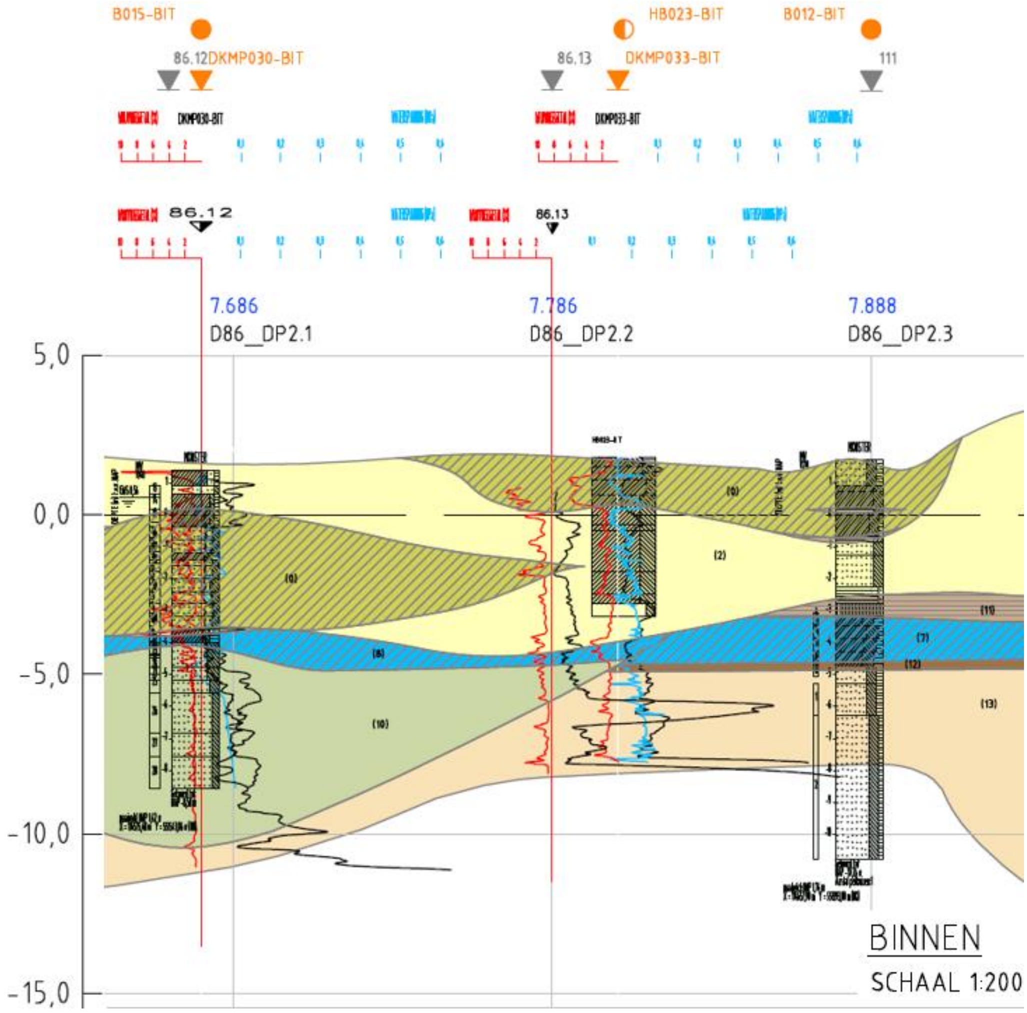


Figure 63: Geotechnical profile Nieuwe Werk; inner side of dike (Source: (Hoogheemraadschap Hollands Noorderkwartier, 2020))

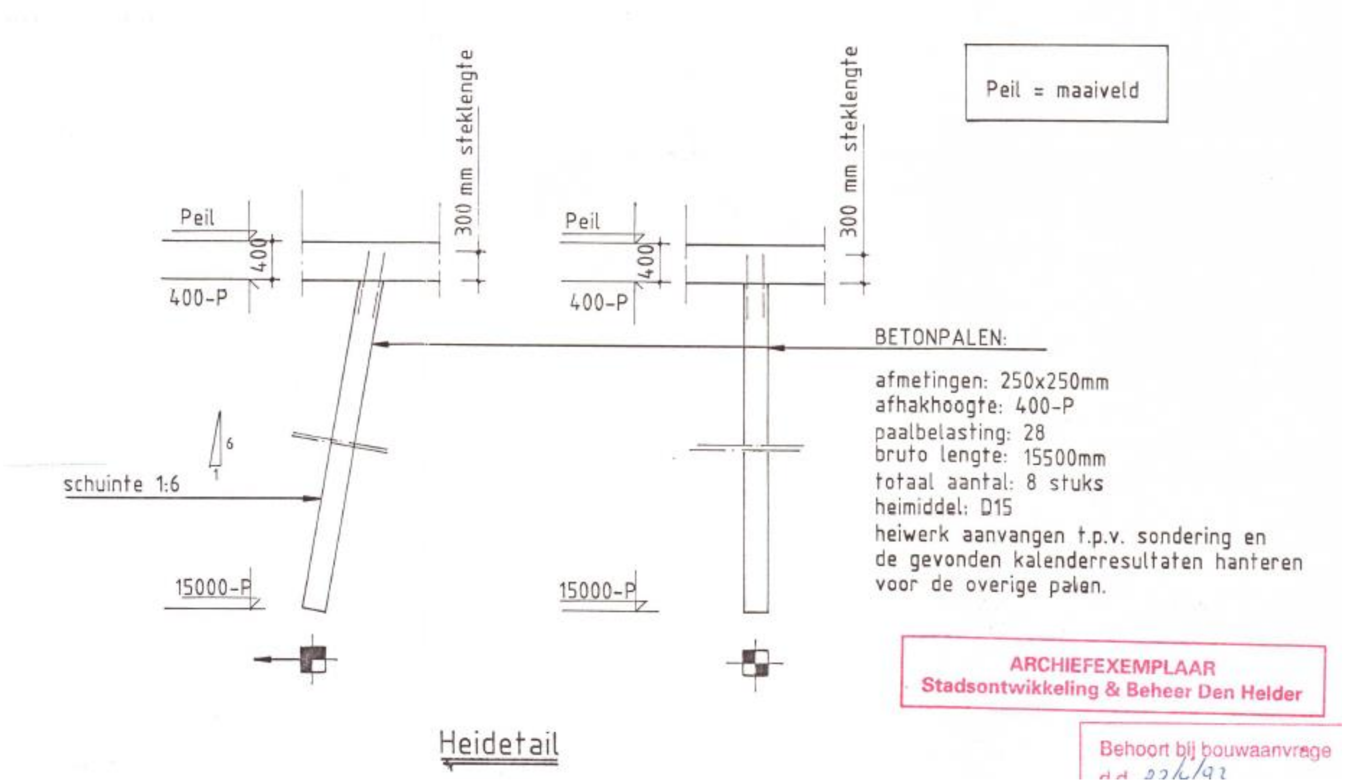


Figure 64: Pile foundation plan Nieuwe Werk 85 (Source: (Konstruktieburo J.S. Oud BV, 1992))

2

Berekening draagvermogen betonpalen $\diameter$ 220 x 220
paalpuntnivo 15 m - bestaand M.V. Sondering D3

Sondering nr. D11

Niveau	Conusw.
-8.00...	1.00
-8.20...	2.00
-8.40...	2.50
-8.60...	2.00
-8.80...	5.00
-9.00...	4.50
-9.20...	3.00
-9.40...	2.00
-9.60...	3.00
-9.80...	3.00
-10.00...	3.00
-10.20...	5.00
-10.40...	6.00
-10.60...	7.00
-10.80...	6.50
-11.00...	5.00
-11.20...	5.50
-11.40...	3.00
-11.60...	1.00
-11.80...	5.00
-12.00...	8.00
-12.20...	8.00
-12.40...	7.00
-12.60...	1.50
-12.80...	2.00
-13.00...	4.00
-13.20...	6.00
-13.40...	10.00
-13.60...	15.00
-13.80...	15.00
-14.00...	15.00
-14.20...	15.00
-14.40...	15.00
-14.60...	12.00
-14.80...	10.00
-15.00...	15.00
-15.20...	15.00
-15.40...	15.00
-15.60...	15.00
-15.80...	15.00
-16.00...	15.00
-16.20...	15.00
-16.40...	15.00
-16.60...	15.00
-16.80...	15.00

PAALGEGEVENS************
Materiaal : beton
Vervaardiging: prefab
Inbrengen d.m.v. heien

*Onzekerheidsfactoren:
Puntweerstand : 1.4
Positieve kleef: 1.4

Paaldoorsnede: vierkant
Paalmaat : 220 mm.
****

Sondering nr. D11

Niveau	Puntspanning
13.00	2.83
13.20	4.08
13.40	6.14
13.60	9.73
13.80	9.2
14.00	8.68
14.20	9.1
14.40	9.38
14.60	9.95
14.80	9.5
15.00	12.55

****Sondering nr. D11
****
PUNTWEERSTAND
Nadere gegevens.

Puntniveau: 15 - Peil
Gemidd. spanning onder
de punt: 15.0 mN/m2
Gemidd. spanning boven
de punt: 10.1 mN/m2

Onzekerh. f. n. pu = 1.4
****

****Sondering nr. D11
****
POSITIEVE KLEEF,
UIT CONUSWEERSTANDEN.
Nadere gegevens.

Positieve kleef gerekend
vanaf 13 m - Peil

Enkele paal.
Red. f. v. groepsw.: 1

Wrijvingsfactor i.g.v.
fijn zand, f.c: .75
Type zand: fijn.
Aangehouden gemiddelde
conusweerst. over 13 -
15m - Peil: 12.0 mN/m2

Onzekerh. f. n. p = 1.4
****

****Sondering nr. D11
****
NEGATIEVE KLEEF,
SLIPMETHODE
Nadere gegevens.

Enkele paal.
Red. f. v. groepsw.: 1

N. v. belasting : 10 kN/m2
Grondwaterniv.: 2 -mv

*LAAG 1
Laagdikte: 2 m
Soort: zand
Dichtheid: laag

*LAAG 2
Laagdikte: 2 m
Soort: klei en/of veen
Dichtheid: matig vast

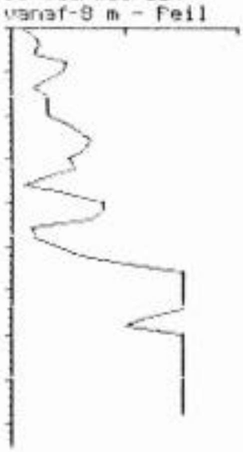
*LAAG 3
Laagdikte: 4 m
Soort: klei en/of veen
Dichtheid: slap

Omslagp. op 8 m - m.v.

****

OBJECT:
i.d.f.
Sondering nr. D11
Maaiveld is 0

Sondeerwaarden
vanaf-8 m - Peil



Paalmaat : 220 mm.
****
Aanlegniveau: 15 -Peil
Puntweerst.: 12.55 mN/m2

Enkele paal.
P. kleef v.a. 13 -Peil

KAR. DRAAGVERMOGEN
Paalpunt : 309.3 kN
Pos. kleef: 71.6 kN
Neg. kleef: 107.5 kN

*TOTAAL : 273.3 kN

Max optredende paalbelasting
274 KN.

Voor palen $\diameter$ 250 x 250

paalpunt. $\frac{250^2}{220^2} \times 309 = 323$ KN.
pos kleef $\frac{4 \times 250}{4 \times 220} \times 71.6 = 81$ KN.
neg kleef ... 4×250 ... $\times 107 = 121$

} 283 KN.

Figure 65: Cone penetration test results at Nieuwe Werk 85 (Source: (Fa. J. Beemsterboer en Zn. , 1990))

Appendix G: Hydraulic Boundary Conditions Section 13-4

Two hydraulic load databases are available for dike section 13-4. These 'HB databases' are intended to be used in combination with WBI2017 software to determine the hydraulic loads required for the assessment. The HB databases that have been made available are:

1. WBI2017_Hollandse_Kust_Noord_13-4_v03
2. WBI2017_Waddenzee_West_13-4_v03

The Water Helpdesk has advised using the WBI2017_Waddenzee_West_13-4_v03 database for the entire diked section, as the assumptions made in determining this database are the most compatible with the WBI2017.

Typical values for wave heights and water levels at the signalling value (category boundary A) and lower limit (category boundary B) for the North Sea and Wadden Sea based on marginal statistics are given in Table 21 (Hoogheemraadschap Hollands Noorderkwartier, 2021).

Table 21: Water levels and wave heights per category boundary along dike traject 13-4 (Source: (Hoogheemraadschap Hollands Noorderkwartier, 2021))

Water system	**Category boundary**	**Water level [m + NAP]**	**Wave height Hs [m]**
North Sea	A	4.26	2.72
	B	3.99	2.48
North Sea/ Wadden Sea	A	4.33	2.62
	B	4.06	2.42
Wadden Sea	A	4.38	1.39
	B	4.11	1.25

Appendix H: Piping Scenarios Calculations

All inputs used in calculating the different scenarios at the sites along the Nieuwe Diep are summarized in this section.

Table 22: Summary general scenario inputs case study Nieuwe Diep

General					
Scenario	**Scenario nr.**	**Shorter seepage path**	**Bigger hydraulic head**	**Lower soil pressure**	**Higher soil pressure**
		Exit point - x	Water level behind dike - x	Top soil layer(s) - x	Add layer kN/m2
Case study	*0*				
Pile foundation	*1*	5			
Barrier effect	*2*		0,1		
Digging foundation	*3*			0,8	
Digging foundation & pile foundation	*4*	5		0,8	
Digging foundation & pile foundation & barrier effect	*5*	5	0,1	4	
Shallow foundation	*6*			0,8	14,7

Table 23: Summary scenario inputs Paleiskade

Paleiskade					
Scenario	**Scenario nr.**	**Shorter seepage path**	**Bigger hydraulic head**	**Lower soil pressure**	**Higher soil pressure**
		Exit point - x	Water level behind dike - x	Top soil layer(s) - x	Add layer kN/m2
Case study	*0*				
Pile foundation	*7*	25			
Digging foundation & pile foundation	*8*	25		0,8	
Digging foundation & pile foundation & barrier effect	*9*	25	0,1	4	
Digging foundation and cellar & pile foundation	*10*	25		4	

Table 24: Summary scenario inputs Zwarte pad

Zwarte Pad					
Scenario	**Scenario nr.**	**Shorter seepage path**	**Bigger hydraulic head**	**Lower soil pressure**	**Higher soil pressure**
		Exit point - x	Water level behind dike - x	Top soil layer(s) - x	Add layer kN/m2
Case study	*0*				
Pile foundation	*7*	23			
Digging foundation & pile foundation	*8*	23		0,8	
Digging foundation & pile foundation & barrier effect	*9*	23	0,1	4	
Digging foundation and cellar & pile foundation	*10*	23		4	

Table 25: Summary scenario inputs Nieuwe Diep

Nieuwe Diep Scenario	**Scenario nr.**	**Shorter seepage path**	**Bigger hydraulic head**	**Lower soil pressure**	**Higher soil pressure**
		Exit point - x	Water level behind dike - x	Top soil layer(s) - x	Add layer kN/m2
Case study	*0*				
Pile foundation	*7*	21			
Digging foundation & pile foundation	*8*	21		0,8	
Digging foundation and cellar & pile foundation	*9*	21		4	

Table 26: Summary scenario inputs Nieuwe Werk

Nieuwe Werk Scenario	**Scenario nr.**	**Shorter seepage path**	**Bigger hydraulic head**	**Lower soil pressure**	**Higher soil pressure**
		Exit point - x	Water level behind dike - x	Top soil layer(s) - x	Add layer kN/m2
Case study	*0*				
Pile foundation	*7*	23			
Digging foundation & pile foundation	*8*	23		0,8	
Digging foundation & pile foundation & barrier effect	*9*	23	0,1	4	
Digging foundation and cellar & pile foundation	*10*	23		4	

Appendix I: Piping Scenarios Results

All results of the calculations of the different scenarios at the sites along the Nieuwe Diep are summarized in this section.

Table 27: Summary scenario results Paleiskade

Scenario summary Paleiskade	Scenarios					
Sub mechanism	0	1	2	3	4	
Hydraulic fracture	6,24E-05	6,24E-05	1,05E-04	1,46E-03	1,44E-03	
Heave	1,26E-02	1,26E-02	1,73E-02	6,72E-02	6,67E-02	
Backward erosion	1,66E-07	1,28E-06	3,94E-07	1,08E-06	7,03E-06	
Piping overall	1,66E-07	1,28E-06	3,94E-07	1,08E-06	7,03E-06	
Scenarios	5	6	7	8	9	10
Hydraulic fracture	1,00E+00	7,79E-06	6,24E-05	1,44E-03	1,00E+00	1,00E+00
Heave	1,00E+00	8,44E-03	1,26E-02	6,67E-02	1,00E+00	1,00E+00
Backward erosion	2,06E-04	9,80E-08	4,56E-11	4,69E-10	5,49E-08	2,99E-08
Piping overall	2,06E-04	9,80E-08	4,56E-11	4,69E-10	5,49E-08	2,99E-08

Table 28: Summary scenario results Zwarte Pad

Scenario summary Zwarte Pad	Scenarios					
Sub mechanism	0	1	2	3	4	
Hydraulic fracture	5,75E-05	5,75E-05	6,74E-05	4,26E-04	4,26E-04	
Heave	2,30E-03	2,30E-03	2,59E-03	6,33E-03	6,33E-03	
Backward erosion	2,61E-07	1,49E-06	3,80E-07	1,54E-06	7,78E-06	
Piping overall	2,61E-07	1,49E-06	3,80E-07	1,54E-06	7,78E-06	
Scenarios	5	6	7	8	9	10
Hydraulic fracture	6,73E-01	4,26E-04	5,75E-05	4,26E-04	6,88E-01	6,58E-01
Heave	5,17E-01	6,33E-03	2,30E-03	6,33E-03	5,33E-01	5,01E-01
Backward erosion	6,49E-04	1,54E-06	3,19E-10	2,80E-09	1,45E-06	9,16E-07
Piping overall	6,49E-04	1,54E-06	3,19E-10	2,80E-09	1,45E-06	9,16E-07

Table 29: Summary scenario results Nieuwe Diep

Scenario summary Nieuwe Diep 34	Scenarios				
Sub mechanism	0	1	2	3	4
Hydraulic fracture	1,57E-02	1,57E-02	1,77E-02	7,25E-02	7,25E-02
Heave	7,56E-02	7,56E-02	8,25E-02	1,51E-01	1,51E-01
Backward erosion	5,41E-05	1,14E-04	7,23E-05	1,07E-04	2,19E-04
Piping overall	5,41E-05	1,14E-04	7,23E-05	1,07E-04	2,19E-04
Scenarios	5	6	7	8	9
Hydraulic fracture	1,00E+00	4,75E-03	1,57E-02	7,25E-02	1,00E+00
Heave	9,97E-01	6,36E-02	7,56E-02	1,51E-01	9,97E-01
Backward erosion	1,94E-03	4,51E-05	1,42E-03	2,45E-03	1,26E-02
Piping overall	1,94E-03	4,51E-05	1,42E-03	2,45E-03	1,26E-02

Table 30: Summary scenario results Nieuwe Werk

Scenario summary Nieuwe Werk	Scenarios					
Sub mechanism	0	1	2	3	4	
Hydraulic fracture	3,31E-07	3,31E-07	6,78E-07	4,73E-06	4,73E-06	
Heave	1,01E-04	1,01E-04	1,71E-04	4,38E-04	4,38E-04	
Backward erosion	7,48E-08	2,57E-07	1,63E-07	4,44E-07	1,42E-06	
Piping overall	7,48E-08	2,57E-07	1,63E-07	4,44E-07	1,42E-06	
Scenarios	5	6	7	8	9	10
Hydraulic fracture	2,37E-01	4,87E-08	3,31E-07	4,73E-06	2,37E-01	1,96E-01
Heave	5,63E-01	7,12E-05	1,01E-04	4,38E-04	5,63E-01	5,09E-01
Backward erosion	1,77E-04	4,49E-08	4,96E-10	3,91E-09	1,56E-06	1,01E-06
Piping overall	1,77E-04	4,49E-08	4,96E-10	3,91E-09	1,56E-06	1,01E-06