Master of Science Thesis

Design and reliability evaluation of a glass flood defence

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MSc. Thesis

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Artist impression on cover page retrieved from E. Böhtlingk (2016)

Preface

The graduation thesis 'Design and reliability evaluation of a glass dike', which is the final result of my master study in Hydraulic Engineering at the Technical University of Delft, is presented in this report. The philosophy behind a transparent dike is to establish a visual connection between land and water. The idea originates from writer and artist Paul Izeboud and is elaborated in an architectural design by Eduard Böhtlingk, who is currently focussed on realization of the project. Based on the architectural design, a first engineering design of the glass dike is given in this report and the reliability of the glass dike is evaluated.

This research could not have been realized without the contribution of many people. I would especially like to express gratitude to the members of my graduation committee, Bas Jonkman, Mark Voorendt, Rob Nijsse and Babette Lassing-van der Spek, for their guidance throughout the process and for their useful feedback.

I would like to thank Jos Wessels for offering me the opportunity to perform this research at TNO by means of a graduation internship. I would also like to thank all other colleagues at TNO's Structural Reliability department for sharing their knowledge and making my time at TNO a pleasant experience.

Lastly, I would like to thank my family for their unconditional support.

Delft, April 2016
Jorrit Kentrop

Abstract

Typical for the Dutch landscape are the many polders, which are situated below sea level and below waters directly surrounding these polders. The flood defences that protect the polders from flooding cause a visual separation between land and water. In 2005, artist Paul Izeboud came with the idea to make a transparent flood defence, thereby making it possible to experience land and water simultaneously, without standing on top of the dike. In 2011, Eduard Böhtlingk elaborated this idea in an architectural design of the glass dike and is currently focussing on realisation of the glass dike inside a regional flood defence in the province of South-Holland that protects the Duifpolder from flooding. However, many impediments are encountered when realizing an innovative flood defence such as the glass dike. Apart from technical issues, aspects like governance, finances and legislations are very important to consider. Discussions about these aspects are started more easily if technical considerations are addressed first. For a large part, this is done in this thesis. Based on the architectural design, a first engineering design of the glass dike is made. After this, the reliability of the glass dike as a whole and the glass elements in particular are evaluated.

Engineering design

The glass dike is a hollow structure next to a channel. Since the weight of the structure is too low to prevent uplift, a pile foundation is applied. When the water level is low, these piles are loaded in compression instead of tension. In order to resist both tension and compression loads, steel tubular piles with concrete fill are applied. These are placed in inclined position, thereby obtaining sufficient resistance against sliding of the structure due to the large hydrostatic pressure. Piping is prevented by placing seepage screens under- and at the sides of the structure. At the connections with adjacent dike sections, these seepage screens are also used to increase the stability of the soil body. The roof is constructed with HE300B steel beams. This way, a large bending moment can be resisted with a limited thickness. Semi-probabilistic design calculations are performed for the strength and stability of the glass dike. This is done with partial factors corresponding to Eurocode reliability class 2. An impression of the engineering design is given in Figure 1.

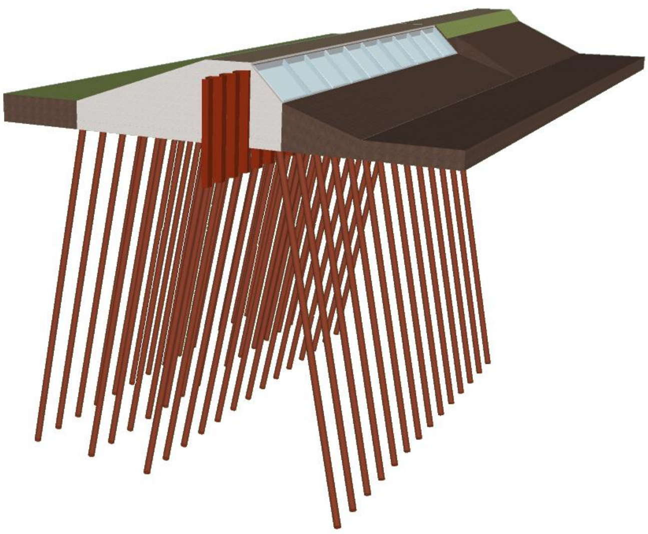

Figure 1: Impression of the engineering design of the glass dike

Reliability evaluation

In order to compute the failure probability of the glass dike as a whole, a fault tree for structural failure of the glass dike is proposed. All failure mechanisms in this fault tree were assumed to be fully independent. This in contrast to the less conservative approach described in LRK2011 (Guideline for water retaining structures in regional flood defences), in which full dependency is assumed between some of the failure mechanisms. Instead of computing the probabilities of all failure mechanisms, estimates were made. Since for most of the failure mechanisms semi-probabilistic design calculations were performed with partial factors from Eurocode reliability class 2, probabilities of these failure mechanisms were assumed to be equal to the maximum allowed failure probability corresponding to this Eurocode reliability class ($p_{f;\,max} = 1{,}3\cdot10^{-6}$). Only for piping, pipe leakage and outflanking, higher values were assumed ($p_{f;\,max} = 1{,}0\cdot10^{-5}$). It appeared that, with these 'hypothetical failure probabilities', the glass dike satisfies

the Flood Safety Standard. Moreover, considerable failure probability space is left over for the glass elements ($p_{f,max} = 5{,}0 \cdot 10^{-5}$). However, since the maximum allowed failure probability that follows from the Eurocode ($p_{f,max} = 1{,}3 \cdot 10^{-6}$) is more strict, it is governing for the design of the glass elements.

Probabilistic design of the glass elements

Due to the brittle failure behaviour of glass, it is not very attractive for use in structural applications. If the maximum admissible tensile stress is exceeded, it fractures without warning. Its strength is subject of many researches, but no good statistical descriptors are found yet. The reason for this, is the large amount of parameters that must be considered. Furthermore, the strength of glass is not solely a material property but depends to a large extend on the quality of the production process. The structural performance can be improved by applying a heat-treatment to the glass, which induces compressive pre-stresses at the surface. It is therefore chosen to apply heat-strengthened (pre-stressed) glass. In order to introduce some robustness, each glass element is composed of four laminated glass plies. If one ply breaks, the load is transferred to the other glass plies.

For glass elements in the glass dike, four important threats are identified: Overloading, impact, fire and explosions. Overloading is caused by hydrostatic pressure, which is the only load that is present under normal conditions. For this failure mechanism, a probabilistic model is proposed and a number of Monte Carlo simulations are performed. This resulted in a sufficient low failure probability for laminated glass elements with a thickness of 53 mm.

For impact, explosions and fire, no probabilistic calculations are performed as only very complex models can provide answers with reasonable accuracy. However, a deterministic calculation was performed for impact due to a boat collision. It was found that a second row of glass elements that can take over the water retaining function, in case the first row of glass elements has failed, is required. Moreover, it is desired to prevent boat collisions by means of a protective structure in front of the glass dike. For explosions, fire and impact due to vandalism, a qualitative analysis is performed. Again, it is found that a second row of glass elements is required. Since glass is a poor structural material with respect to fire, measures to prevent this and to reduce consequences are desired. More research should be performed on this subject.

Generalization

In the last part of the thesis, the lessons learned from the glass dike study were used to say something about the considerations that play a role for glass flood defences in general. For primary flood defences, the Flood Safety Standard will be changed in 2017. For glass flood defences, the design process will become more practical as calculated failure probabilities can be compared to the Flood Safety Standard more directly. However, if only part of a dike trajectory is replaced by a glass flood defence, the probability of a breach should be computed for the entire trajectory. For regional flood defences, the Flood Safety Standard will continue to prescribe a design water level with a certain exceedance probability. This needs to be translated to a maximum failure probability. Design guidelines can be used for this.

The required reliability that follows from the Eurocode is evaluated in a different way than the reliability as prescribed by the Flood Safety Standard. The latter demands an integral reliability for the water retaining structure as a whole, while the Eurocode sets reliability requirements to individual failure mechanisms. For the design of the glass elements, it was found that the Eurocode is governing in almost all flood defences. For primary flood defences, the Flood Safety Standard is only more strict for dike trajectories with a relative large number of water retaining structures and for which a very small maximum probability of flooding is prescribed, in the order of 1/33.000 per year. As a result of the generally stricter Eurocode reliability requirements, possibilities to adjust failure probability spaces and to optimize the design with respect to the Flood Safety Standard are limited.

The required glass thickness and additional measures in order to reduce threats like a boat collision or fire depends on the required reliability. Stricter requirements lead to thicker glass and/or more protective measures. Hydraulic boundary conditions also play a large role, especially the water depth that has to retained by the glass. In general, realization of a glass flood defence is technically feasible.

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Appendices

Part I

General backgrounds

1. Introduction

When designing and constructing hydraulic structures, most often ‘proven technologies’ are chosen. These are technologies which already are widely used and for which a lot of knowledge and experience is available (De Groot, 2015). Since failure of a hydraulic structure can have large consequences to society, risk aversion is the main reason to prefer proven technologies. To assist the designer and reduce risks, the government provides technical guidelines. It is common practice to design and assess hydraulic structures based on these guidelines. They can only be produced for proven technologies, for which the reliability has been validated in practice. For new and innovative technologies, the availability of guidelines is very limited. Although it is possible to apply innovative technologies in the design of hydraulic structures, the lack of technical guidelines makes it hard to evaluate the design of these structures. On the other hand, due to environmental changes on aspects like the economy, society, and climate, the demand for innovations is increasing.

One of the projects for which above mentioned situation certainly is the case, is the glass dike based on the idea of Paul Izeboud (2005) and architectural design of Eduard Böhtlingk (2011). This is a structure with glass on both the inner- and outer side of the flood defence. On the outer side the glass has a water retaining function. The structure has the same shape as the adjacent soil bodies and provides room for a restaurant and educational facility on the inside. The main philosophy behind the idea is that land and water can be experienced simultaneously, without standing on top of the dike. This also increases the awareness that a substantial part of the land area in the Netherlands is situated below sea level (De Architect, 2014).

Figure 2: Cross-section of glass dike (De Architect, 2014)

Usually, hydraulic structures in flood defences in the Netherlands are built using concrete, steel and soil. Using glass as material for the water retaining elements is very unusual. Although glass is used as a structural material in some ‘dry’ structures, little is known about the reliability when using it as structural material in a flood defence. When designing dikes and water retaining structures, some common failure mechanisms are assessed, such as piping and instability. For innovative structures, such as the glass dike, also other (unknown) failure mechanisms could be of relevance. Realization of such an innovative hydraulic structure as part of a flood defence is only possible if it can be shown that the design meets applicable safety standards.

Initiator Eduard Böhtlingk would like to realize the glass dike in a regional flood defence in the polder of Midden-Delfland. The exact location is proposed at the last part of the Duifpolderkade before this takes a turn and becomes the Schouwkade. It will be part of the flood defence that retains water from the Noordvliet, thereby protecting the Duifpolder. The difference between the water level and the polder is approximately 3 meters.

Figure 3: Proposed location of the glass dike

Many aspects of the glass dike are similar to other hydraulic structures. For this, part of the possible failure mechanisms are also similar. For some other aspects, design- and assessment calculation methods can deviate from the methods described in guidelines. Using glass as material to permanently retain water is very unusual in flood defences. In this thesis, the architectural design is upgraded to a technical hydraulic design. For this upgraded design, a reliability analysis is performed. The lessons that are learned from this study are used to say something about the considerations that play a role for glass flood defences in general.

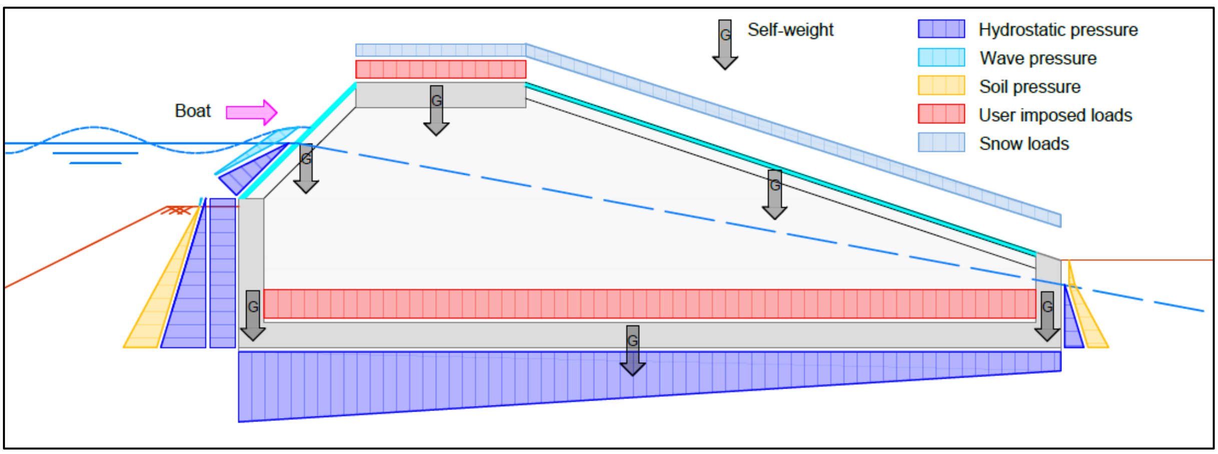


Figure 4: Schematization of loads in a cross-section of the glass dike

2. Research description

2.1 Problem analysis

The glass dike is a very unusual structure in a flood defence. In the field of flood protections, using glass as material for the water retaining elements is an innovative solution. Impediments related to application of innovative technology in flood defences are described in section 2.1.1. Besides the primary function of retaining water, it will also fulfill other functions. The most important secondary function is exploitation for recreational and/or educational facilities. In this sense, it is a multifunctional flood defence. Multifunctional flood defences and related impediments for realization are described in section 2.1.2. In section 2.1.3, the impediments for realization of innovations and multifunctional flood defences are elaborated for the case of a glass dike. Furthermore, in this section some first technical design challenges are identified. Finally, in section 2.1.4, a problem definition for the research is given.

2.1.1 Innovations in flood defences

Innovation can refer both to technical aspects as to processes. Technical innovations could be improved construction methods, new materials or new products (Knoeff, et al., 2013). A technology that has been used extensively in some field of practice could be new to another field of practice. Then, for the latter it could be regarded as an innovation.

If there is a failure somewhere in the flood defence system, consequences like economic damage or even human casualties, can be substantial. Therefore, the technical reliability plays a dominant role in the design process. This is one of the reasons to prefer technologies that have been tested and validated in practice extensively. As a consequence, only a little number of innovations find their way to practice. Furthermore, flood defences must meet flood safety standards, which are set by the government. To accomplish this, most flood defences are designed and evaluated by using technical guidelines. These guidelines are also based on tests and experience with existing technologies. For application of innovative technologies, technical guidelines are little helpful. Testing and validating new technologies is very costly and takes time, making it unattractive to realize flood defences with innovative technology. Below, a summation is made of the most important impediments for applying innovative technology in flood defences:

- In the design of most flood defences, the reliability is evaluated with help of technical guidelines. It is difficult, time consuming and costly to deviate from this procedure;
- Rules for assessment are essential to properly evaluate the state of the flood defence at some points during the lifetime. For innovative technology, these rules have to be developed;
- The required maintenance might be different for different technologies. This will complicate the maintenance work for the total flood defence system;

2.1.2 Multifunctional flood defences

Many flood defences are multifunctional in the sense that they fulfill more functions than just retaining water. Think of all the dikes with a road on top of the crest or all the houses that are built partially within a dike. However, a house does not have a water retaining function by itself. In the Netherlands, this is called a ‘non water retaining object’. These objects can have an influence on the reliability of the flood defence, but do not contribute to the water retaining function. For this type of multifunctional use, some guidelines are available to evaluate the influence of the object on the reliability of the flood defence. This way, dike managers are able to efficiently determine which types of shared use can be permitted. In this research, a flood defence is called multifunctional, if objects and/or elements that directly or indirectly fulfill (part of) the primary function (providing a physical boundary that prevents water to flow into the polder), also fulfill secondary functions. The possibilities for realization of multifunctional flood defences is primarily determined by the room in safety policies of flood defence managers (Ellen, et al., 2011). The design of multifunctional flood defences is often different from designs of non-multifunctional flood defences. This introduces some technical challenges. However, also financial, spatial and juridical aspects can introduce impediments for realization of multifunctional flood defences. The most important are summarized in Table 1.

Financial	It might be necessary to reinforce the flood defence at some point in the future. Changed flood safety standards could be a reason for this. If parties fail to make agreements about costs concerning adaptation of the flood defence, the whole project could be canceled.
	Many flood defence managers only allow multifunctional use if the extra costs of realization, maintenance and inspection due to the multiple use are proportional and can be socially justified. In other words, they only allow it if there is sufficient need to do so.
	By law, the owner of the flood defence is liable for damage that could occur. Since multifunctional use will increase the risk of damage, the owner will be reluctant to share the flood defence for multifunctional use.
	Projects which involve multifunctional flood defences require an integral approach. Flood defence managers have little experience with this. It might be necessary for them to invest in knowledge and skills on this subject.
	Since it is difficult to weigh benefits against extra costs, it is also difficult to distribute them fairly over different stakeholders.
Spatial	Multifunctional use of flood defences is often only possible through co-operation with private parties. These parties have more interests exploitation than flood safety. For flood defence managers, this is a threatening situation.
	Multifunctional flood defences can contribute to the quality of space. In some cases however, spatial quality decreases due to a large number of compromises that must be made to satisfy all regulations and boundary conditions (Klijn & Bos, 2010).
Juridical	The responsibility of flood defence managers is to maintain a certain flood safety level. Policies to accomplish this, leave little room for multifunctional use.
	Policies for large rivers in the Netherlands strongly restricts multifunctional use of flood defences. The main reason for this is to prevent damage due to flooding of the riverbed.
	Besides the Water Act, some multifunctional flood defences also have to comply with the Housing act. The Housing Act prohibits a governmental body to make private agreements that are contrary to criteria in public construction regulations.
	Governance of multifunctional flood defences deviates from the governance of 'normal' water retaining structures. It is difficult to make a distribution of tasks for the involved parties.
Technical	Since objects in flood defences complicate assessment and frequent inspections, flood defence managers are reluctant to provide permits for multifunctional use.
	Multifunctional flood defences are more sensitive to failure due to human error
	Due to the non-typical character of multifunctional flood defences, safety evaluation in the design of these flood defences is difficult, costly and time consuming.
	Due to the non-typical character of multifunctional flood defences, it is hard to assess the safety of the flood defence at some point during the lifetime.

Table 1: Impediments for multifunctional flood defences. Partly based on (Ellen, et al., 2011).

2.1.3 Glass dike

The primary function of the glass dike is retaining water. The idea behind a design of glass is the possibility to use it for recreational and educational purposes as well. For this, it can be regarded as some form of multifunctional flood defence. Using glass as a structural element in a flood defence is very unusual. For this, it can be characterized as innovative technology. Some of the impediments stated in above sections also play a role when realizing a glass dike. In Table 2, relevant impediments for realization of a glass dike will be extracted from above sections and explained. It should be noted that some impediments for innovative technology are similar to technical impediments. It is chosen to only show these under innovative technology.

Innovative technology	For reliability evaluation of the glass dike, deviation from technical guidelines is necessary. This is difficult, time consuming and costly.
	Rules for assessment are essential to properly evaluate the state of the glass dike at some points during the lifetime. For the glass dike, these rules have to be developed.
	The required maintenance for the glass dike is different than for 'normal' flood defences. This will complicate the maintenance work for the total flood defence system.
Financial	It might be necessary to upgrade the glass dike at some point in the future. Changed flood safety standards could be a reason for this. If parties fail to make agreements about costs concerning adaptation of the glass dike, the whole project could be canceled.
	Many flood defence managers only allow multifunctional use if the extra costs of realization, maintenance and inspection due to the multiple use are proportional and can be socially justified. In other words: the glass dike should add value to society.
	By law, the owner of the flood defence is liable for damage that could occur. Since multifunctional use will increase the risk of damage, the owner could be reluctant to share the flood defence for realization of a glass dike.
	Projects which involve multifunctional flood defences require an integral approach. Flood defence managers have little experience with this. It might be necessary for them to invest in knowledge and skills on this subject.
	Since it is difficult to weigh benefits against extra costs, it is also difficult to distribute them fairly over different stakeholders.
Spatial	Realization of the glass dike is possible through co-operation with private parties. These parties have more interests in exploitation than flood safety. For the flood defence manager, this is a threatening situation.
Juridical	The responsibility of flood defence managers is to maintain a certain flood safety level. Policies to accomplish this, leave little room for multifunctional use.
	Besides the Water Act, the glass dike also has to comply with the Housing act. The Housing Act prohibits a governmental body to make private agreements that are contrary to criteria in public construction regulations.
	Governance of the glass dike deviates from the governance of ‘normal’ water retaining structures. It is difficult to make a distribution of tasks for the involved parties.
Technical	Since the glass dike complicates assessment and frequent inspections, the flood defence manager could be reluctant to provide permits.
	The glass dike is more sensitive to failure caused by human errors, e.g. improper alterations to the structure by the owner/user.

Table 2: Impediments for realization of a glass dike

<u>Design challenges</u>

The technical impediments stated above are more focused on uncertainties and technical processes, rather than real technical problems/challenges. In order to properly define the goals of this research, in this section some first ‘real’ technical issues with respect to the design of a glass dike are identified. It is very likely that more technical issues will reveal itself during the design process.

- Effects of (unusual) hard structures in soft soils are difficult to predict. This makes it hard to quantify the reliability of the glass dike.
- The points where hard structures and soft soils are connected are most sensitive to erosion. For the glass dike, this might be an issue to be solved.
- Intuitively, glass as water retaining elements seems sensitive to vandalism, terrorist attack or boat collisions. This should be investigated.
- It is unclear what should be done if a glass element is damaged.
- The glass elements have to stay transparent to fulfill the purpose it is designed for.
- Glass is known for its brittle failure behavior. Technical solutions or possibly special types of glass need to be investigated.
- The glass elements must be safely connected to the structure and to each other, without leakage.

2.1.4 Problem definition
Ellen et al (2011) concluded that the impediments stated in section 2.1.2 are mainly issues of (dis)trust between different stakeholders. If they co-operate from the beginning, it is possible to make agreements to overcome most of these issues. In the report it is also concluded that if it is shown that realizing a reliable multifunctional flood defence is technical possible, the dialog about nontechnical issues is more easily opened. This is especially of importance when innovative technology is applied. For the glass dike, this is very recognizable. Showing the technical feasibility of the glass dike would be a large step toward realization. Taking away technical uncertainties is only possible through development and reliability evaluation of an engineering design. The most important problem for this is the lack of suitable technical guidelines. Above mentioned leads to the following problem definition:

The applicability of technical guidelines for the design and reliability evaluation of innovative flood defences is limited. As a result, it may occur that innovative projects like the glass dike, are not carried out.

2.2 Objectives and research questions

2.2.1 Objectives
The main goal of this research is to develop an engineering design of the glass dike, based on the idea of Paul Izeboud (2005) and architectural design of Eduard Böhtlingk (2011), that satisfies applicable safety standards. As a side product, there should be an evaluation of impediments in the engineering design process and advices on how to handle these in similar projects.

2.2.2 Research questions
Two research questions are necessary to comprehend the intended goal of the research:

(1) *Based on an existing architectural design, which engineering design of the glass dike satisfies applicable safety standards?*

(2) *What are the problems that are encountered when technical guidelines for flood defences and building codes are used to design a multifunctional glass flood defence and what advices could be given to overcome these problems?*

2.3 Work approach

In order to answer the main research questions and thereby accomplish the goals of this research, the project is divided in several sub-questions. These sub-questions and the approach to answer these sub-questions are described in this section.

The outline of this thesis is narrowed down through an investigation of literature. First, some general aspects regarding flood defences in the Netherlands are elaborated. The different types, management and safety standards are discussed. It is important to also include expected changes in legislation. The results will answer sub-question 1. Sub-question 2 deals with possible reliability evaluation methods, in case of innovative flood defences. In sub-question 3, the possibilities and problems when using glass as a structural element will be discussed. Possible gaps in the knowledge will be identified.

(1) *What are the different types of flood defences? Which laws, standards and technical guidelines for the design and reliability evaluation are applicable and what future changes can be expected?*
(2) *What method(s) can be applied to evaluate the reliability of an innovative flood defences?*
(3) *What consideration play a role when using glass as structural elements and what knowledge is still missing?*

Before the reliability in the design of the glass dike can be evaluated, there should be a first design with technical solutions. For this, technical solutions must be proposed and design checks must be performed. This is done by using a combination of technical guidelines for flood defences and building codes. When possible, a semi-probabilistic method will be applied. In order to dimension the glass elements, the knowledge gathered in the literature study will be used. An answer will be given to sub-question 4. The engineering design must satisfy applicable safety standards. In technical guidelines and building codes, a semi-probabilistic approach (with partial factors) is prescribed to accomplish this. For innovative technology, no partial factors are prescribed and an alternative method should be used. The reliability of the glass dike will be evaluated with probabilistic methods.

(4) *What technical additions and changes to the architectural design of the glass dike should be implemented in order to prevent failure mechanisms and/or reduce failure probabilities?*
(5) *What optimizations should be implemented in order to obtain a design that satisfies applicable safety standards?*

The design and reliability evaluation of a glass dike will give insight into the problems that are encountered when designing and evaluating glass flood defences in general. In the last stage of this research, a generalization of the considerations that play a role when designing and evaluating the reliability of glass flood defences is made. The following sub-questions will be answered:

(6) *Based on the glass dike, what considerations play an important role when designing- and evaluating the reliability of glass flood defences in general?*

2.4 Report overview

In Figure 5, the relation between the research questions and the structure of the report is shown.

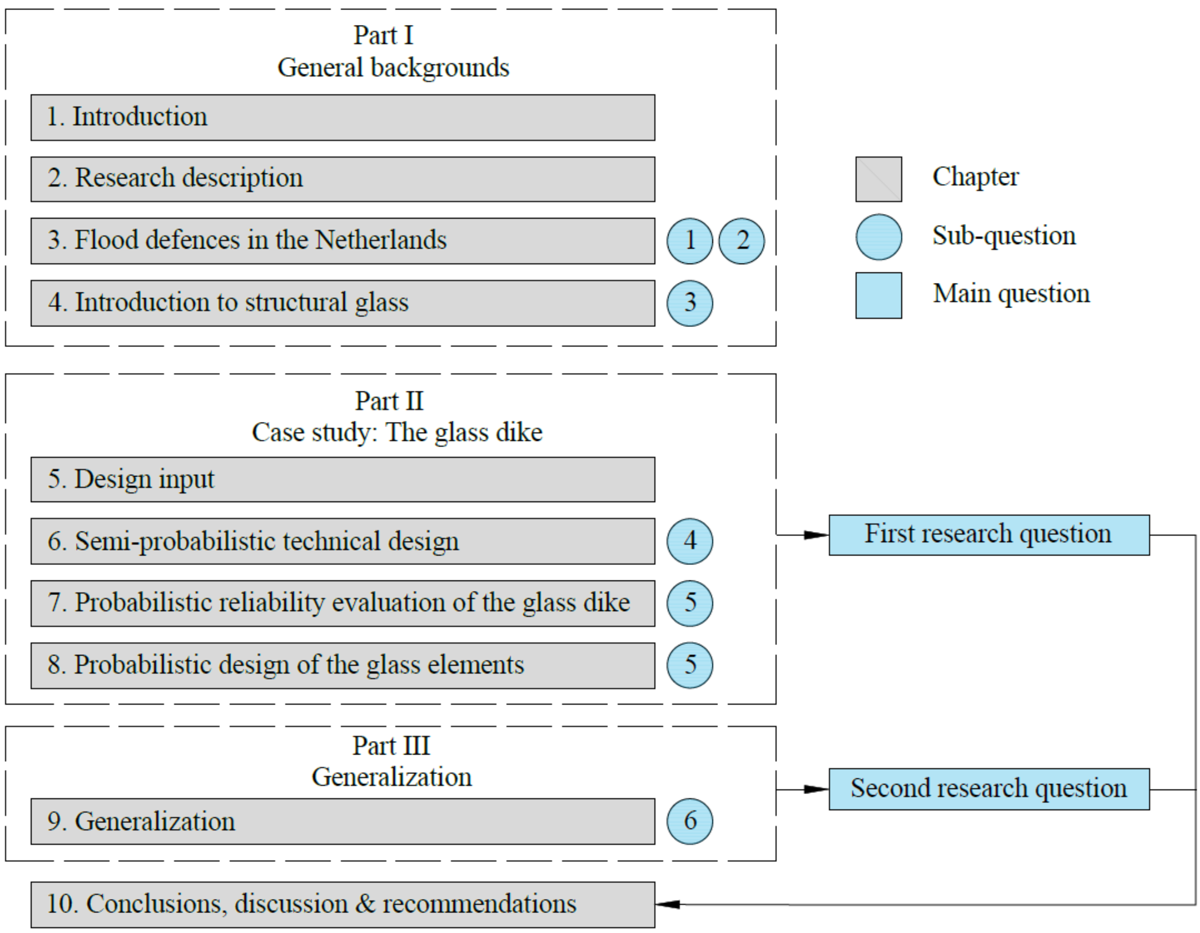


Figure 5: Structure of the report

3. Flood defences in the Netherlands

In the Netherlands, a substantial part of the land area lies below sea level. The population in this area is large and still growing. Also, a large part of economic activities takes place in this area. Drainage and pump systems are used to keep the polders dry. However, drainage causes subsidence, resulting in even lower surface levels of the land. On the other hand, the sea level is rising. This means that hydraulic loads on flood defences are increasing, while the resistance is decreasing and consequences of floods are increasing. This leads to higher flood risks. Protection against floods has been an important concern to society for a large part recent history. All this makes that flood defences are an important part of the Dutch landscape and that they are intertwined with all kinds of activities in society. This chapter describes the different types of flood defences. Also the management of flood defences and regulations to accomplish and maintain required safety levels are described.

3.1 Classification

In the Netherlands, a distinction is made between primary and regional flood defences. The definitions of both types are given below (Water Act, 2009):

Primary flood defence: Water retaining body or structure of frontline water.
Regional flood defence: Water retaining body or structure of inland water.
Frontline water: Surface water with an uncontrollable water level, like major rivers, lakes and the sea.
Inland water: All surface water bodies except frontline water.

3.1.1 Classification by category

Simply stated, the function of a flood defence is to retain water. However, (implicitly) based on the consequences in case a flood defence loses its function, a differentiation of categories can be made as described below (TAW, Grondslagen voor Waterkeren, 1998):

1. The flood defence belongs to a system that surrounds a dike ring area and directly retains frontline water;
2. The flood defence belongs to a system that surrounds a dike ring area, but does not directly retain frontline water;
3. The flood defence is located in front of a dike ring area and retains frontline water;
4. The flood defence is located in front of a dike ring area, but does not retain frontline water;

Since failure of a category 1 flood defence directly leads to flooding of the dike ring area, this is the most critical category. A category 2 flood defence can function as a backup in case a category 1 flood defence fails or it retains water that is (occasionally) disconnected from frontline water by another (movable) flood defence. Category 3 flood defences retain frontline water, but no land is situated behind it. Instead, there is inland water. The function of this category of flood defences is to reduce loads on category 2 flood defences. The same holds for category 4 defences. However, this category retains inland water instead of frontline water.

For regional flood defences, a classification as elaborated above is not applicable. These flood defences can better be classified as follows:

1. The flood defence retains water from regional rivers or polder discharge channels;
2. The flood defence does not permanently retain water, but fulfills a backup function;
3. The flood defence retains frontline water, but is not a primary flood defence;

Examples of category 3 regional flood defences are foreland barriers and summer (river) dikes. Category 2 of regional flood defences is similar to category 2 of primary flood defences. These flood defences are meant to reduce damage in case a flood defence that directly retains water fails.

3.1.2 Classification by type

Depending on the character of the water and therefore hydraulic loads that have to be resisted different types of flood defences are appropriate. This also depends on secondary functions that have to be fulfilled. A summation of different types is given below (TAW, Grondslagen voor Waterkeren, 1998):

- Dunes
- Higher land areas
- Dikes
- Special structures
- Water retaining hydraulic structures

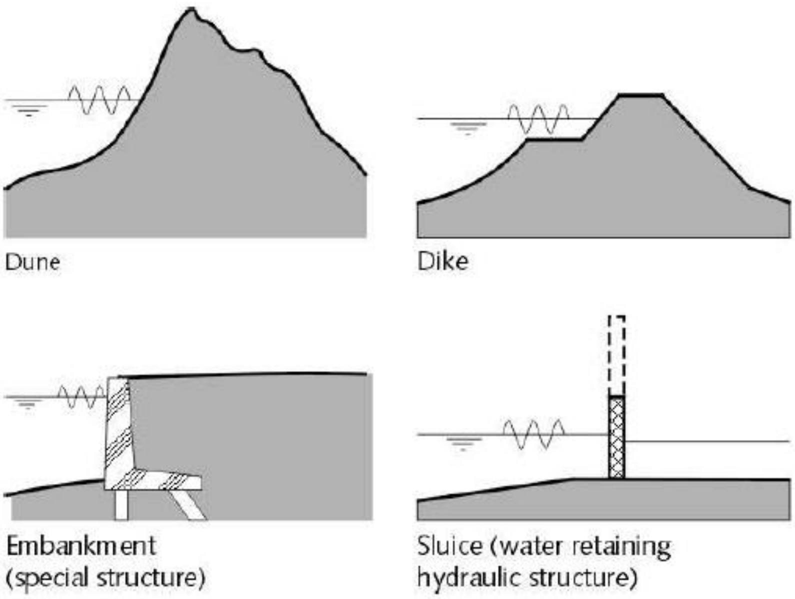


Figure 6: Types of flood defences

Dunes, dikes and higher land areas consist of soils like sand and clay. Dunes and higher land areas are formed by nature, while dikes are men made. However, the maintenance of dunes is partly conducted by men. Most of the flood defences consist of soil. These are cheap and easy to construct.

Special structures could be cofferdams, sheet pilings, concrete retaining walls and other hard structures. These types are often applied when the available space is limited or when there is a need to with respect to functions like ship mooring. Water retaining hydraulic structures are structures like locks, weirs and tidal flood barriers. Tidal flood barriers are closed during high water levels, while weirs in rivers are closed when discharges are low. Locks are also meant to maintain a surface water level difference. These three examples share the property that they also have to fulfill a ship navigation function. To accomplish this, they include movable parts. Another example of a water retaining hydraulic structure is a pumping station. In short, this type is characterized as flood defences that are made for other utilitarian functions that crosses the flood defence.

Not rarely, it is difficult to make a clear distinction between the mentioned type of flood defences. In a lot of cases, a combination of more than one type is applied. This is often a combination of a soil body and special structures or water retaining hydraulic structures. Sometimes even buildings can fulfill a water retaining function. Then in fact, the building is a special structure in a flood defence. (TAW, Grondslagen voor Waterkeren, 1998). A multifunctional flood defence as described in the problem analysis is such a structure.

3.2 Management

The total length of flood defences in the Netherlands is estimated to be 3.800 kilometers for primary flood defences and 14.000 kilometers for regional flood defences (Inspectie Verkeer en Waterstaat, 2011) & (LRK2011). Safety against floods must be guaranteed for all these flood defences. To accomplish this, a framework of regulations, responsible organizations and agreements is essential. This section describes this framework. First the relevant life cycle stages of flood defences are described.

3.2.1 Lifecycle phases

The phases in the lifetime of flood defences can be differentiated according to the stages shown below:

- Design
- Construction
- Operation, monitoring & maintenance
- Upgrading or demolition & renewal

Design

Contrary to most civil engineering projects, for a flood defence, the design is rarely the start of its lifetime. This is especially the case for dikes. Often, there already is a dike that does not satisfy safety standards anymore. A design is then made for the reinforcement of this dike. For special structures and water retaining hydraulic structures, the design stage often is the beginning of the lifetime. In the design stage, the current and expected boundary conditions are determined, which must be resisted by the flood defence. Boundary conditions that are expected in the future must be determined according to the service lifetime. This is the duration for which the flood defence must fulfill its function. Costs and functioning of the flood defence are largely dependent on the design. Therefore the design stage is very important.

Construction

After the design is made, the flood defence can be constructed. This phase is accountable for the largest part of the total life cycle costs. During the construction phase, the water retaining function of the flood defence must be maintained. To accomplish this, temporary flood defences can be constructed. In some cases it can also be important to prevent hindrance to shipping. The constructability is implemented in the design.

Operation, monitoring & maintenance

During this phase of the lifetime, the flood defence is in use, thus fulfilling its function. The functionality must continuously be guaranteed. This is done by monitoring the flood defences. Monitoring consists of frequent inspections, but also a total assessment that is performed once in twelve years. Maintenance works are regularly executed, but also when inspections give reason for it. Assessment is based on demands that are imposed on the structure and/or dimensions of the flood defence. This can lead to either approval or disapproval. If a flood defence is disapproved, it has to be upgraded or renewed. Operation is more applicable to water retaining hydraulic structures. Since these structures include movable parts, they have to be operated.

Upgrading or demolition & renewal

Upgrading or demolition and renewal of a flood defence is necessary when it does not satisfy flood safety standards anymore. Degradation of the defence is the most obvious reason for this. However, another reason can be changed hydraulic boundary conditions. Although there is no consensus under scientists on the topic of climate change, it appears to cause sea level rise and increased river discharges. Therefore, a large number of flood defences that were designed for a certain service lifetime, now appear to be insufficient safe. The choice to upgrade or demolish and renew depends on the type of a flood defence. Dikes are most often upgraded. Structures can also be upgraded, but at some point they have to be replaced. For this reason, it is economical more efficient to design special structures and water retaining hydraulic structures for a longer lifetime.

Although the design is just one phase of the lifecycle, the choices that are made in this phase have a large effect on the other lifecycle phases as well.

3.2.2 Governance

Now that the different types and categories of flood defences including relevant lifecycle phases and legislations are described, the distribution of tasks and responsibilities can be elaborated. Flood defences are all managed by governmental organizations. From large to small, these are the Ministry, Provinces and water boards. Municipalities also have a role, but have no direct responsibilities regarding the management of flood defences.

Ministry

The Ministry of Infrastructure and the Environment is responsible for subjects of national interest, like the safety against flooding. Regarding flood safety, the ministry has the role of upper supervisor. Partly based on European legislations, the Ministry is responsible for national legislations and sets up the framework for provinces to develop their own regional legislations and policies. Furthermore, they are responsible for the direct supervision over the managing bodies of primary flood defences. The Dutch department for Public Works and Water Management (Rijkswaterstaat), the executive organ of the Ministry, manages national waters like major rivers, including both primary and regional flood defences.

Provinces

Provinces are charged with the task of structuring the development of regional areas. They also are responsible for the development of standards and policies regarding regional flood safety and groundwater management. Except for flood defences along national waters, the provinces are responsible for the management of flood defences along regional waters and thereby keep supervision over the water boards and municipalities.

Water boards

The water boards are charged with the care for regional water safety and management. As executive organizations they are responsible for the construction, maintenance and inspection of flood defences. They also manage a large part of the primary flood defences. For the primary flood defences, they are under the supervision of the ministry and for regional flood defences under the supervision of provinces. Water boards are independent governmental bodies, levying their own taxes and administering justice.

Municipalities

Municipalities have no responsibilities regarding the management of flood defences. They are however, in charge over the spatial planning of cities and villages. Flood defences must be given a place in these plans. For this, co-operation with water boards is essential.

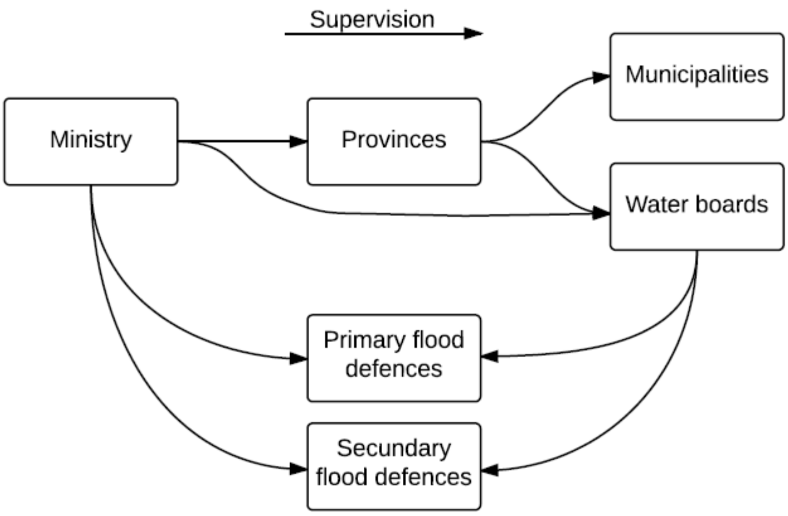


Figure 7: Supervision chart regarding flood defences

3.2.3 Legislation

For flood defences, legislations in several themes are applicable. Examples of themes are spatial development, structural safety, flood safety, but also water quality. Planning and evaluation of spatial development plans is a responsibility of the government. The quality of water is also a responsibility of the government. However, initiators of projects must comply to the condition that are set by these governmental organizations. Furthermore, initiators of realization of a structure in a flood defences must show that the structure complies to laws regarding structural- and flood safety. Laws that are relevant for realization of a new flood defence are described below.

Water Act

Since 2009, the Water Act has become applicable as a replacement for several individual laws on the subjects of flood safety and water quality. In this act, flood safety standards are given for primary flood defences. These are specified as a maximum probability of exceedance of a certain water level for which the flood defence must be designed. Also, a chart is attached which shows all primary flood defences in the Netherlands. Tasks and responsibilities of different governmental bodies regarding primary and regional flood defences are prescribed in this Act. For regional flood defences no safety standards are given. There is an obligation for provinces to develop these standards, based on the standards for primary flood defences. Regulations regarding regional flood defences are described in provincial water regulations. (Water Act, 2009)

In the Delta Program, plans are described to use flood safety standards that are specified as the maximum inundation probability due to failure of the flood defence. From 2017, this will replace the standard that is based on the maximum probability of exceedance of a water level. The new standards will be specified based on individual risks, group risks and risk of economic damage and will include more failure mechanisms. Also, instead of dike rings, dike sections are regarded. This way, more differentiation between flood defences is possible and more cost effective solutions are expected. The Water Act has already provided the opportunity to use the new standards (Deltaprogram, 2014). The new standards are only applicable to primary flood defences. The development of standards, regarding regional flood defences, will remain the responsibility of provinces.

The Water Act also gives regulations for the Water permit. This permit is obligatory for activities in or near surface- and groundwater.

Housing Act

Buildings may not endanger the safety of inhabitants, visitors and the environment. This Act forms the basis for regulations regarding construction, functionality and demolition of buildings. These regulations are partly recorded in the Dutch building code 2012 (Bouwbesluit) and have the purpose to secure the safety, health, usability, energy efficiency and environment. Every building must comply to these regulations. If a structure in a flood defence is accessible to people, the Dutch building code 2012 is applicable. The municipality supervises over the compliance to the rules. The Housing Act also contains juridical and administrative rules for different stakeholders. (Housing Act, 1991)

WABO

The WABO is an Act which gives regulations regarding the environmental permit (omgevingsvergunning). In 2008, this permit replaced a large group of individual permits and exemptions. It regulates activities in spatial, construction and environmental themes. Most permanent and temporary changes to the environment require an environmental permit. (Wabo, 2008)

3.3 Design instruments

Since the total length of flood defences in the Netherlands is very large, it is beneficial to have instruments, which contain rules and advices for designers and managers. This way, the wheel does not have to be reinvented for every flood defence project. Furthermore, a more uniform situation will be established. Several design instruments are therefore provided by the government. These are based on technical reports and applicable laws. Design instruments can be advisory, but can also have a legal status. Technical guidelines are advisory, while it is a legal obligation to design structures conform building codes. The position of design instruments with respect to technical research and legislations is shown in
Figure *8*.

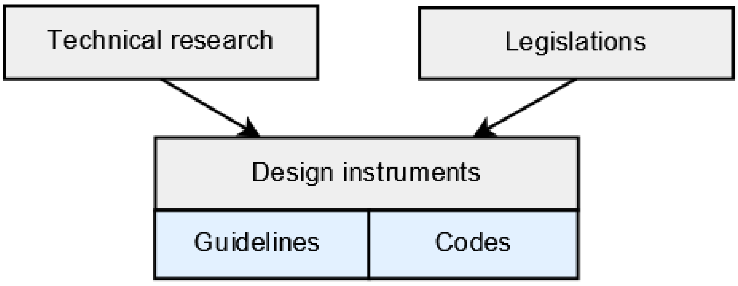


Figure 8: Position of design instruments

3.3.1 Codes

For all objects, goods and structures, which are accessible/available to the public, codes are applicable. These are prescribed by law, meaning that they have a legal status. Besides the prescription of safety standards, most codes also give rules for technical calculations. Codes are developed by leading experts in several fields of practice. For structures (in flood defences), the Eurocodes are applicable.

Eurocodes

Eurocodes are a set of technical reports, which prescribe safety standards and calculation rules regarding the design of (hydraulic) structures. The Eurocodes are implemented in the Dutch building code 2012 (Bouwbesluit). Therefore, these codes have a legal status. There are 10 separate Eurocodes, with each consisting of several parts (JRC EC, 2015):

- EN 1990: Basis of structural design
- EN 1991: (Eurocode 1) Actions on structures
- EN 1992: (Eurocode 2) Design of concrete structures
- EN 1993: (Eurocode 3) Design of steel structures
- EN 1994: (Eurocode 4) Design of composite steel and concrete structures
- EN 1995: (Eurocode 5) Design of timber structures
- EN 1996: (Eurocode 6) Design of masonry structures
- EN 1997: (Eurocode 7) Geotechnical design
- EN 1998: (Eurocode 8) Design of structures for earthquake resistance
- EN 1999: (Eurocode 9) Design of aluminum structures

For structures in flood defences, safety standards are prescribed by both the Eurocodes and the Water Act. In that case, the most stringent standard applies.

3.3.2 Technical guidelines

Technical guidelines are meant to advice the designer and manager with the design and assessment of flood defences. For each type of flood defence, described in section 3.1, separate guidelines are developed. In the descriptions below, a distinction is made between guidelines for design and guidelines for assessment. Also primary and regional flood defences are treated separately.

Primary flood defences
A summation of the most important technical guidelines, which are currently used to design primary flood defences, is given below:

- Guideline sandy coasts (LD2003);
- Guideline rivers (LR2003);
- Guideline for water retaining hydraulic structures (LK2003);

These guidelines are provided by the government. Sometimes references are made to technical reports. If a flood defence is designed with rules from a suitable guideline, implicitly it has been shown that the design complies to the applicable flood safety standard.

Rules for the assessment of flood defences are described in the WTI2006 (Legal Assessment Instruments 2006). This consists of the VTV2006 (Instructions for Safety Assessment 2006) and HR2006 (Hydraulic Boundary conditions 2006). The VTV2006 describes a three layer approach for the assessment of flood defences: simple-, detailed- and advanced assessment. An advanced assessment is used for flood defences that cannot be assessed with basic rules. The WTI has, in contradiction to other technical guidelines, a legal status. In 2011, a new version of the WTI was developed. This version was not officially approved and therefore has no legal status, but an advisory role.

The WTI2017 will replace the WTI2006. A conversion is made from safety standards based on exceedance probabilities to standards based on flood risks. In order to accomplish a smooth transition, tools like probabilistic calculation models, are currently being developed (Rijksoverheid, 2015b). In 2014, a report (OI2014) was published by the Ministry, in which guidelines are given to design flood defences based on current technical guidelines, but with some alterations to satisfy the new flood safety standards. Based on the safety of people and economic optimizations, the WTI2017 will prescribe maximum admissible failure probabilities for the design and assessment of primary flood defences. These failure probabilities are not yet made final, but the OI2014 gives signal values of failure probabilities that can be used to estimate maximum admissible failure probabilities. As a rule of thumb, the signal value of the failure probability should be multiplied by three. If the water level at a frequency of exceedance equal to the obtained value is used as a design water level, the OI2014 states this will lead to a design of the flood defence that will also satisfy the new flood safety standards. In short, the OI2014 gives a conversion rule to obtain a design water level from a failure probability as described by the new flood safety standards.

Regional flood defences
Safety standards for regional flood defences are less strict than for primary flood defences. Contrary to primary flood defences, the Water Act does not obligate the government to provide technical guidelines for regional flood defences. However, the UvW (Union of water boards) and IPO (Inter provincial consult) found there was a need for technical guidelines. Therefore a series of guidelines is developed. Since the flood safety standards for regional flood defences are not expected to change in the near future, current technical guidelines will remain applicable. The most important are summarized below:

Design
- Guideline for the design and improvement of flood defences along discharge channels (LBK2009);
- Guideline for the design and improvement of flood defences along regional rivers (LRR2009);
- Guideline water retaining hydraulic structures in regional flood defences (LRK2011);

Assessment
- Guideline safety assessment of regional flood defences (LTR2015);

In Dutch these are called (in the same order):
- Handreiking ontwerpen & verbeteren van boezemkaden (LBK2009);
- Handreiking ontwerpen & verbeteren van waterkeringen langs regionale rivieren (LRR2009);
- Leidraad waterkerende kunstwerken in regionale waterkeringen (LRK2011);
- Leidraad toetsen op veiligheid regionale waterkeringen (LTR2015);

3.4 Innovations in water retaining structures

An important feature of innovative technologies is that designers and managers must work outside their traditional framework. Therefore, application of innovative technologies always induce an increase in uncertainties and thereby risks. In conventional flood defence projects, technical guidelines are used to obtain the engineering design. Based on experience with existing techniques, these guidelines are developed in such a way that by applying the given design rules, the reliability requirements are automatically satisfied. It is the responsibility of the initiator/developer of the project to proof that the reliability of the flood defence is in accordance with the applicable flood safety standard. The flood defence manager is responsible for the assessment. Since innovations are by definition new, no experience is available. Hence, the design rules in guidelines cannot be used and both the designer as the manager are hampered (Knoeff, et al., 2013). This problem is underlined by the fact that the Dutch department for Public Works and Water Management (Rijkswaterstaat) has initiated the corporate innovation program. In this program, governmental bodies, companies and knowledge institutes are brought together to bring in innovative ideas and discuss possibilities for realization. Also the process from idea to practice is investigated, in order to improve it. (Rijkswaterstaat, 2013).

3.4.1 Provisions in design instruments

In technical guidelines for flood defences, a distinction is made between the ‘simple’, ‘detailed’ and ‘advanced’ method to design water retaining structures. For the simple method, partial factors are prescribed to perform semi-probabilistic calculations. Using this method is only possible for conventional structures. The result is a rather conservative design. For some projects, it is desirable to follow a less conservative approach and obtain a more optimized design. Cost effectiveness is the main reason for this. Then, the detailed method should be used. In this method, partial factors are recalculated. For some unconventional structures, unknown or other failure mechanisms could occur. In that case, it is impossible to use the simple or detailed method. The advanced method prescribes the use of probabilistic calculations to assess the reliability in the design of these structures. Innovative technology is by definition unconventional. Therefore, the advanced method should be followed in the design. Hence a reliability evaluation based on probabilistic design calculations should be performed.

4. Introduction to structural glass

The brittle failure behaviour of glass makes that it is not the favourite material of most structural engineers. However, It has the unique property of transparency, which is very attractive from an aesthetic point of view. The use of glass in buildings is increasing and there is an increasing demand from architects to also use glass for load carrying structural elements. The glass elements at the water side of the glass dike have to prevent water from the channel to flow into the polder. So the elements should resist the hydrostatic pressure from the water, but should also be strong enough to resist impacts from for example a stone that is thrown by a vandal. The load bearing function of the glass makes that the panels should be regarded as structural elements. An introduction to the structural application of glass is given in this chapter.

4.1 Mechanical behavior

Glass is a brittle material. Up to the point of maximum stresses, glass behaves as an elastic material. In absence of any plastic deformations, failure occurs very abrupt and without warning. This in contrast to steel structures, which deform plastically before collapsing. The brittle behavior is due to the silicon rich composition of the material. This property is also responsible for its hardness and high mechanical strength. Another disadvantage of the disability of glass to deform plastically is that small flaws in a glass element can lead to very large stresses locally (Doulkari, 2013). Since a crack in glass will usually develop very rapidly and will not stop before it has reached the edge of the element, these flaws can lead to total failure of the glass. Therefore, these imperfections should be taken into account when determining the 'practical' glass strength. This is the part of the theoretical strength that can be utilized when taking into account these flaws. Examples of imperfections are little scratches or nickel-sulphide inclusions. Another consequence of this, is that there is a large scatter in the loads at which individual pieces of glass will break. In summary, the following main disadvantages to the use of glass are important to consider for structural applications (Bos, 2007):

- There is a large scatter in the stress at which individual pieces of glass will break. The coefficient of variation is typically somewhere in between 10 and 30 %.
- Glass breaks very abrupt, without any warning.
- When the glass breaks, it breaks completely. A crack will continue to develop, even if the initial stress which induced the crack has already disappeared.

Another interesting characteristic of glass is that the strength is correlated to the duration of loading. Long lasting loads, contribute more to the accumulation of cracks. This leads to an increase of the probability of critical cracks to develop. Which in turn results in a lower load bearing capacity for long term loads. Table 3 gives an overview of the main mechanical properties of non-strengthened float glass compared to other materials. (Doulkari, 2013)

Material	Type	Young`s modulus	Yield point	Tensile strength	Compressive strength	Failure behavior	Coefficient of expansion
Glass	Silico-soda-lime	70.000	-	45	700	Brittle	$0.9 \cdot 10^{-5}$
Concrete	C25/30	26.700	-	2.6	25	Brittle	$1.0 \cdot 10^{-5}$
Steel	S335	210.000	360	510	-	Ductile	$1.2 \cdot 10^{-5}$
Aluminum	EN AW-6060	70.000	160	215	-	Ductile	$23.5 \cdot 10^{-6}$
Unit		N/mm^2				-	K^{-1}

Table 3: Comparison of mechanical material properties (Doulkari, 2013)

Silico-soda-lime glass is the most common type of glass used in practice. The tensile strength given in Table 3 is the characteristic value used for design calculations. It should be noted that this value concerns glass that has not been strengthened by one of the methods elaborated in next section. The theoretical strength is much larger, up to 21.000 N/mm^2 is possible. Since cracks and surface flaws do not grow under compression, the compression strength that may be used in design is also much larger than the practical tensile strength. (Fröling, 2013)

4.2 Types of glass

As mentioned before, Silica-soda-lime glass is the most common type of applied glass. This type of glass is produced by four basic types of operations, namely: batching, melting, fining and forming. Batching is the process in which materials are selected based on characteristics like chemistry, particle size and uniformity. After the batching process, the materials are melted with a glass furnace. The fining process is intended to create a molted glass that has a uniform composition and that is bubble free. The forming process is different for different applications of the glass. Flat glass, which is widely used in buildings, is formed by the float process. The flat glass that results from the float process is called annealed glass. Besides these standard operations, also some post processes can be applied, which are meant to improve the mechanical properties of the glass. Different types of glass that result from post-treatments of flat glass produced by the float process are described below. (Le Bourhis, 2008)

Annealed glass
Flat glass produced by the standard float process without any post treatment is called annealed glass. Typical for the standard float process is that the glass is slowly cooled to avoid internal stresses. When this type of glass breaks, it splits into large fragments. These fragments can cause injuries to people. However, when more glass plates are laminated, these fragments still have some residual strength, thereby contributing to the bearing capacity of the element. (Haldimann, Luible, & Overend, 2008)

Heat strengthened glass
Since the strength of glass is highly correlated to flaws on the surface and cracks grow under tension, it makes sense to apply some process that results in compressive stresses at the glass surface. Heat strengthening is such a process. Float glass is heated and subsequently cooled rapidly by cold air jets. The surface cools faster than the center, which results in tension stress in the center of the plate and compressive stress at the surface. Cracks at the surface of the glass are only allowed to grow if the plate is loaded such that tension stresses induced by the load exceed the initially present compressive stress. When heat strengthened glass breaks, it will split into smaller fragments than annealed glass. Therefore, it has a smaller residual strength than annealed glass when used in laminated glass elements. (Haldimann, Luible, & Overend, 2008)

Fully tempered glass
Fully tempered glass is produced with a similar post-treatment as for heat strengthened glass. The difference is that the cooling is performed even more rapidly. As a result, the residual stresses in the center and the surface of the glass are larger and hence, it is stronger. However, if fully tempered glass breaks, it will shatter completely. This property can prevent injuries to people that are nearby the breaking glass. For this reason it is sometimes also called safety glass. It can also be a disadvantage as it has no post-breakage strength when used in laminated glass. (Haldimann, Luible, & Overend, 2008)

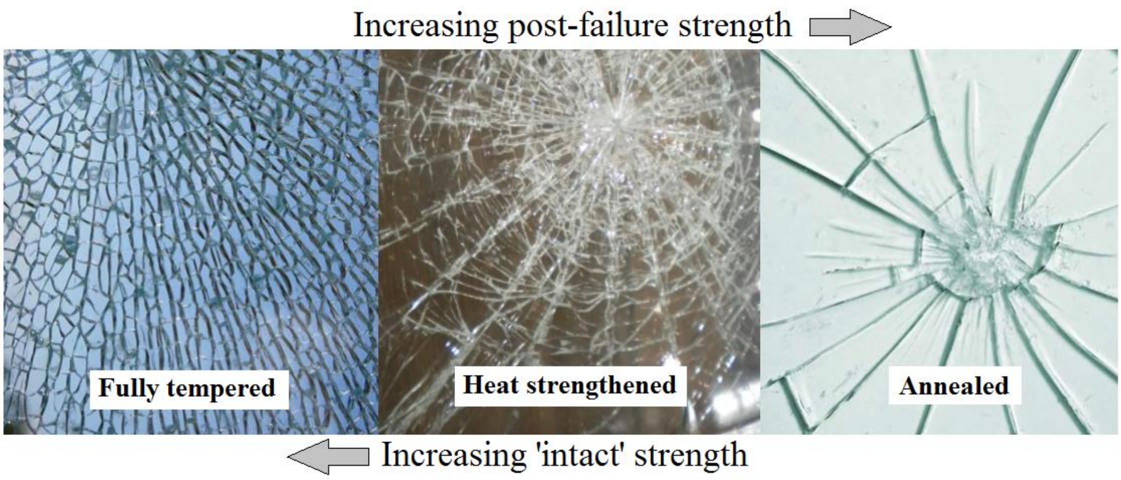


Figure 9: Structural performance of different types of flat glass

Chemically tempered

Yet another method to strengthen the glass is chemical tempering, in which chemicals exchange ions on the surface of the glass in order to create compression. Since this is much more expensive than for example heat strengthening, it is not widely applied. (Ford, 2001)

Laminated glass

The brittle failure behavior of glass makes that it has very little redundancy, thereby making it very unattractive to use for structural applications. However, a significant increase in redundancy can be achieved if a number of glass plates are bonded by plastic interlayers. For this, different plate thicknesses with different heat treatments can be used. If one glass ply breaks, the other plies can take over the load. Furthermore, the pieces of the broken glass will stick to the interlayer, such that personal injuries are less likely. As explained above, the residual strength of glass after breaking depends on the type of post-treatment it has had. However, this is only relevant when the glass plates are laminated. If not laminated, broken glass will simply fall down. Figure 9 gives a qualitative description of the structural behavior of the different glass types. (Haldimann, Luible, & Overend, 2008)

When using laminated glass, the stress is not distributed linear over the cross-section of the panel as there are discontinuities at the interlayers. This is due to the large difference in stiffness of the glass and the interlayer material. A common applied interlayer material is Polyvinyl Butyral (PVB). This material has a Young's modulus of 6 N/mm^2 and a Poisson ratio of 0.43. For the determination of the strength of a laminated glass panel, this effect can be taken into account by means of the equivalent thickness. Based on the composition of the laminated glass panel and the material properties of glass and the interlayer, the equivalent thickness is computed and can be compared to the thickness of single homogenous glass plate with equal strength. The equivalent thickness of laminated glass depends on the bonding between the plates. For full bonding (100%) the equivalent thickness is equal to the real thickness, while for no bonding (0%) the panel is just a combination of non-co-operating plates. This is illustrated in Figure 10. The nominal thickness of a PVB foil is 0.38 mm, but it is common to use two (0.76 mm) or even four (1.52 mm) layers of foil for just one interlayer. Another type of interlayer foil is ionoplast (SentryGlas) from DuPoint. This is a much stiffer material, which also provides more bonding. (Fröling, 2013)

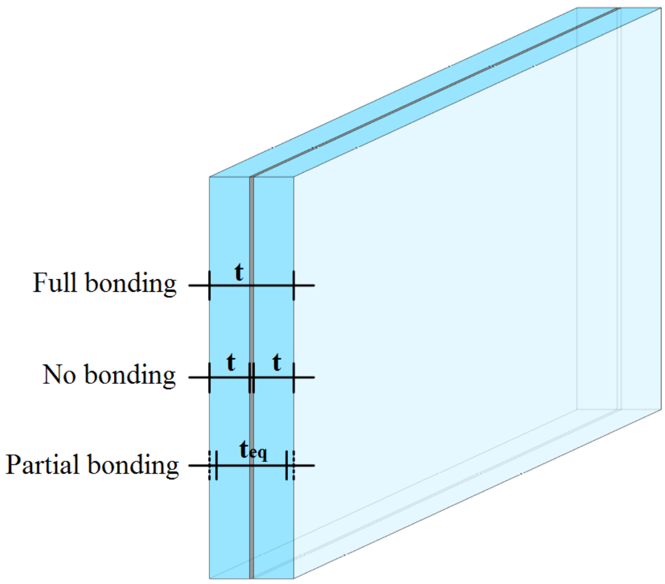


Figure 10: Equivalent thickness of a laminated glass panel

4.3 Glass strength statistics

As stated before, the strength of glass is highly correlated to damages it has accumulated, to date. This is however not the only factor that is responsible for the large scatter in strength. The quality of the production process is also of large influence. In section 4.3.1 the results of tests that were performed (Veer, Louter, & Bos, 2009) on respectively annealed- , heat strengthened- and fully tempered float glass are discussed. In section 4.3.2, possibilities to improve the structural properties of float glass are discussed.

4.3.1 Strength tests (Veer, Louter, & Bos, 2009)

Four point bending strength tests were performed on glass specimens of 1000 millimeters long, 100 millimeters width and 10 millimeters thick. Of these specimens, one third have had no post-treatment (annealed float glass), one third was heat-strengthened and one third was fully tempered. Of each group, half of the specimens were tested lying and the other half were tested in standing position. All specimens were professionally cut with automated cutting machines and finished by automated grinding and polishing. The results are shown below:

Tempering	Orientation	Average failure stress	Standard deviation	Average pre-stress	Standard deviation of pre-stress
-	-	MPa	% of average	MPa	% of average
Non (Annealed)	Lying	42.0	21.8	-4.5	37
Non (Annealed)	Standing	27.5	20.1	-4.5	37
Heat-strengthened	Lying	104.0	27.7	-64.3	3.8
Heat-strengthened	Standing	71.3	15.8	-64.3	3.8
Fully tempered	Lying	157.4	18.9	-100.6	12.9
Fully tempered	Standing	98.0	13.7	-100.6	12.9

Table 4: Results of four point bending tests on annealed, heat-strengthened and fully tempered float glass.

The first thing that can be noticed, is that specimens that were tested lying fail at considerably higher stresses. A good explanation for this could be that the edges are significant weaker than the surface of the float glass. It was attempted to fit Weibull descriptors to the results of each test group. A reasonable fit was only found for fully tempered glass. The results of annealed and heat-strengthened glass strength tests show a clear bi-linear Weibull behaviour. According to the authors, this indicates that there is more than one type of failure. Weibull plots for the heat-strengthened specimens are shown in Figure 11.

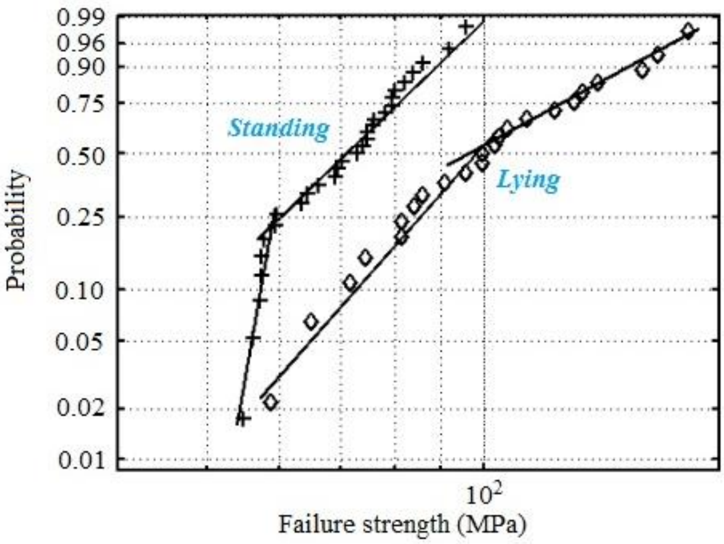


Figure 11: Weibull plots for heat- strengthened glass tests

For engineers, it would be most convenient if the strength of a certain type of glass could be computed by one formula. One option could be to only take into account the lower values of the failure stresses and fit these to a single Weibull function. Since higher values of the failure stresses are disregarded, this is a safe approach. However, the obtained Weibull descriptors would only be valid for glass specimens with equal size, coming from the same production process and loaded in the same way. Amongst others, the failure stress of glass depends on the following factors:

- Type of test (e.g. four point bending test or ring on ring test)
- Size of the test specimen
- Thickness of the test specimen
- Aspect ratio of the test specimen
- Orientation of the test specimen (standing or lying)
- Tested side glass plates (for lying specimens)
- Type of pre-stressing treatment
- Quality of the pre-stressing treatment
- Type of edge treatment

During the experiment it was found that in some cases, the glass failed at an applied stress that is lower than the value of the pre-stress. From this, it can be concluded that the commonly assumed principle that the failure stress is equal to the summation of intrinsic strength and pre-stress is not valid. Furthermore, since the variation in failure stress is much larger than the variation in pre-stress, it can be concluded that failure of glass is effected more by defects and material behaviour rather than the level of pre-stress.

4.3.2 Influence of quality control in the production process

As mentioned earlier, the strength of glass is strongly influenced by the quality of the production process. The pre-stressing level for example shows large variation over the surface of a glass panel. This is caused by the inhomogeneous flow of air during the cooling process. As a result, the pre-stressing is typically larger at the edges. Furthermore, the pre-stress level varies with the direction in which its measured. Unfortunately, no production process exists for which this problem is solved, to date. (Veer & Rodicheff, not yet published)

There are options to improve the strength of glass that can be chosen by the engineer. To start with, the quality of edges can be improved by finishing with automated grinding and polishing machines. From tests, it appeared that this gives larger failure strengths than conventionally produced glass. Alternatively, the glass could be cut by a water jet. This will not increase the failure strength, but gives less variation than is found in tests with conventionally produced glass. As a result, the glass is more reliable. (Veer & Rodichev, 2011)

Another measure during the production process to increase the reliability of a glass panel is to apply a heat soak test. This test will eliminate glass plies in which one or more nickel-sulphide inclusions are present. This is mainly relevant for fully tempered glass. (Kasper, 1999)

Quality control is not only important during the production process. The strength of a glass panel can be strongly reduced if it gets damaged, regardless at what point during the lifetime. This should be prevented as much as possible. Another threat is corrosion. This is elaborated further in next section, which describes some points of attention with respect to the durability of (laminated) glass.

4.4 Durability

Corrosion
Under normal conditions, glass is a very durable material. This statement is supported by the fact that buildings aged over a couple of centuries exist that still have the original placed glass windows. However, when glass is used in humid environments, the durability can be compromised by corrosion. The water breaks the interatomic bonds in the glass, which causes sodium, calcium and magnesium ions to be released. When the volume of water is small, the alkali concentration increases, which in turn stimulates the attack on the interatomic bonding of the glass. This is only the case if the glass is exposed to prolonged contact with the same small volume of water. This effect does not occur if the glass is exposed to large volumes of water like a basin. If the water is free to evaporate, there is also no threat. So corrosion by water is mainly a problem for small volumes of trapped water. For example during storage in a humid room, where glass plates are placed against each other. (Button , et al., 1993)

As stated before, the strength of glass depends on small imperfections on the glass surface as tensile peak stresses are developed locally. These imperfections are the result of many causes, but most of them are caused during production and placement. The amount of small damages increases during the lifetime of the glass. For example due to grit that is transported by wind, but also due to human actions like cleaning. So over time the strength of the flat glass will decrease. (Button , et al., 1993)

Durability under loading
Glass is a perfectly linear elastic material. In absence of any ductility, it cannot permanently deform, so it will go back to its original shape, even after a large number of load cycles, hence glass is not susceptible to dynamic fatigue. Furthermore, glass does not exhibit creep. It is however sensitive to static fatigue, sometimes also referred to as stress corrosion. When the glass is loaded in bending, the tensile stress at the surface causes small cracks to open and grow. In humid environments, water can accumulate inside the cracks and stimulate the strained interatomic bonds to break. The longer the loading, the larger the stress corrosion. The result is that the strength of the glass permanently loaded in bending will decrease over its lifetime. Static fatigue can be prevented by applying post-treatments like heat strengthening or tempering, under the condition that the residual surface compression stress (pre-stress) is larger than the tensile stress induced by the load. (Button , et al., 1993)

Interlayers in laminated glass
As explained in section 4.2 the co-operation of glass plies in a laminated glass plate depends on the bonding that is provided by the interlayers, which in turn depends on the material properties. For both PVB and SentryGlas, the Young's modulus and shear modulus depend on temperature. The values are strongly reduced at higher temperatures. These foils also exhibit creep, causing the co-operation of glass plies to reduce for long lasting loads (Button , et al., 1993). This is illustrated in Figure 12. In calculations for the load carrying capacity of laminated glass, the effect of creep is often taken into account by reducing the values of the Youngs modulus and Shear modulus for increasing load duration. See Table 5 for the Young's modulus of ionoplast (SentryGlas) foil for varying temperature and load duration (Kuraray, 2015).

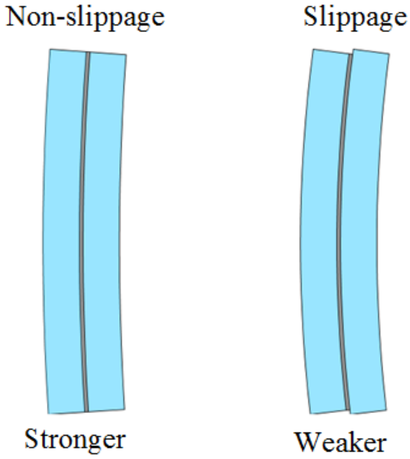


Figure 12: Laminated glass subjected to respectively short and long term loading

Young's Modulus (MPa)		1 s	3 s	1 min	1 h	1 day	1 mo	10 yrs
Temperature	10 ^{0}C	692	681	651	597	553	499	448
	20 ^{0}C	628	612	567	493	428	330	256
	30 ^{0}C	581	561	505	416	327	217	129
	40 ^{0}C	228	187	91,6	27,8	13,6	9,86	8,84
	50 ^{0}C	108	78,8	33,8	12,6	8,45	6,54	6
	60 ^{0}C	35,3	24,5	10,9	5,1	3,87	3,24	2,91
	70 ^{0}C	11,3	8,78	5,64	2,52	1,77	1,44	1,35
	80 ^{0}C	4,65	3,96	2,49	0,96	0,75	0,63	0,54

Table 5: Young's modulus of SentryGlas foil for varying temperature and load duration (Kuraray, 2015)

4.5 Susceptibility to (special) loads

Compression and shear

The compression and shear strength of glass have never been tested successfully. This is because the glass always fails due to tension that is induced during the tests. If structural glass fails, it is always because the tensile strength is exceeded. This is also the case for glass loaded in compression as the compression force is never perfectly symmetric and tension forces are always developed somewhere in the element. Only for hard impacts with substantial velocity, crushing of glass has been observed. (Button , et al., 1993)

Soft impact

Impact on laminated glass plates is still subject to research. FE models that can simulate impact events exist, but are still limited to relative simple impact problems (Don & Karunarathna, 2013). An example of a soft impact problem is a person that is falling on a glass floor or balustrade. Since the person can fall in many different ways, this is difficult to model. Often, an energy approach is applied. During impact, the kinetic energy of the object is partly transferred to the glass. An alternative to impact modelling is performing impact tests, in which an amount of kinetic energy is released on the glass and the damage is evaluated. In laminated glass, soft impact usually leads to damage in the outer glass ply (the opposite side of the attack side). This is because the tensile stress is maximum at this side. Off course, more layers could be damaged for impacts with more energy. A variety of tests have been developed in different countries. In most of these tests it is attempted to simulated the impact of a falling person. (Button , et al., 1993)

Hard impact

Additional to damage in outer glass plies, for hard impacts it is likely that damage is caused on the attack side of the laminated glass plate as well. Hard impacts induce forces that are concentrated on a very small surface area. The object that is hitting the glass is very stiff, hence very little kinetic energy can be stored in potential energy in the form of elastic deformations. Since the duration of impact is very short when both impactor and target are stiff, the impact force of a hard impact is larger than the impact force of a soft impact with the same amount of energy. Another effect of the short duration of the impact is that the laminated glass plate cannot fully deform during the impact due to inertia. The result is crushing of the glass ply at the point of impact at the attack side of the plate. If this glass ply is thermally strengthened or tempered, the compression zone could be perforated, leading to shattering of the ply (Don & Karunarathna, 2013).

Thermal stress

Typical for glass plates is its sensitivity to stresses induced by internal temperature differences in the plate. This could occur when the edges are kept cool in the frames while the middle of the plate is heated, for example by sunlight. The middle of the plate wants to expand, which leads to tensile stresses at the edges that don`t expand. In contrast to beams, this problem cannot be resolved by providing enough space in the frames for the glass to expand. A better understanding of this can be achieved by regarding Figure 13. Tensile stresses can also develop if the middle of the plate is cooling down more rapidly than the edges, which are kept warm by the frames. If glass fails due to thermal stress, a crack usually starts at one of the edges. This can be prevented by avoiding contact between the glass and metal frames, for example by providing rubber or some other insulating material in between. Another option is to use heat strengthened or fully tempered glass, which are not sensitive to thermal stress due to the pre-stress. (Button , et al., 1993)

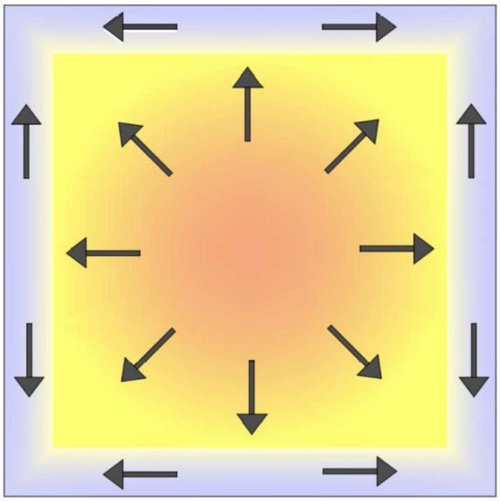


Figure 13: Thermal stress in window heated in the middle

Explosion

In some circumstances it is important to consider the rare event of an explosion. Apart from heat, a large pressure wave propagates away from the source of the explosion. The pressure wave is composed of a overpressure, followed by a vacuum (or large under pressure). The amount of pressure is proportional to the energy that is released in the explosion and inversely proportional to the distance from the source. Since the propagation velocity of pressure waves in air is very high, the load on the glass should be schematized as an impulse load. For these type of loads, dynamic effects in the glass play an important role. The resistance of a glass element against a certain explosion load can best be verified with experiments. Since explosion loads are usually very large, glass that has to resist this type of loading is often laminated. In combination with fixed supports, the glass remains in its frame at relative large pressures, even if all glass layers are broken. For this, a good design of the supports is crucial. It should be noted that explosions are often accompanied with hard impacts of objects that are launched by the explosion. (Button , et al., 1993)

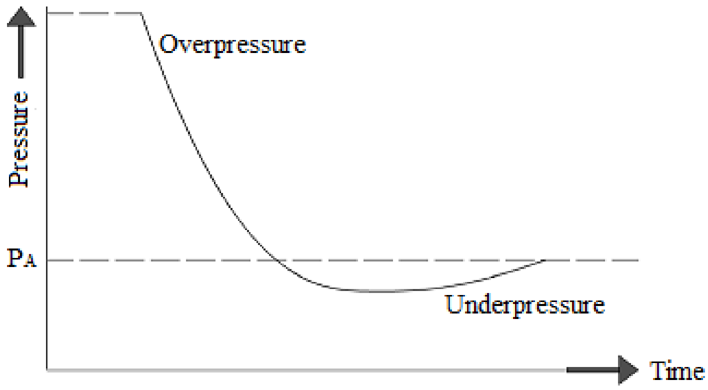


Figure 14: Pressure wave caused by an explosion

Fire

Glass is an incombustible material. However, flat glass will start to become soft at a temperature of approximately 600 degrees Celsius. Therefore, glass is not a good structural material with respect to fire. The fire resistance of glass is judged based on the following four criterial: Load bearing capacity, Integrity, Insulation and Radiation. The first criteria is very important to structural glass. It is required that the element continues to carry the loads during a predefined period of time. The duration is related to the time that is needed for people to evacuate a building. If structural glass is exposed to a fire for longer than one hour, it is likely that it will collapse. The other criteria are meant to stop or slow down the fire. (Button , et al., 1993)

4.6 Examples from practice

Some examples of high performance structural glass are shown in this section. Figure 15 shows the glass platform 'Step into the void'. As can be seen, not only the floor is made of glass, but also vertical parts of the box (to which loads are transferred) are made of glass. Figure 16 shows a wall, which is part of a flood defence situated along a river in the U.K. The glass elements only retain water when the water level in the river is very high.

Figure 15: Glass platform 'Step into the void' in the French Alps (Pilkington, 2016) *(Left)*
Figure 16: Glass high water protection in the United Kingdom (IBS , 2016) *(Right)*

Figure 17 shows a large submerged glass tunnel inside an aquarium in Georgia. Because of the round shape, stresses at the surface of the glass are limited. As a result, the glass is able to resist the very high hydrostatic pressure.

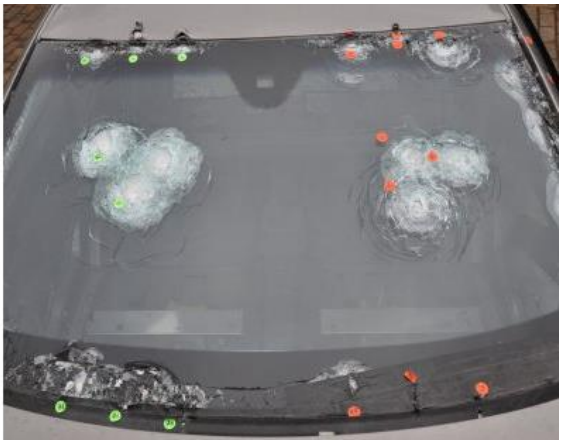


Figure 17: The Ocean Voyager in the Georgia Aquarium (Wikipedia, 2016) *(Left)*
Figure 18: Bullet proof glass (kogelvrij.nl, 2016) *(Right)*

In Figure 18, a bullet proof window shield of a car is shown. In this example, the load differs from the loads in the other examples. In this case, the glass is subjected to high velocity impacts, for which exceedance of the practical tensile stress is not the only criterion for failure. Crushing of the glass will occur at the location where the bullet hits the glass.

Part II

Case study: Glass dike

5. Design input

In large parts of the Netherlands, the landscape is defined by the many dikes that protect the land from flooding. Apart from the physical separation of land and water, there also is a visual separation. Experiencing land and water simultaneously is only possible by standing on top of the dike (Böhtlingk & Robinson, 2015). The main idea behind the glass dike is to eliminate this visual separation and originates from artist P.Izeboud. The idea was demonstrated in an impression video that shows a section of a primary flood defence entirely made of glass. In this video, it can be seen how a large sea vessel is sailing past it. The idea was picked up by architect E. Böhtlingk. He decided to aim for realization of the glass dike in a regional flood defence, for which safety regulations are much less strict than for primary flood defences.

5.1 Location

The flood defence that E.Böhtlingk would like to use to realize the glass dike is located in the polders of Midden-Delfland. The proposed location is on the last part of the Duifpolderkade, just before it takes a turn into the Schouwkade. It retains water from the Noordvliet. Here, the water level is high enough above the level of the land area in the polder to achieve the desired effect: Experiencing the water level differences in Dutch polders. The location is accessible by car from the Schouwkade. You can also get there by cycling or walking via the Duifpolderkade. On the other side of the water, a water pumping station is located. This pumps water into the Noordvliet.

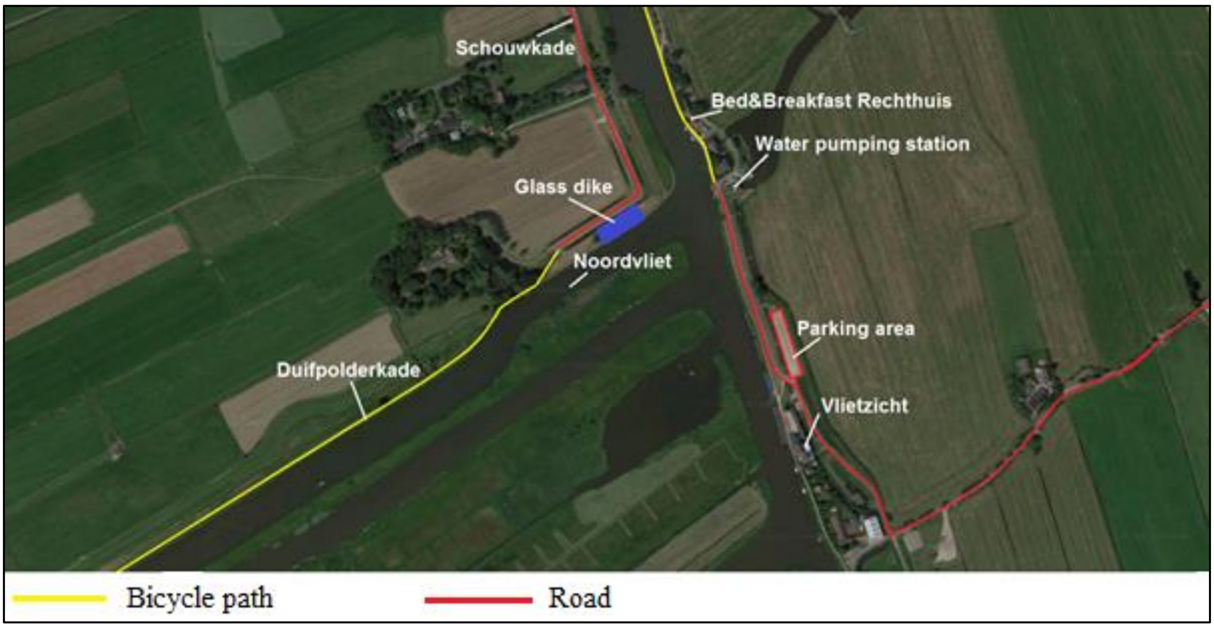


Figure 19:Desired location of the glass dike

5.2 Architectural design

The architectural design as proposed by E. Böhtlingk is the main input of the engineering design. In order to come to an optimal engineering design, all technical solutions that are implicitly defined in the architectural design are released. However, the aesthetic part of the architectural design must remain unchanged. Also the main outer dimensions of the glass dike are more or less fixed. The reason for this, is that the shape of the structure must follow the shape of the soil body. Sketches of the proposed shape, layout and dimensions of the glass dike, as proposed by E. Böhtlingk (2015) are shown in Figure 20, Figure 21 and Figure 22.

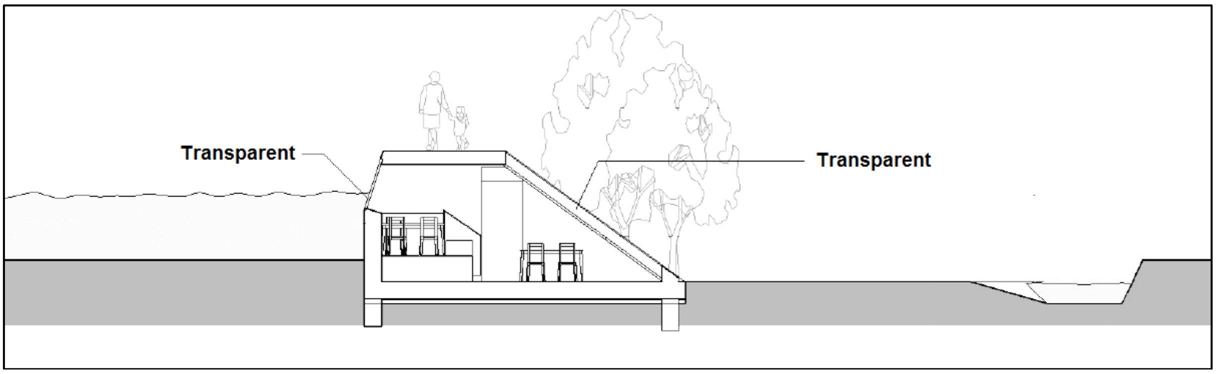


Figure 20: Cross-section of the architectural design of the glass dike (Böhtlingk, 2015)

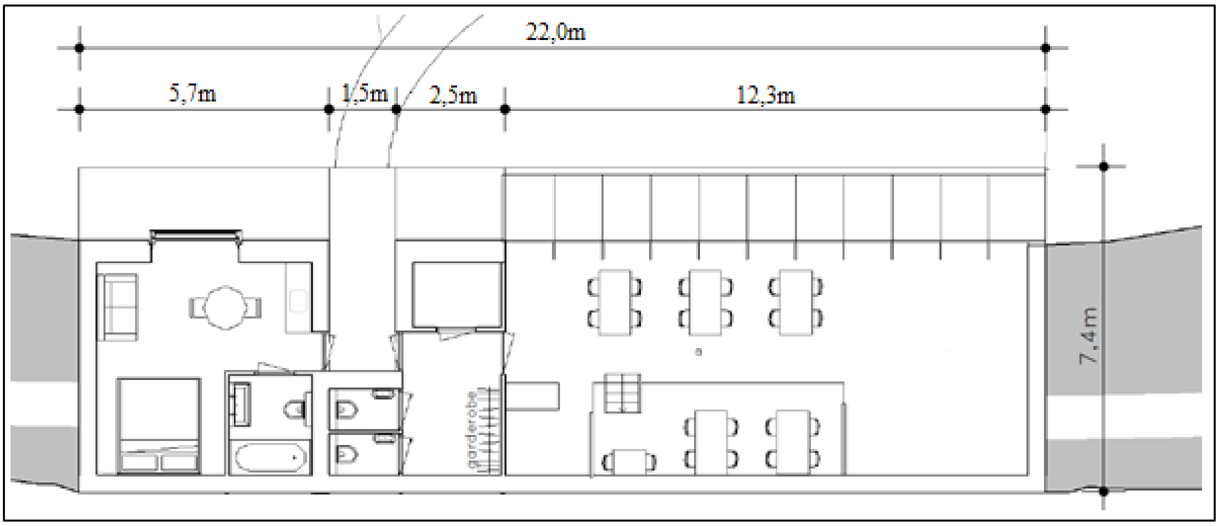


Figure 21: Layout and dimensions of the architectural design of the glass dike (Böhtlingk, 2015)


Figure 22: Artist impression of the glass dike

5.3 Flood defence system

In case the glass dike will be located as depicted in Figure 19, it will be part of the flood defence system that protects the Duifpolder. South-East of the polder, the flood defence retains water from the Noordvliet. It also retains water from the Bree- or Lichtvoetswatering (North-East), the Oostgaag (North-West) and the Middelwatering (South-West). The Duifpolder and its flood defences are shown in Figure 23. The total length of flood defences, by which the Duifpolder is protected amounts to 8,80 km, from which 1,70 km along the Middelwatering, 1,60 km along the Noordvliet, 2,50 km along the Oostgaag and 3,00 km along the Lichtvoetswatering (Centrum voor Onderzoek Waterkeringen).

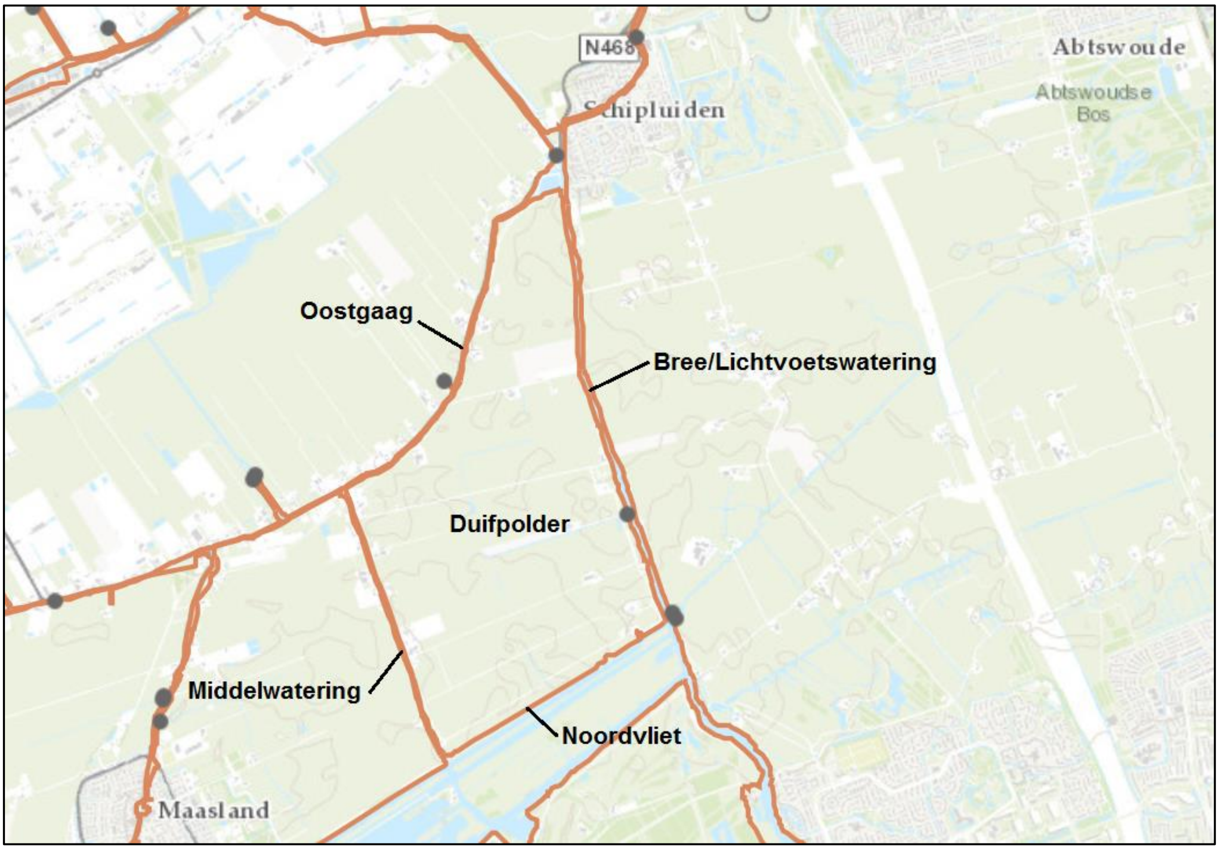


Figure 23: Flood defences surrounding the Duifpolder (Legger Regionale Waterkeringen, 2015)

Besides a number of non-water retaining objects (houses, bridges, etc), only one water retaining structure is situated in the flood defence. This is a water pumping station on the eastside of the polder. The province of Zuid-Holland sets the flood safety standards of regional flood defences in the polders of Midden-Delfland. The flood defence surrounding the Duifpolder has to be designed for a water level with an exceedance probability of 1/100 per year (Waterverordening Zuid-Holland, 2015).

The land area of the polder is approximately 350 ha, while the surface area of the waters directly connected to the adjacent waters is estimated at 680 ha. Consequently, a breach in the flood defence will lead to flooding of the polder with a depth of 0.65 to 1.30 m. The number of buildings in the polder is small. The land area mainly consists of meadows with cattle, but there is also a limited number of greenhouses (Centrum voor Onderzoek Waterkeringen). Therefore, a breach in the flood defence will lead to economic damage, but loss of human life is not likely to occur.

5.4 Stakeholders

In order to properly define the requirements for the design of the glass dike, the stakeholders involved in the project are described. Some of them have more interest in realization of the glass dike than others. Taking as much as possible interests into consideration in an early stage of the realization process may improve the design. Since the glass dike is planned in a remote area, the number of stakeholders is limited. An overview of the most important stakeholders and their interests is given in Table 6.

Stakeholder	Description	Interest(s)
E.Böhtlingk	Initiator of the glass dike project.	Realization of the glass dike. Personal interest.
Water board Delfland	Responcible for regional water management and flood safety.	Flood safety in Duifpolder. Non-restricted maintenance, inspection and assessment.
Minicipality Midden-Delfland	Management and decision making on local affairs.	Spatial planning, providing building licenses and enforcing building regulations.
Province Zuid-Holland	Management and decision making on provincial level.	Supervision over water boards regarding safety of flood defences.
Local inhabitants	People living nearby and in the Duifpolder.	Safety against flooding. Changed environment.
Local businesses	Horeca and farmers.	Safety against flooding. Reïnforcement of business environment.
Visitors	Schools, tourist, locals etc.	Education and recreation.
Civil contractors	Execution of civil engineering projects.	Building the glass dike structure. Financial interest, making profit.
Exploitant(s)	Providing the horeca and educational facilities in the glass dike.	Financial interest, making profit.

Table 6: Stakeholders and interests

5.5 Boundary conditions

In this section, the physical boundary conditions on the desired location of the glass dike are given. These serve to determine part of the design loads and identify failure mechanisms. Unnatural boundary conditions are not treated here. Unnatural boundary conditions may be laws, budget restrictions etcetera. Natural boundary conditions are divided into hydraulic-, geotechnical- and climate boundary conditions. In addition, an overview is given of the non-water retaining objects that are present on the location. The derivation and background information of the boundary conditions is given in Appendix A.

5.5.1 Hydraulic

Hydraulic boundary conditions in the Noordvliet are relative mild. The characteristic are described below:

Water levels Noordvliet

Water level with 1/100 exceedance probability:	N.A.P. -0,03 m
Average water level:	N.A.P. -0,43 m
Low water level:	N.A.P. -0,63 m

Water levels in the Duifpolder

Average water level:	N.A.P. -3,15 m
High water level:	N.A.P. -3,00 m
Average groundwater level:	N.A.P. -3,15 m
High groundwater level:	N.A.P. -3,00 m

Phreatic line

A linear phreatic line between the Noordvliet and the groundwater table is assumed. This is not the case in reality, but this assumption suffices for the first engineering design.

Waves

Wind induced waves:	< 0,10 m
Ship induced waves:	-

5.5.2 Geotechnical

Soil composition

In 2008, soil investigations were performed for the soil body at the desired location of the glass dike. The results of this investigations are shown in Appendix A.2. For the first engineering design, a simplification is made of the soil composition. The scheme of this simplification is shown in Figure 24.

The derivation of this scheme is shown in Appendix A.2. The soil is mainly composed of clay and peat. A sand layer is probably present below the depth of the cone penetration test, but the depth is unknown.

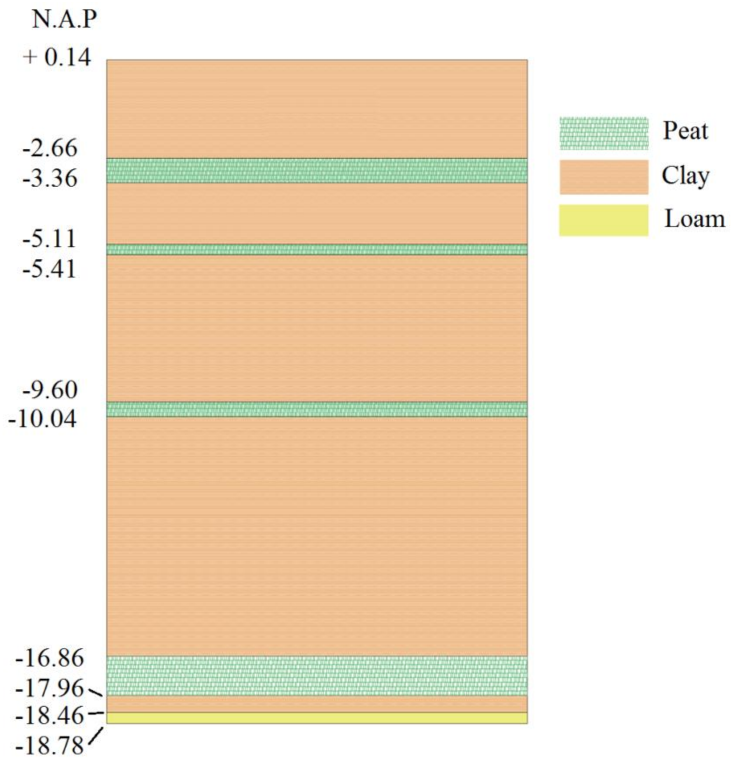


Figure 24: Simplified soil composition at/near the glass dike

Soil parameters

From the q_c values in the cone penetration tests, the parameters of the soil can be obtained. The q_c value is 0,2 MPa for clay layers, but also for peat layers. Eurocode 7 gives a table with soil parameters based on the q_c values. These are the representative values and are shown in the Table 7.

Parameter	Symbol	Value for clay	Value for peat
Dry specific weight	γ_{dry}	13 kN/m^3	12 kN/m^3
Saturated specific weight	γ_{wet}	13 kN/m^3	12 kN/m^3
Effective angle of internal friction	φ’	15^o	15^o
Effective cohesion	c’	0 kPa	0 kPa
Undrained shear strength	$F_{undr} = c_u$	10 kPa	20 kPa

Table 7: Soil parameters according to Eurocode 7

An alternative method to determine soil parameters is to look at the ratio of sleeve friction over cone resistance. With this 'friction ratio’ the type of soil can be determined. This soil type in turn, can then be used as input in the above mentioned table with soil parameters given by the Eurocode. This is less conservative than the approach described above. Since parameters obtained from the soil investigations that are presented in Appendix A.2 are very poor, there is a good change that performing extra soil investigations at the exact location of the glass dike will reveal better soil properties.

Dike profile

The dimensions of the existing dike profile are shown in Figure 25. These dimensions are obtained from in-situ measurements and data received from water board Delfland. Since data about the profile of the dike under the water line is not available, the minimum slope as given in the ledger is assumed (Legger Regionale Waterkeringen, 2015). The ledger is also used for the water depth, but now the maximum value is assumed.

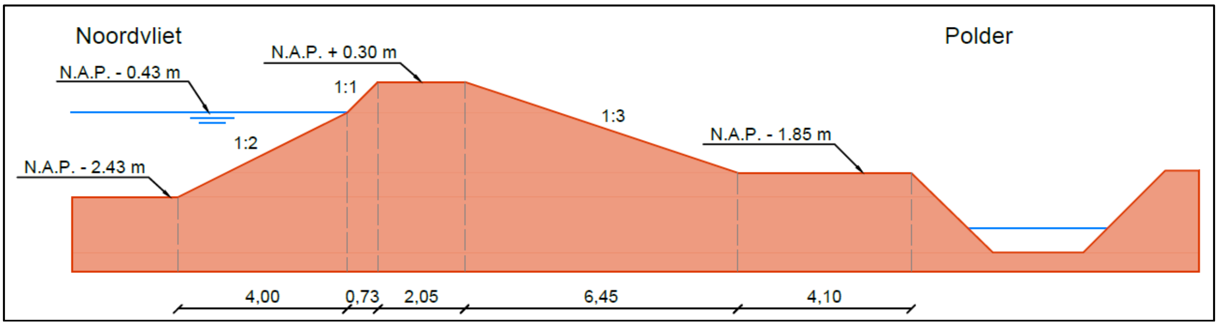


Figure 25: Dimensions of the existing dike profile

5.5.3 Climate

Other boundary conditions that can have an influence on the design of the glass dike are snow, precipitation, wind and ice. Snow loads are prescribed by the Eurocode. This is elaborated in section 6.3.4. Since the structure is very low, wind can be neglected. It is however of importance for determination of the wave height as readily described in Appendix A.1. Since there is no drifting ice in the Noordvliet, no ice dams can be developed. Hence the water level is not increased due to ice dams that block the water discharge. Expansion of an ice layer can induce forces on the embankments along channels. Since this force is very small, it is neglected. Lastly, earthquakes are not expected in Midden-Delfland.

5.5.4 Non-water retaining objects

There is a telephone cable situated in parts of the dikes along the Noordvliet. There are no pipelines situated at the location of the glass dike. Besides some trees, there are no other non-water retaining objects. The trees are shown in Appendix A.3.

5.6 Defining the required reliability

The glass dike is a multifunctional flood defence, meaning that it has to function as more than just a flood defence. In this case it also has to function as a building. The glass dike has to comply to both the Flood Safety Standard as to requirements for structural safety as prescribed by the Eurocodes. In this section, reliability requirements for the glass dike that follow from these regulations are described. Since these requirements partially overlap each other, a comparison between the Flood Safety Standard and the Eurocodes is made in section 5.6.3.

5.6.1 Flood safety standard

As explained in section 3.2.3, there is a difference between safety standards for primary flood defences and regional flood defences. The glass dike is part of a regional flood defence, which must be able to safely withstand a water level with an exceedance probability of 1/100 per year. The LRK2011 (Guideline water retaining hydraulic structures in regional flood defences) prescribes two methods to determine the required reliability of a structure that is part of a regional flood defence. Usually, the dike section approach is applied, but in some cases it is chosen to apply the dike ring approach.

Dike section approach

A commonly accepted design approach to satisfy the Flood Safety Standard is the 'dike section approach' as described in LRK2011. In this guideline, maximum failure probabilities for different types of failure are given, which are related to the water level exceedance probability as prescribed by the Flood Safety Standard. Flood safety is achieved if the water retaining function of a flood defence is safely fulfilled, implying that the probability of excessive amounts of water flowing into the polder is low. Hydraulic structures in flood defences may lose their water retaining function if one of the following situations occur:

1. The stability of the structure is lost
2. The strength of one or more structural elements is lost
3. Insufficient height of the structure with respect to the water-level
4. Non-closure of moving elements

The glass dike will not contain any moving elements, so situation 4 cannot occur. For stability and strength of the structure, a yearly failure probability of 1/100 times the exceedance probability is considered sufficiently safe. Then, a maximum probability of 1/10.000 per year for structural failure is obtained for the glass dike (p_f = 0,0001). The corresponding reliability index (β) is found with the standard normal probability density function. It is found that $\beta = -\Phi^{-1}(0{,}0001) \approx 3{,}6$ for a reference period of one year.

Apart from the strength and stability, the glass dike also has to be sufficiently high in order to fulfil its water retaining function. LRK2011 prescribes that the probability of a volume of water flowing over the structure being larger than admissible, is smaller than the safety standard of the flood defence (exceedance probability). For the glass dike, this means that $P(V \geq V_{admissible}) \leq 1/100$ per year. The quantity of the admissible volume of water depends on the land use and storage capacity in the polder. In Figure 26 a scheme is shown, which depicts how the required reliability of the glass dike is defined if the dike section approach as prescribed in LRK2011 is applied.

Note that values in the scheme correspond to a one year reference period. Summation of all failure probabilities in the scheme will give a total failure probability being larger than the water level exceedance probability as prescribed by the Flood Safety Standard. So one could say that the flood safety standard is not met if this scheme is applied. However, the Flood Safety Standard states that the flood defence has to be designed to safely withstand a water level with a certain exceedance probability. This exceedance probability should not be confused with a failure probability. Then in fact, the maximum failure probabilities that follow from LRK2011 should be seen as individual requirements for different types of failure. This is supported by the fact that consequences for different types of failure are different as well. Structural failure can lead to substantial damage and even casualties, while overflowing water may only cause hindrance and little damage.

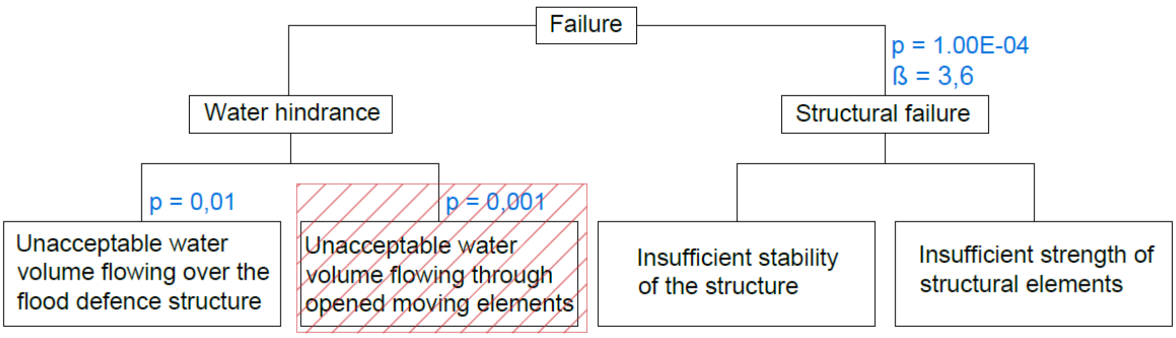


Figure 26: Reliability requirements of the glass dike for a one year reference period

Dike ring approach

Alternatively to the dike section approach, the required reliability of the glass dike may also be defined with the dike ring approach, but only on condition it does not lead to a lower required reliability. In this approach, the required reliability of a dike section or water retaining structure is deduced from the reliability of the dike ring. In the case of the glass dike, the dike ring is composed of all dike sections that surround the Duifpolder and protect it from flooding. As described in section 5.3, it is mainly composed of dikes, but also a water pumping station that is located at the East-side of the polder is present. Besides this, a number of non-water retaining objects (houses, bridges, etc) are present. LRK2011 prescribes that if a dike ring approach is followed, the maximum admissible probability of the top event, which is a breach somewhere in the dike ring, may not be larger than 10% of the water level exceedance probability as given by the Flood Safety Standard. The other 90% is reserved for overflow and overtopping. The guideline also gives rules for subdivision of failure probability spaces over different parts of the dike ring. Although values of the failure probability spaces may be defined by the flood defence manager, subdivision of the types of flood defences in the dike ring should correspond to Figure 27.

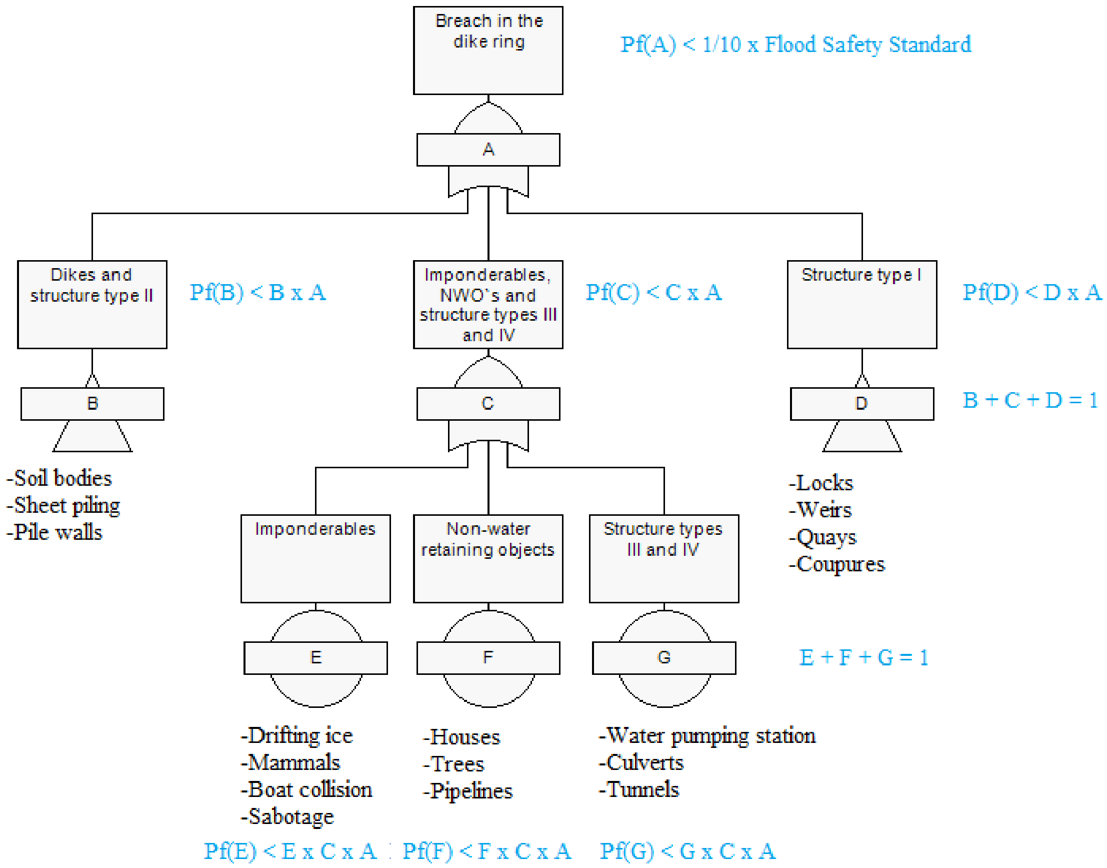


Figure 27: Subdivision of the types of flood defences in a dike ring and distribution of failure spaces.

If a couple of structures of the same type and with (almost) equal loading are present, it is allowed to cluster these structures. For these clusters, it is not obligated to further subdivide the failure probability space. The main reason for this, is the large dependency through hydraulic loading. For the system of flood defences surrounding the Duifpolder, the scheme in Figure 27 is applied. This results in the scheme as shown in Figure 28.

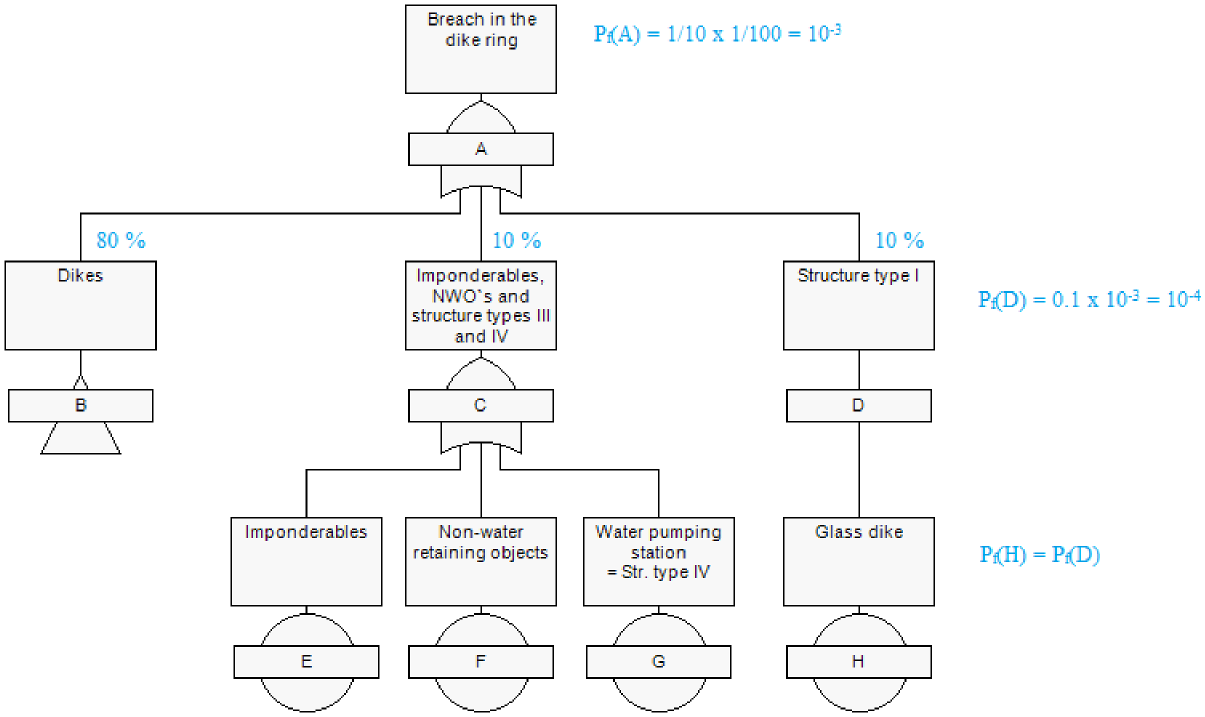


Figure 28: Failure probability spaces for flood defences surrounding the Duifpolder

Usually, applying the dike ring approach leads to a lower admissible failure probability on structure level compared to the dike section approach. However, the 'dike ring' that protects the Duifpolder mainly consists of dikes and only a very limited number of structures. As a result, each water retaining structure is assigned a relatively large failure space. For the glass dike, this results in an admissible yearly failure probability of $1 \cdot 10^{-4}$ ($\beta \approx 3{,}6$). Off course, this only holds if the distribution of failure probability spaces as shown in Figure 28 is applied. Note that this probability for structural failure is exactly equal to the maximum admissible probability for structural failure that follows from the dike section approach. From this, it is concluded that in the case of the glass dike, the dike ring approach does not lead to stricter safety requirements than the dike section approach.

5.6.2 Eurocodes

According to the Eurocodes, structural safety is obtained if a structure is sufficiently stable and structural elements contain sufficient strength. Eurocode 0 prescribes reliability indices (β) corresponding to reliability classes. These reliability indices can be used to calculate maximum allowed failure probabilities and are shown in Table 8.

Reliability class	Minimum value for β	
	1 year reference period	50 years reference period
RC3	5,2	4,3
RC2	4,7	3,8
RC1	4,2	3,3

Table 8: Eurocode reliability classes

Reliability classes in the Eurocode correspond to consequence classes: CC1, CC2 and CC3. For these classes the consequences of failure are described. Consequence class 1 is described as little economic damage, little consequences for human health and no social consequences. Reliability class 3 however, corresponds to large economic damage, loss of human lives and large social consequences. Consequence class 2 is in the middle of these classes with substantial economic damage, medium consequences regarding the loss of lives and medium social consequences. Structural failure of the glass dike will have limited economic consequences. However, if water is flowing into the structure, people might get trapped, hence there is a chance that lives are lost. Furthermore, if the failure is due to loss of strength of the glass elements, people will lose trust in glass as water retaining elements. This can be described as some form of social consequence. From these considerations, it is concluded that RC2 should be applied.

As can be seen in Table 8, reliability class 2 corresponds to reliability index β = 3,8 for a reference period of fifty years and β = 4,7 for a reference period of one year. In Eurocode 0, it is stated that designing with partial factors that are given in the Eurocodes will lead to a structure with a reliability index minimum as high as given for the reliability class 2. For the other classes, a correction factor is given that should be applied to the partial factors. However, it is also stated that this reliability only holds on the level of a structural element (strength) or a single failure mechanism on structure level (stability). A structure is composed of a number of structural elements and can fail in multiple ways. Therefore, the reliability of the structure as a whole, is (much) smaller than the reliability of a single structural element.

5.6.3 Comparison of the Eurocode with the Flood Safety Standard

Now, the required reliability of the glass dike is defined with respect to both the Flood Safety Standard as with respect to the Eurocode. As mentioned before, these requirements partially overlap each other. This overlap is shown in Figure 29.

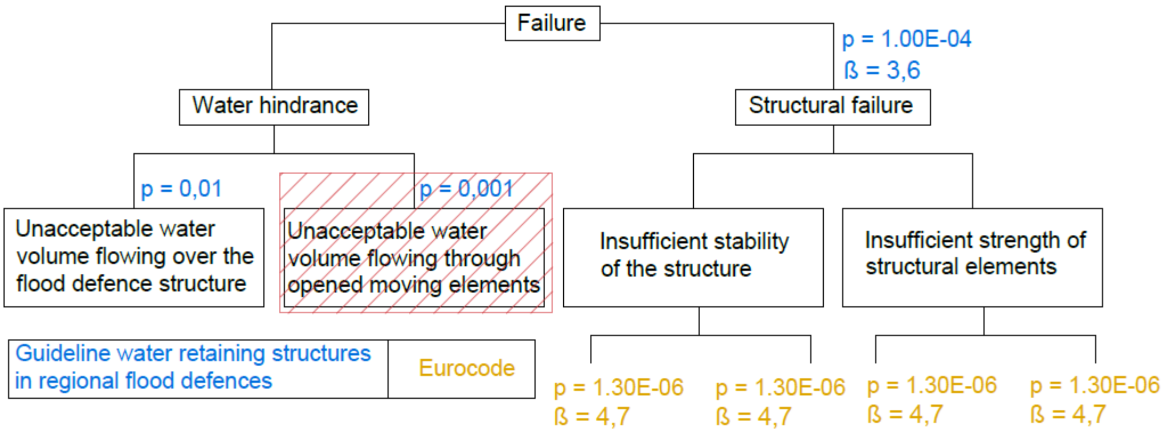


Figure 29: Comparison of reliability requirements from the Flood Safety Standard and Eurocodes

As can be seen, the overlap is on strength and stability of the structure. For overlapping parts, the most stringent requirement should be used. Since reliability requirements from the Eurocode apply to individual failure mechanisms, while the required reliability that follows from the Flood Safety Standard applies to the structure of the glass dike as a whole, it is not clear which is governing. In chapter 6, partial factors from Eurocode reliability class 2 are applied to obtain the semi-probabilistic design. Then, it should be checked whether this lead to a reliability of the total structure (system) that satisfies the reliability as prescribed by the Flood Safety Standard. This is verified in chapter 7.

5.7 Program of requirements

This section gives an overview of the requirements regarding the design of the glass dike. These are divided into safety, serviceability, technical and other requirements. Some requirements (e.g. flood safety standards) are strict, while others are preferences. The engineering design of the glass dike must fulfill strict requirements at all times. Preferences can play a role in the decision process of different design solutions.

5.7.1 Safety

Retaining water

- The glass dike is part of a regional flood defence which must be designed to safely withstand a water level with an exceedance probability of 1/100 per year (strict)

Dutch building code 2012 (Bouwbesluit)

The design of the glass dike must satisfy requirements that are set by the Dutch building code 2012 (Bouwbesluit) (strict). The Dutch building code 2012 is applicable to new and existing buildings and gives regulations on the following subjects:

1. Structural safety
2. Fire safety
3. Health
4. Functionality
5. Energy-efficiency and the environment
6. Prescriptions for installations
7. Prescriptions for the use of buildings and open terrains
8. Construction and demolition works

Construction works should always have a place in the design process. However, the regulations in the Dutch building code 2012 regarding construction and demolition works do not include design related requirements, but are more related to general safety aspects like for instance the correct (read: safe) placement of cranes. For the engineering design described in this report, only regulations on the subject of structural safety are included. The glass dike also has to satisfy requirements on the other subjects, but these are not treated in this report. For structural safety, the Dutch building code 2012 refers to the Eurocodes. The requirements below are related to the Eurocodes:

- With respect to structural safety, the glass dike has to be designed according to Eurocode reliability class RC2 (strict)
- Structures that are accessible to people should be robust. In practice, this means that vulnerable structural elements must be able to fail, without succumbing of the total structure (strict)

5.7.2 Serviceability

Aesthetic

- The glass elements should retain water to a depth of at least 0,50 m (strict)
- The glass elements should retain water to a depth of 0,65 m (preferred)
- Given low water, in winter it should be possible to look under a layer of 0,15 m thick ice from inside the glass dike (strict)
- It should be possible to look through the glass dike from both the polder as from the water (strict)
- The transition of the shape of the soil body and the glass dike should be smoothly (strict)
- The dimensions of the cross-section of the glass dike should be exactly equal to the shape the cross-section of the adjacent soil body (preferred)
- Columns inside the glass dike to support the roof should be avoided (preferred)

Accessibility & dimensions

- The glass dike should be accessible to people (strict)
- The glass dike should be large enough to provide a bride suite and expo. For this, dimensions are given in Figure 21 (strict)
- The footpath on top of the dike should be preserved (strict)

Preventing water hindrance

- According to the water regulation Zuid-Holland, the probability of exceedance of a water quantity in the polder larger than the storage capacity is 1/25 per year (strict)
- The retaining height of the glass dike should be minimum equal to N.A.P. +0,10 m (strict)
- The water retaining glass elements may not be able to open or close (strict)
- Leakage at the connections of the glass elements may not occur (strict)

5.7.3 Technical

Constructability

- During construction, the flood defence must fulfill its water retaining function without reduction of safety against flooding (strict)
- In case a pile foundation is applied, a construction method should be applied such that large pore pressures in the dike will not occur. Water board Delfland prescribes the use of foundation techniques which induce no or small pressure waves in the soil (strict)
- The glass dike should be constructible (strict). Using standard materials, shapes and construction techniques will reduce risks and costs (preferred)

Maintenance and inspection

- Elements of the glass dike that require inspection should be accessible without the need to make large changes to the soil body or structure (strict)
- Elements of the glass dike that require maintenance should be accessible without the need to make large changes to the soil body or structure (strict)

Removability

- With respect to the removability of the structure, a shallow foundation should be applied (preferred)

5.7.4 Other

Sustainability

- It should be possible to adjust the height of the glass dike (preferred). For instance if the flood safety level is changed in future
- The reference period (lifetime) of the glass dike is 100 years (strict)
- Demands with respect to the functionality and safety of the glass dike should be satisfied during the entire lifetime (strict)

Economic/financial

- Financial/economic aspects should play a role in the decision process when different technical solutions are evaluated (preferred)

6. Semi-probabilistic engineering design

Before the reliability of the glass dike can be evaluated, there should be a design with technical solutions. In this chapter, the architectural design of the glass dike will be upgraded to an engineering design with hydraulic engineering solutions. By looking at the loads and boundary conditions, relevant failure mechanisms will be identified. Based on the requirements that were determined in previous chapter, design solutions to prevent these failure mechanisms will be selected. After that, the structure is dimensioned and design checks are performed. This is done by using a combination of technical guidelines for flood defences and Eurocodes. Since the design of the glass elements plays a crucial role in the total design of the dike, this is elaborated in a separate chapter (8).

6.1 Description of the structure

The engineering design of the glass dike as proposed in this chapter, is obtained by mainly regarding the primary function: retaining water. An important part of the design process is performing safety checks for relevant failure mechanisms. In order to properly identify failure mechanisms and necessary safety checks, it is important to determine which parts/elements of the glass dike structure contribute to the flood protection function and which do not. To do this, the following assumptions are made:

- Water retaining glass elements are supported by the roof and wall at the waterside
- The wall at the waterside is supported by the floor, side walls and one intermediate wall
- The roof is supported by the side walls and one intermediate wall
- The side walls and intermediate wall are supported by the floor
- The floor is supported by a pile foundation or directly by the soil
- The glass elements on the polder side of the dike are supported by the roof and floor

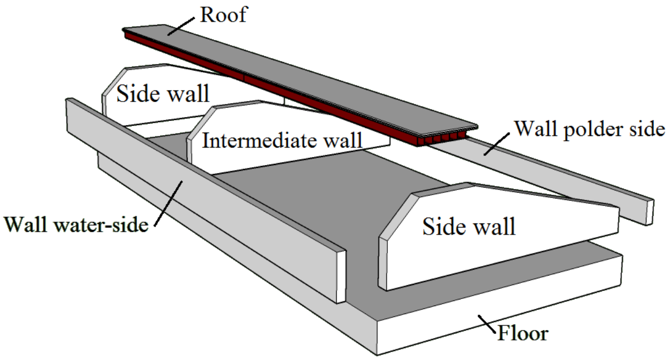


Figure 30: Structural elements (glass excluded)

From these assumptions it follows that glass elements on the polder side of the glass dike do not contribute to the flood protection function. An overview of the elements that do contribute to the flood protection function is shown in Figure 31.

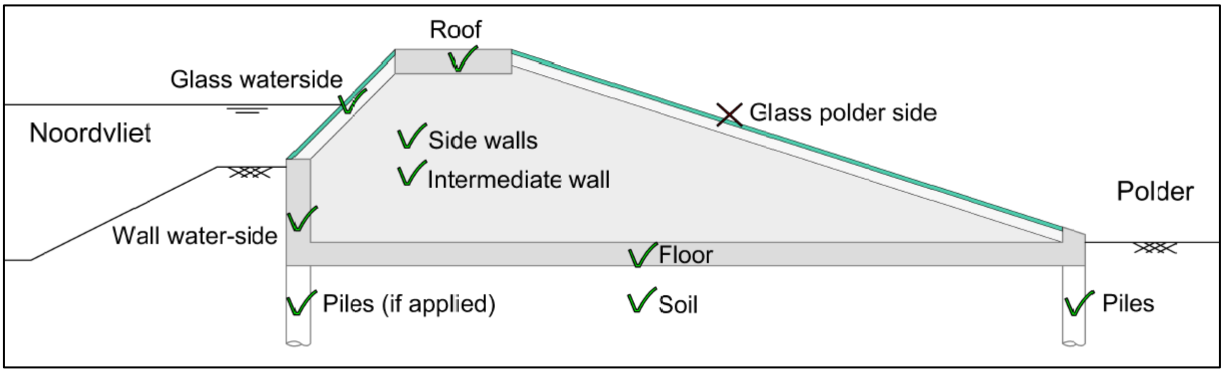


Figure 31: Overview of parts/elements with water retaining function

An important aspect of the design, is the interaction of the structure with the surrounding soil body. The transitions between soil and structure is part of the design. Figure 32 gives an overview of the physical design boundaries.

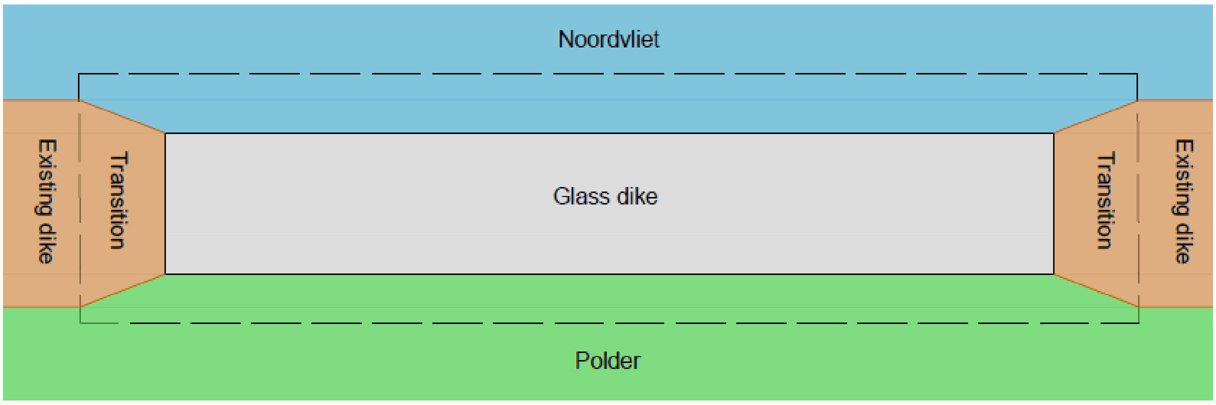


Figure 32: Physical design boundaries of the glass dike (top view)

6.2 Failure mechanisms

In order to make an engineering design of the glass dike, failure mechanisms that could occur must be identified. The way by which the water retaining function is negatively affected, is called a failure mechanism. A distinction is made between failure and succumbing. If a flood defence loses its water retaining function, it is failing. Succumbing has occurred if the flood defence shows large geometrical deformations due to collapse or distortion. Failure is possible without succumbing of the flood defence. An example is overflowing water due to very high water levels. Vice versa, succumbing is possible without failure (TAW, Grondslagen voor Waterkeren, 1998). For the glass dike, this could be the case when a glass element is broken while the water level is lower than the element. In many cases, succumbing leads to failure. Consequences of failure without succumbing are often small compared to consequences of failure due to succumbing of the flood defence.

6.2.1 Summary of failure mechanisms

Failure without succumbing of the glass dike actually means that unacceptable amounts of water are flowing into the polder, without loss of strength and stability of the structure. This is possible if water flows over the glass dike. In this report, this type of failure is referred to as non-structural failure or water hindrance. The following failure mechanisms could be relevant (LK2003):

- Overflow
- Wave overtopping

In this report, overflow and wave overtopping are only called failure mechanisms if the amount of water that is passing the flood defence becomes unacceptable. This depends on the storage capacity of the water system inside the polder. Unacceptable overflow and wave overtopping discharges can occur if the height of the flood defence is insufficient or if the water level and waves are high. Usually, these mechanisms can also undermine the stability of the flood defence due to erosion of the inner slope and surrounding soil body. The overflowing water and overtopping waves are then actually causes of succumbing.

Some structural elements could collapse, without loss of the water retaining function of the glass dike. Therefore, if the structure succumbs, this does not necessarily imply failure. However, often immediate repair is needed in order to prevent failure later in time, for example when hydraulic loads are expected to become larger than the residual resistance against failure mechanisms. In most cases succumbing is caused by large hydraulic loads. Loss of strength and/or stability then often leads to direct failure. For this reason, succumbing is regarded as a failure mechanism, even if the water retaining function is not lost (yet). The causes of succumbing are divided in loss of strength of structural elements, loss of stability of the structure as a whole and failure of the adjacent soil bodies (transitions). Partly based on LK2003 and (TAW, Grondslagen voor Waterkeren, 1998), the following failure mechanisms could be relevant:

Loss of stability

- Lateral shear
- Overturning
- Uplift
- Insufficient bearing capacity of the soil
- Erosion (Due to overtopping, overflow and currents)
- Undermining
  - Piping
  - Human actions
  - Pipe leakage
  - Animal holes

Loss of strength

- Structural elements that contribute to the water retaining function (Figure 30)
  A. Bending
  B. Shearing
  C. Buckling
  D. Tension
  E. Impact
  F. Dynamic fatigue
  G. Static fatigue
  H. Punch

Different failure mechanisms apply to different structural elements. In Table 9, an overview is given of the structural elements (also see Figure 30) and corresponding potential failure mechanisms.

Structural element	Failure mechanism(s)
Glass elements (water retaining)	A, E, G
Wall water-side	A, B
Roof	A, B
Floor	A, B
Piles (if applied)	B, C, D
Side walls	A, B
Intermediate wall	C
Columns (if applied)	C, H

Table 9: Loss of strength, structural elements and corresponding potential failure mechanisms

A first check of the resistance against most of these failure mechanisms can be obtained by looking at a cross-section of the glass dike. This does not hold for the transitions with the adjacent dike sections as these are often sensitive to erosion. This is especially the case if settlements of the structure and the soil body are unequal. Overflowing water and overtopping is then concentrated at these locations. This illustrates that interaction of soil and structures can lead to an increased probability of failure mechanisms to occur. At transitions, similar failure mechanisms as for dikes are relevant. In addition, interactions with the structure should be taken into account. Partly based on LK2003 and (TAW, Grondslagen voor Waterkeren, 1998), the following failure mechanisms could be relevant:

Transitions

- Macro instability of adjacent soil bodies
- Micro instability of adjacent soil bodies
- Instability of soil retaining elements (e.g. sheet piling)
- Erosion (due to wave attack, overtopping, overflow and currents)
- Undermining
  - Piping
  - Outflanking
  - Human action
  - Pipe leakage
  - Animal holes

Different geometric conditions and load combinations can lead to different sliding planes in the soil. Therefore, the failure mechanism macro instability actually consists of multiple forms of similar failure mechanisms. The failure mechanism 'erosion' also consists of different similar failure mechanisms and can be subdivided by cause: Wave attack, overtopping, overflow and currents. Another thing that should be mentioned is that the design input as shown in previous chapter does not include sheet piles. However, it is likely that the engineering design includes seepage screens in the form of sheet piles. Therefore, the failure mechanism 'instability of soil retaining elements' is already mentioned in this section.

6.2.2 Relevant failure mechanisms

The glass dike is situated at water with relative mild hydraulic boundary conditions. It is therefore likely that not all of the identified failure mechanisms are relevant for the design. Other boundary conditions and design choices may influence the possibility of failure mechanisms to occur as well. For example, sliding is an important failure mechanism to consider if a shallow foundation is applied. In case of a pile foundation, the probability of this failure mechanism to occur is much smaller since the piles have to shear first. Similar reasoning holds for the bearing capacity of the soil. In this section, a selection is made of the identified failure mechanisms, which are actually relevant for the engineering design of the glass dike. This is done by comparing the boundary conditions with the necessary circumstances for failure mechanisms to occur.

Overflow & overtopping

The glass dike has to satisfy the minimum height requirement as imposed by the water board. The design water level is well below this height, so overflow cannot be an issue. However, if a shallow foundation is applied, settlements could lead to a decreased 'crest-level' of the structure. Overtopping can only occur during high water and if there are waves that propagate towards the glass dike. Even if this occurs, the overtopping discharge will be low since the waves are very small.

Sliding, overturning, uplift & soil bearing capacity

For hydraulic structures on shallow foundations, sliding, overturning, uplift and insufficient soil bearing capacity are all relevant. Sliding and overturning are not relevant in case of a pile foundation, but uplift and the bearing capacity of the soil in horizontal and vertical direction still need to be checked. The soil is very week at the desired location of the glass dike. Achieving a life time of 100 years without unacceptable settlements is regarded unlikely without a pile foundation. Furthermore, for the functionality of facilities inside the glass dike, even small settlements are not desired. For now, it is assumed that a pile foundation is applied. The correctness of this assumption will be verified in the design process.

Undermining

Since no pipelines are situated near the location of the glass dike, undermining due to pipe leakage is not a relevant failure mechanism. However, some new connections will be made to the glass dike in order to provide drinking water and to dispose waste water. These lines will probably be situated below the structure. This may influence the location of the seepage screens. After realization of the glass dike, someone (e.g. the operator) can choose to make some adjustments to the structure. He/she could for instance choose to construct a storage basement. This is an example of human action, which could seriously undermine the stability of the structure and transitions. This failure mechanisms is relevant, but not relevant for development of the engineering design. Animals can also undermine the stability of flood defences. Mammals can dig holes in the soil that is surrounding the glass dike structure. This can undermine the stability of the structure itself and the transitions. These mammals will drown if they dig holes under the phreatic line of the dike, so no undermining is expected under the structure. At the transitions, above the phreatic line, these animals can cause problems. However, this is not different than other parts of the dike and monitoring is performed to prevent this type of undermining. Therefore, animal holes are not regarded as a relevant failure mechanism for the design of the glass dike.

Piping and outflanking are only relevant for hydraulic structures founded on sand and/or surrounded by sandy soil bodies. Piping can also occur if a sand layer, which is directly connected with the channel, is situated below a cohesive impermeable layer. This impermeable layer has to be lifted up first, before erosion can occur and pipes are developed. The glass dike however, is surrounded by clay and peat. On the inner side of the dike, there is a thin sand layer. This layer is not connected to the channel. Therefore, piping and outflanking are not relevant for the engineering design. However, due to settlements under pile foundations a gap can be formed between the soil and the structure, in which a flow of water might be developed. This can also be described as some form of piping or outflanking and should be taken into account in the engineering design. (TAW, Technische Rapport Zandmeevoerende Wellen, 1999)

Loss of strength
Insufficient strength is by definition a relevant failure mechanism for the design of a structure. Different types of structural failure correspond to different structural elements. For instance, buckling is relevant for slender elements loaded in compression, while bending and shearing are relevant for elements loaded in transverse direction. The design checks that should be performed for the structural (water retaining) elements will become clear after the stress distributions for different load combinations are obtained.

Macro instability, micro instability & erosion
For aesthetic reasons, it is preferred to design the glass dike such that the shape and dimensions of the cross-section are equal to those of the adjacent dike. In that case, the soil body at the transitions will not deviate from other (near) parts of the dike. The macro stability of the dike is recently evaluated (Bardoel & Leemans, 2013). Constructing a glass dike will not negatively influence this stability. Therefore, macro stability is not relevant for the first engineering design. If the soil body at the transitions does deviate from the dike, the macro stability should be re-evaluated. Due to the impermeable soil layers around the glass dike, micro-stability is an irrelevant failure mechanism (TAW, Technisch Rapport Waterkerende Grondconstructies, 2001). At the transitions on the outer side of the glass dike, erosion could be a problem. Although the dike itself has no erosion protection, at the transitions of structures with soil, protection might be necessary. Due to the mild hydraulic boundary conditions, no erosion is expected at the transitions on the polder-side of the structure. In *Figure 31*, it can be seen that a slope of soil will be situated under water in front of the glass dike. This sloped layer of soil has to be eroded entirely and further to jeopardize the stability of the structure. Since the soil is very cohesive, this is not likely.

Overview
Table 10 gives an overview of the failure mechanisms that are relevant for the first engineering design of the glass dike.

Water hindrance	Stability	Strength	Transitions
Overflow[a]	Sliding[a]	Water retaining glass	Macro instability[d]
-	Overturning[a]	Wall water-side	Outflanking[b]
-	Uplift	Roof	Erosion outer slope
-	Insufficient soil bearing capacity	Side walls & Intermediate wall	-
-	Piping[b]	Floor	-
-	Pipe leakage	Columns[c]	-
-	-	Piles	-

Table 10: Overview of relevant failure mechanisms

a: Only if a shallow foundation is applied instead of a pile foundation.
b: No pipes in the soil, but gaps directly between the structure and the soil.
c: Only if columns inside the glass dike are applied.
d: Only if the cross-section of the dike profile at the transitions is expected to be less safe than the existing dike profile.

The failure mechanisms mentioned in this section are only the most important ones that are needed to obtain a first engineering design, which is needed to evaluate the reliability. It should be noted that extreme incidental loads are not mentioned as failure mechanisms in this section. However, extreme loads on the glass, like a boat collision, can be regarded as failure mechanism. This is elaborated in the design of the glass elements.

6.3 Loads

The loads that are included in the design of the glass dike are shown in Figure 33. Additional to these loads, soil pressures and hydrostatic pressures are also acting on the side wall which can only be seen in a length section.

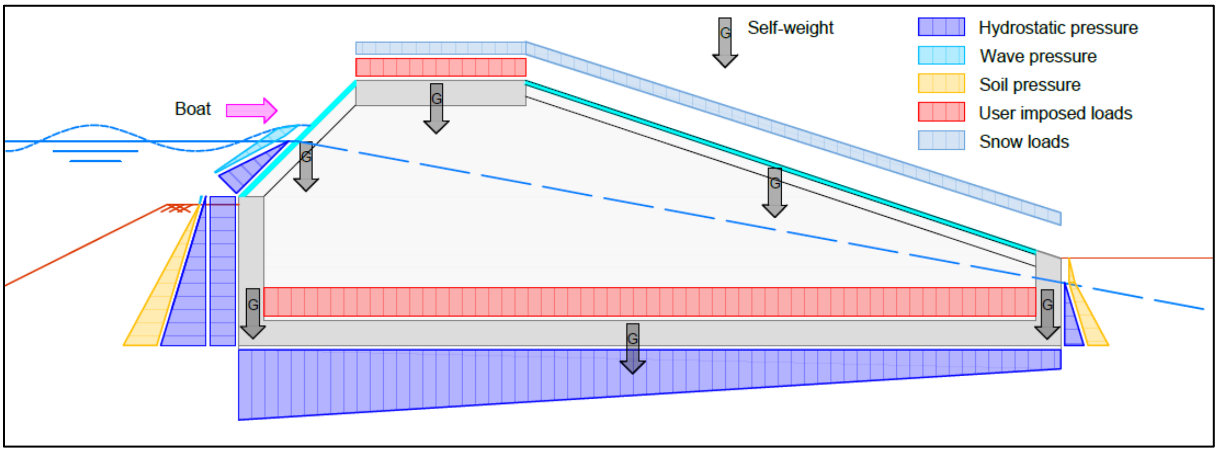


Figure 33: Schematisation of design loads

Hydraulic- and geotechnical loads are dominant for stability calculations of the structure. Boats can only collide against the glass elements. It is assumed that the concrete wall at the waterside and the roof are minimum as strong as the glass elements. Therefore, resistance against a boat collision is elaborated in chapter 8. As mentioned in previous chapter, ice-, wind- and boat induced wave loads can be neglected.

6.3.1 Load combinations & partial factors

The glass dike has to be designed according to reliability class RC2 in the Eurocodes. In order to accomplish this, partial load factors from the Eurocodes are applied. One way to achieve differentiation in reliability classes is to apply multiplication factors K_{FI} on the prescribed partial factors (for unfavourable loads). For reliability class RC2, this factor is equal to 1,0. Hence the prescribed load factors can be directly applied. The 'Guideline for water retaining hydraulic structures' also prescribes load factors. However, these correspond to the highest safety class in the old Dutch building safety standards (TGB) and are therefore too conservative. Per design check, the governing load combination and partial factors are determined. These are applied according to the expression below (NEN-EN 1990+A1+A1/C2, 2011).

$$\sum_{j\geq 1} y_{G,j} G_{c,j} + y_{Q,1} Q_{c,1} + \sum_{i>1} y_{Q,i}\, \psi_{0,i} Q_{c,i}$$

In which:

$G_{c,j}$	= Characteristic value of permanent load j	[kN]
$Q_{c,1}$	= Characteristic value of dominant variable load	[kN]
$Q_{c,i}$	= Characteristic value of variable load i	[kN]
$y_{G,j}$	= Partial factor for permanent load j	[-]
$y_{Q,1}$	= Partial load factor for dominant variable load	[-]
$y_{Q,i}$	= Partial load factor for variable load i	[-]
$\Psi_{0,i}$	= Reduction factor for non-dominant variable load i	[-]

The Eurocode allows hydraulic loads to be taken into account as permanent loads, even if the load is not constant in time. For permanent loads, load factors are lower. Since the design water level already is a 1/100 year water level, this is justified. Other permanent loads are self-weight and soil pressure. Variable loads are snow, waves and user imposed loads. Boat collisions are characterized as incidental loads and are always dominant.

6.3.2 Hydraulic loads

Hydrostatic pressure
The hydrostatic pressure and hydrostatic pressure force are calculated with respectively the following formula`s:

$p = \gamma_w h$ and $F_h = \frac{1}{2}\gamma_w h^2$

In which:

p	= Hydrostatic pressure	[kN/m^2]
F_h	= Force resulting from hydrostatic pressure	[kN/m]
γ_w	= Specific weight of water	[kN/m^3]
h	= Water depth	[m]

Applying the load factor can be done in two ways. One possibility is to multiply the water depth with the load factor and then calculate the hydrostatic pressure force. The other possibility is to calculate the hydrostatic force based on the design water level and then apply the load factor. Since the formula for the force is proportional to the square of the water depth, the first option results in the highest hydrostatic pressure force. When designing flood defences, often the most conservative method is used. However, multiplying the water level, which already is based on a probability of exceedance of 1/100 per year, with a partial factor results in an unrealistic high water level and consequently unrealistic high hydrostatic pressure force. Therefore, it is chosen to apply the load factor after the hydrostatic force is calculated. A similar approach is used when wind loads on buildings are calculated, for which wind pressures are proportional to the square of the wind velocity.

Wave pressure
For the calculation of wave pressures the following assumptions are made:

- Waves do not break, but are fully reflected
- The wave height at the glass dike is twice the height of the incoming wave
- The wave pressure is hydrostatic above the water line
- The wave pressure is maximum at the water line
- The wave pressure decreases linear under the water line and is zero at the bottom

Since clay is impermeable and the time scale of waves is very small, wave pressures cannot develop in the pores of the soil. The wave pressure at the waterline is calculated with the following formula:

$$w_1 = \gamma_w a_l = \frac{1}{2}\gamma_w H_l = \frac{1}{2}\gamma_w 2H_i = \gamma_w H_i$$

In which:

w_1	= Wave pressure at the waterline	[kN/m^2]
γ_w	= Specific weight of water	[kN/m^3]
a_l	= Amplitude of the wave at the structure	[m]
H_l	= Height of the wave at the structure	[m]
Hi	= Height of the incoming wave	[m]

6.3.3 Geotechnical loads

For geotechnical loads, a distinction is made between active-, passive- and neutral soil pressure. Which type should be applied depends on deformations and/or displacement of the structure. Passive or active soil pressure develops when the structure/element displaces/deforms respectively towards or from the soil. This holds for horizontal soil pressures. In case there is no or very little displacement, neutral soil pressure should be used in the calculation. First the vertical effective soil pressure is calculated. This is done with following formula:

$$s'_v = \sum_{i=1}^{n} y_{d,i} d_i + \sum_{j=1}^{m} y_{w,j} d_j - p$$

In which:

s'_v	= Effective vertical soil pressure	[kN/m^2]
$y_{d,i}$	= Dry specific weight of layer i	[kN/m^3]
d_i	= Thickness of layer i	[m]
$y_{w;j}$	= Wet specific weight of layer j	[kN/m^3]
d_j	= Thickness of layer j	[m]
p	= (Ground)water pressure	[kN/m^2]

Note that a distinction is made between soil layers above the (ground)water level and soil layers below the (ground)water level. The effective horizontal soil pressure is calculated by multiplying the vertical effective soil pressure with a coefficient for active, passive or neutral soil pressure. The coefficients are calculated with help of formulas of Rankine (Jaky for neutral soil pressure coefficient):

$$s'_{h;a/p/n} = K_{a/p/n} s'_v$$

$$K_a = \frac{1-\sin(\varphi')}{1+\sin(\varphi')}$$

$$K_p = \frac{1+\sin(\varphi')}{1-\sin(\varphi')}$$

$$K_n = 1 - \sin(\varphi')$$

In which:

$s'_{h;a/p/n}$	= Effective horizontal soil pressure	[kN/m^2]
K_a	= Coëfficient for active soil pressure	[-]
K_p	= Coëfficient for passive soil pressure	[-]
K_n	= Coëfficient for neutral soil pressure	[-]
φ'	= Effective intergranular friction angle	[°]

The horizontal soil pressure forces are obtained by integrating the soil pressures over the height of the soil layers. As can be seen in the boundary conditions, a layer of peat is present approximately at and above the level of the foundation. It is assumed that this layer is completely removed during construction of the glass dike and replaced by clay. Therefore, soil characteristics for clay are used when calculating soil pressures against the structure.

6.3.4 Snow loads

The magnitude of snow loads depends on the height and location of the structure. Eurocode 3 prescribes location specific formulas for calculation of the snow load. For the location of the glass dike this is shown below:

$$S = 0{,}164Z - 0{,}082 + \frac{A}{966} = 0{,}164 \cdot 3 - 0{,}082 + \frac{0{,}1}{966} = 0{,}40 \text{ kN/m}^2$$

In which:

S	= Value of the snow load	[kN]
A	= Height above N.A.P.	[m]
Z	= Zone number on maps given in the Eurocode	[-]

6.3.5 User imposed loads

The user imposed load on the floor depends on the type of usage. Eurocode 1 describes different load classes. For the floor of the glass dike, load class C3 (restaurants, cafes, dinner rooms, reading rooms etcetera) is applicable. For this class, a distributed load of 4,00 kN/m^2 should be applied.

The roof should in fact be regarded as a bridge for pedestrians and cyclists. Therefore, the loads prescribed by Eurocode 2 for loads on bridges should be applied. For the distributed load, the value must be obtained with the expression below:

$$q_f = 2{,}0 + \frac{120}{L + 30} = 2{,}0 + \frac{120}{22 + 30} = 4{,}30\ \mathrm{kN/m^2}$$

In which:

q_f	= Value of the user imposed distributed load	[kN/m^2]
L	= Length of the span	[m]

User imposed concentrated loads are only used to design details of structural elements. For instance, the connection of two elements. For general strength calculations, as performed in this thesis, only the distributed load is applied.

6.3.6 Pressure indices

Hydrostatic-, wave- and soil pressures are dependent on the dimensions of the structure and the boundary conditions. These loads are computed for different situations. Since this procedure is repeated multiple times for different design checks, indices are given for pressures at specific points on the glass dike. This way, the calculations become more uniform. The indices are given in Figure 34.

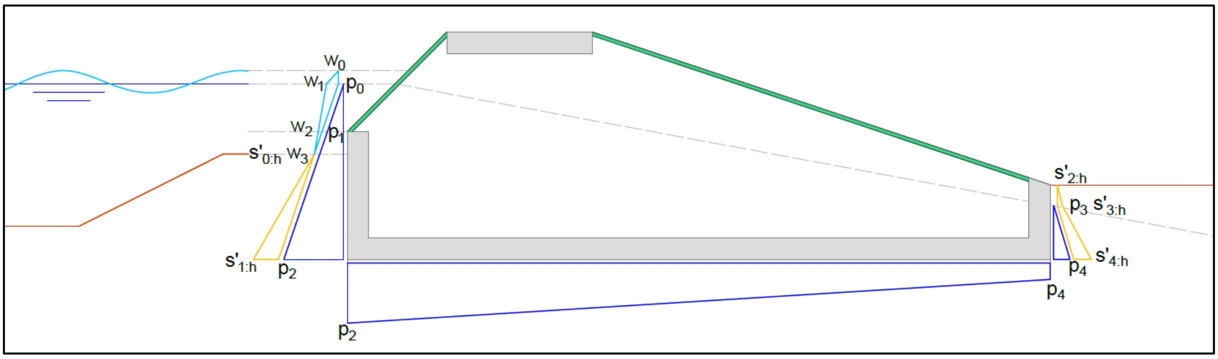


Figure 34: Pressure indices

w_0	= Wave pressure at the crest of the wave (=0)	[kN/m^2]
w_1	= Wave pressure at the waterline	[kN/m^2]
w_2	= Wave pressure at the bottom of the glass (top of the concrete wall)	[kN/m^2]
w_3	= Wave pressure at the water bottom (=0)	[kN/m^2]
p_0	= Hydrostatic pressure at the water line (=0)	[kN/m^2]
p_1	= Hydrostatic pressure at the bottom of the glass (top of the concrete wall)	[kN/m^2]
p_2	= Hydrostatic pressure at the bottom of the floor on the water-side	[kN/m^2]
p_3	= Hydrostatic pressure at the groundwater level on the land-side (=0)	[kN/m^2]
p_4	= Hydrostatic pressure at the bottom of the floor on the land-side	[kN/m^2]
$s'_{0;h}$	= Effective soil pressure at the water bottom (=0)	[kN/m^2]
$s'_{1;h}$	= Effective soil pressure at the bottom of the floor at the water-side	[kN/m^2]
$s'_{2;h}$	= Effective soil pressure at the ground level on the land-side (=0)	[kN/m^2]
$s'_{3;h}$	= Effective soil pressure at the groundwater level on the land-side	[kN/m^2]
$s'_{4;h}$	= Effective soil pressure at the bottom of the floor at the land-side	[kN/m^2]

In the calculations, the type of soil pressure (horizontal/vertical, active/passive/neutral) is stated behind the index. Example: effective horizontal (neutral) soil pressure at the bottom of the floor is denoted as: $s'_{1;h;n}$

6.4 Upgrading the architectural design to a hydraulic design

6.4.1 Approximate dimensions

In order to properly perform design checks, the main dimensions of the glass dike are determined. As mentioned before, the cross-section of the glass dike should be (almost) equal to the cross-section of the soil body of the adjacent dike sections. Therefore, the visible slopes and crest of the glass dike will be equal to that of the soil body. As a result, the structure will be 9,90 meters wide instead of 7,40 meters as proposed in Figure 21. The length will remain 22,00 meters. The 'crest level' will be equal to N.A.P. + 0,30 meter as this is the level of the adjacent dike just after reinforcement. By doing so, it is 0,33 meter higher than the design water level, thereby preventing overflow. The depth of the structure is determined by the thickness of the roof, the thickness of the floor and the required height of the ceiling. For the latter, the Dutch building code 2012 prescribes a minimum height of 2,60 meters. This is adopted in the design. The main preliminary dimensions of the cross-section of the glass dike, with rough estimates of the thicknesses of structural elements, is shown in Figure 35. After the design checks are performed, the final dimensions of the first engineering design are given.

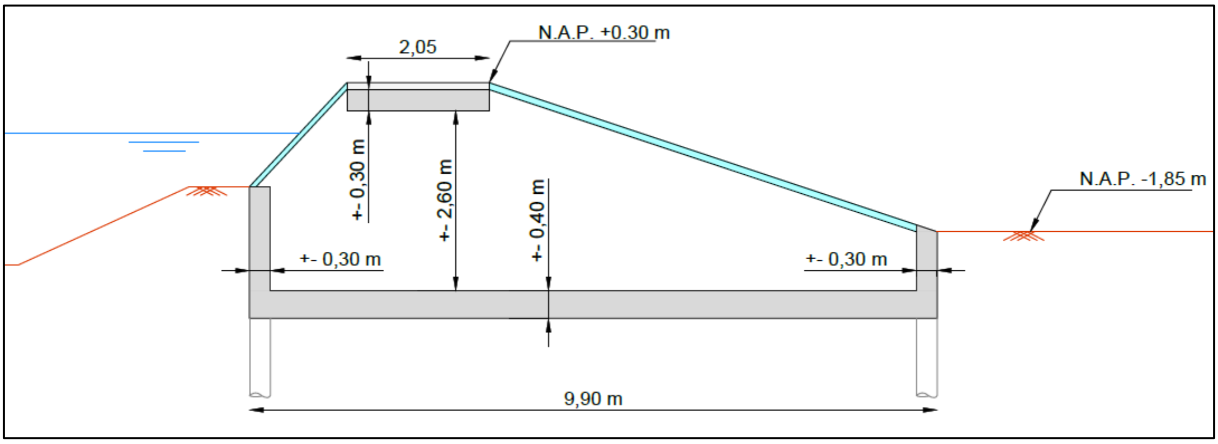


Figure 35: Preliminary dimensions in a cross-section of the glass dike

6.4.2 Design variants

Making choices for technical solutions is a procedure that is intertwined with performing design checks. For example: The type of piles that should be applied (as will be determined in next section) depends partly on the forces that act on the piles. Another example is the building material for structural elements. The roof will be subjected to a large bending moment. Therefore, an alternative for 'simple' reinforced concrete could be more attractive or even necessary. This kind of design choices are discussed in this section. Where needed, reference shall be made to the appendix with the relevant design check. Since the engineering design will be based on input from an architectural design, there will be no proposal and selection of main design alternatives.

Pile foundation

It was assumed that a pile foundation should be applied. This assumption will be verified in section 6.5.1. The piles should be able to resists compression as well as tension loads. Furthermore, the requirement to use a foundation technique that induces only small or no pressure waves in the soil, as stated in section 5.7.3, is applicable. Another consideration to be taken into account is the low bearing capacity of the soil, making it impossible to use very heavy equipment on the location of the glass dike. The pile type and execution method are strongly related. The following techniques are considered:

- Weight drop hammering
- Internal weight drop hammering
- Vibration
- Pressing
- Drilling
- Screwing

The methods, corresponding pile types, advantages and disadvantages are shown in Table 11 (Jansen, Janssen, & Muskens, 2005), (Vroom Funderingstechnieken, 2015), (Volker Staal en Funderingen, 2015).

Execution method	Pile type(s)	Loads	Advantages	Disadvantages
Weight drop hammering	Concrete or Steel	Tension & Compression	-Soil displacing method -Low cost -Maximum inclination 1:1	-Large pressure waves in soil
Internal weight drop hammering	Steel tube with concrete fill	Tension & Compression	-Small pressure waves in soil -Soil displacing method -Light equipment -Low to medium cost	-Maximum inclination 1:5
Vibration	Concrete or Steel	Tension & Compression	-Small pressure waves in soil -Soil displacing method -Light equipment -Low cost	-Inclination not possible -Only small diameter piles
Pressing	Concrete or Steel	Tension & Compression	-No pressure waves in soil -Soil displacing method	-Very heavy equipment -Short length piles
Drilling (Auger pile)	Concrete	Tension & Compression	-No pressure waves in soil -Pile length can be adjusted in-situ while constructing	-Soil removing method -Not suitable for inclined piles in weak soil -High cost
Screwing	Steel tube with concrete fill	Tension & Compression	-No pressure waves in soil -Light equipment	-Soil removing method -High cost

Table 11: Pile types and execution methods, advantages and disadvantages

The most important criteria to choose a certain execution method and pile type are risk and costs. Other criteria only serve to determine if the method can be applied at all (Jansen, Janssen, & Muskens, 2005). For example: An expensive foundation method that does not induce pressure waves (vibrations) in the soil is never chosen over a cheap foundation method that induces large pressure waves if no damage can occur due to these waves. Another example is the equipment that is needed for execution. It is either possible to use heavy equipment on the construction site or it is not. An exception to this rule is if the criteria implicitly influences the total cost of the project, even if the method itself is more expensive.

The glass dike has to resist horizontal forces. In weak soil, piles are schematized as pendulums. Therefore, (part of) the piles need to be placed under an angle. Design checks for the number of piles and inclination angles are shown in section 6.5.1. Furthermore, only light weight equipment can be used for the placement of piles. As stated before, it is not permitted to induce large pressure waves in the soil as this can lead to overpressures in the pores of the soil, thereby compromising the stability of the dike. Based on these criteria, only two methods can be applied: Screwing and internal weight drop hammering.

Screwing piles does not lead to any pressure waves in the soil. Therefore it is an excellent foundation technique to use for the foundation of the glass dike. A disadvantage is that it is a soil removing method. A soil displacing method leads to larger shaft friction, thereby increasing the admissible tension/compression force. Internal weight drop hammering is a soil displacing method, but induces pressure waves in the soil. However, since the hammering is performed inside a steel tubular pile, the energy is transmitted directly to the tip of the pile. The effect is that horizontal pressure waves in the soil are strongly reduced. Since this internal weight drop hammering is also economically more attractive, this method is chosen. The corresponding pile type is a steel tubular pile, which is filled with (reinforced) concrete after placement. This pile type can be loaded in compression and tension, but is also able to resist bending moments induced by forces perpendicular on the pile.

Seepage screens

In section 6.2.2 it was mentioned that at the surface of concrete elements, gaps in the soil may develop due to settlements. In these gaps, a flow of water could arise. This is especially the case for structures with a pile foundation and could also be described as some form of piping. This is prevented by placement of short seepage screens. The design of these screens is elaborated in this section. A distinction is made between piping- and outflanking screens. Piping screens are placed below the structure, while outflanking screens are placed next to the structure at the transitions with the adjacent soil bodies. Piping- and outflanking screens are connected to each other. The following requirements should be satisfied:

- The seepage screens should be able to follow little displacements of the structure
- Displacement of the seepage screens may not lead to defects at the connections with the structure
- Outflanking screens should intersect the phreatic line for the 1 in 100 year water level

Since flows of water can only develop at the surface of the concrete, very short piping screens may be applied. A length of 1 meter is sufficient (TAW, Technische Rapport Zandmeevoerende Wellen, 1999). At the transitions, also the stability of the soil body plays an important role. In section 6.2.2 (Relevant failure mechanisms) it was mentioned that stability of the adjacent soil bodies is an important failure mechanism to consider if the soil body at the transitions becomes less safe than the original dike profile. In Figure 36, the red line represents the profile of the glass dike structure. The black line represents the soil in front of the glass dike. As can be seen, the total profile of the glass dike is smaller than the profile of the adjacent dike sections, depicted in brown. Then, also the soil body at the transitions is locally smaller than the profile of the adjacent dike sections. The loss in safety against instability at the transitions can be compensated by applying sheet piles. Applying outflanking screens with a width of 3 meters, measured from the connection with the glass dike, is assumed to be sufficient. Since the outflanking screens now contribute to the stability of the soil bodies, an additional requirement is that they should be sufficient strong and stable.

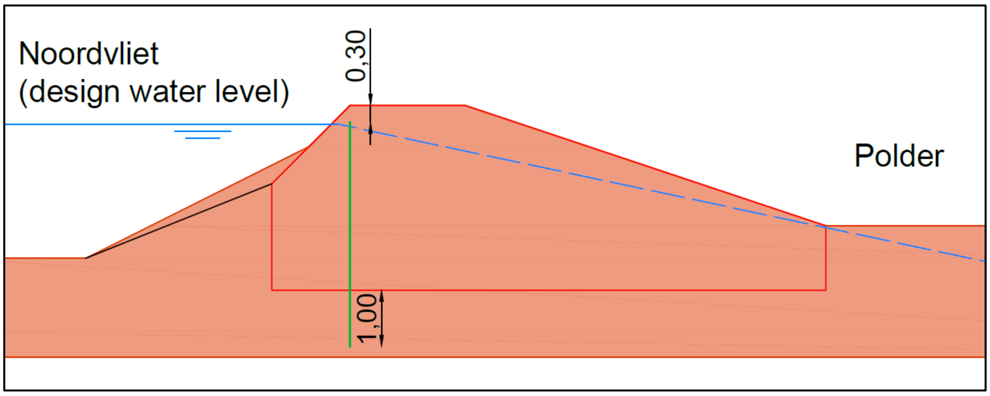


Figure 36: Location of the seepage screens

The considered materials that can be used are steel, wood and plastic. The outflanking screens are partially situated above the groundwater level. At this location wood may putrefy while plastic will remain unaffected. Steel is also expected to decay (corrosion), but in fresh water environments only with 0,02 mm per year on average on the water side of the flood defence. On the dry side of the flood defence, corrosion is negligible (CUR-publicatie 166, 1997). During construction, a cofferdam with steel sheet pilings is applied. Then, it is beneficial to also apply steel seepage screens as this can be constructed immediately after the cofferdam is finished. For this, the same equipment can be used. Other advantages are that steel is relatively cheap and that it can deform without large loss of strength, making it able to follow small displacements of the structure. By applying the same steel profile as used for the cofferdam, the outflanking screens are assumed to be sufficient stiff to ensure stability of the soil body at the transitions of the glass dike. Another aspect to consider is the location of the seepage screens when looking at a cross-section of the glass dike. Since waste water and drinking water connections will be established with pipe lines below the structure, coming from the polder side of the glass dike, it is best to situate the seepage screens as far as possible to the water-side. The considerations above, result in the seepage screens as depicted by the green line in Figure 36.

Roofstructure

The roof will be constructed with HE300B steel beams. The reason for this follows directly from the large bending moment that has to be resisted (see section 6.5.2). Using concrete for the roof would result in a thickness of 0,40 meter. In that situation, the largest contribution to the bending moment is the self-weight. Furthermore, to maintain a free height of 2,60 meter inside the glass dike, the total structure must be lowered, thereby increasing hydrostatic pressures on the glass, walls and floor. A small reduction of the roof thickness can be obtained if pre-stressed concrete beams are used.

However, more practical reasons play a role. The roof consists of two spans. One from the side wall on the North to the intermediate wall and one from the intermediate wall to the side wall on the South of the glass dike. See Figure 37. If the roof is constructed as one continuous beam on three supports, the largest bending moment is obtained at the intermediate support (wall). This moment is smaller than the maximum bending moment that is obtained if the roof is constructed as two beams (disconnected at the intermediate support). In-situ casting of a concrete roof is very impractical and delivering prefabricated concrete beams with a length of 22 meters at the location of the glass dike even more impractical. The advantage of steel beams is that they are much lighter and can be delivered in sections, which can be connected at the construction site. This way, it can be constructed more easily, thereby reducing risk and cost.

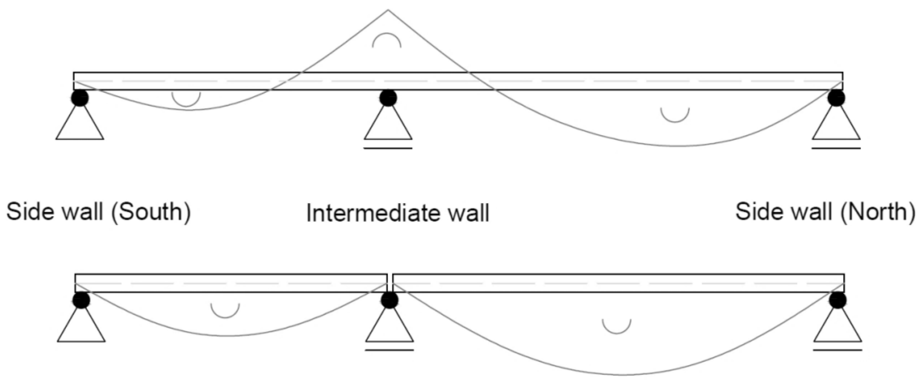


Figure 37: Schematization of the roof and corresponding qualitative bending moment lines

By welding plates to the flanges of the beams and making holes in these plates, the beams can be connected with bolts. Stresses from bending moments are transferred via these bolts. Therefore, the beams are connected at the point where the bending moment is approximately zero.

6.5 Design checks

In this section, design checks are performed to determine the size of elements and to verify if choices for design variants were properly made. The structure of this section is based on the main groups of relevant failure mechanisms, respectively: stability, strength and transitions.

6.5.1 Stability

Since the soil consists of weak layers, it was assumed that a pile foundation is necessary. This is verified in this section. In order to do this, design checks for a shallow foundation are performed. From this, it will appear if a shallow foundation is feasible. If not, a pile foundation should be applied, for which design checks are performed as well. The result of this section is a fully dimensioned foundation.

Shallow foundation - uplift

In this design check, the dimensions of the structure given in section 6.4.1 are adjusted to obtain a design with a shallow foundation that is safe for uplift. With this adjusted dimensions, the bearing capacity for a shallow foundation can be checked.

The hydrostatic pressure under the structure is the driving force for uplift to occur. The situation with the design water level of N.A.P. -0,03 m gives the largest upward pressure. All downward directed forces provide resistance against uplift. The least favourable situation is the case when only the self-weight of the structure is taken into account. Since friction between the soil and walls only have a minor contribution to the resistance, this is neglected. Furthermore, the soil in front of the glass dike might be eroded due to currents or propeller wash and the soil on the polder side of the glass dike might be removed for maintenance works or some other arbitrary reason. The structure is safe for uplift if the design value of the total self-weight is larger than the design value of the upward hydraulic pressure:

$$G_{stb;d} \geq U_{dst;d}$$

In which:

$G_{stb;d}$ = Design value for the total self-weight [kN]

$U_{dst;d}$ = Design value of the upward hydrostatic pressure force [kN]

The resistance against uplift can be increased by changing the thickness of concrete elements and/or by using concrete with higher density. Since the crest-level of the dike is fixed, only the concrete walls and the floor thickness can be increased. The largest effect is achieved by increasing the floor thickness. However, this means that the bottom level of the foundation is lowered, hence the upward hydraulic pressure increases as well. Determining the optimal dimensions is therefore an iterative procedure. The calculation is performed with help of a spreadsheet. The dimensions of the elements that result from this procedure are shown in Table 12.

Element	Thickness [m]
Roof	HE300B
Wall water-side	0,30
Wall polder-side	0,30
Floor	**1,00**
Side walls	0,40
Intermediate wall	0,30

Table 12: Thicknesses of concrete elements in order to prevent uplift for shallow foundation

The safety against uplift is verified with a calculation for above dimensions specifically. This is shown in Appendix B.1. As can be seen, safety against uplift is only achieved for a shallow foundation if a very (almost extreme) thick floor is constructed.

Shallow foundation - bearing capacity

For the uplift safety check, the situation with high water is governing. In previous section it was concluded that a very heavy structure is needed to prevent uplift. However, when the water level is low, the hydrostatic pressure under the structure is small, which results in large pressures on the soil exerted by the structure. For calculation of the bearing capacity, the Brinch Hansen method as prescribed by Eurocode 7 is used. A distinction is made between the undrained- and drained bearing capacity. The undrained situation is applicable for impermeable soil just after realization of a structure. After a while the overpressures in the pores disappear and the drained situation is applicable. In the case of the glass dike, the soil is preloaded by a dike, which is removed to construct the glass dike structure. Therefore, a situation somewhere between the drained and undrained situation is applicable just after construction. In this report, both the drained- and undrained bearing capacity are computed. For computation of the bearing capacity, the horizontal and vertical design loads and the moments around the center of the foundation that are induced by these design loads are input for the Brinch Hansen formula. The full calculation of design loads and the bearing capacity is shown in Appendix B.2. The result is shown here:

Undrained bearing capacity of the soil: R/A’ = 20,55 kN/m^2
Drained bearing capacity of the soil: R/A’ = 5,04 kN/m^2

In the expression ‘R/A’, R stands for the total bearing capacity and A’ for the effective area of the foundation. The maximum pressure on the soil exerted by the structure is obtained with help of the following formula:

$$\sigma'_{\max} = \frac{\sum V}{BL} + \frac{\sum M}{\frac{1}{6}LB^2} = \frac{6153{,}93}{9{,}9 \cdot 22} + \frac{1604{,}51}{\frac{1}{6} \cdot 22 \cdot 9{,}9^2} = 32{,}72 \text{ kN/m}^2$$

In which:

σ'_{max}	= Maximum effective soil pressure directly below the structure	[kN/m^2]
ΣV	= Design value of the vertical resultant load	[kN]
ΣM	= Design value of the resultant moment, induced by design loads	[kNm]
B	= Width of the structure	[m]
L	= Length of the structure	[m]

Since the upward hydraulic pressure is already taken into account in the computation of the resultant vertical load, the above obtained value of the maximum exerted soil pressure is in fact the effective soil pressure. As can be seen, the bearing capacity of the soil is far too low. Therefore, a pile foundation should be applied. The soil parameters obtained from Eurocode 7, belonging to the q_c values in the cone penetration tests, correspond to extremely weak soil. Performing soil investigations at the location of the glass dike more thoroughly may reveal better soil properties, but based on the results of this calculation, it is very likely that a shallow foundation will still be inappropriate.

Based on the design checks for uplift and bearing capacity, it is concluded that a pile foundation should be applied.

Pile foundation - tension

Since a shallow foundation appeared to be inappropriate for the soil conditions at the desired location of the glass dike, a pile foundation should be applied. This is again checked for uplift (tension) and bearing capacity (compression). Regarding the loads, it is expected that tension forces on the piles are governing for the pile design. Therefore, the pile foundation is first dimensioned for tension forces and then for compression forces. The design is an iterative procedure. A first estimate of the dimensions of the foundation is made, for which design checks are performed. If it appears that it is under-/over dimensioned, it is re-dimensioned and recalculated. This process is repeated until appropriate dimensions are obtained. A first pile scheme is shown in Figure 38.

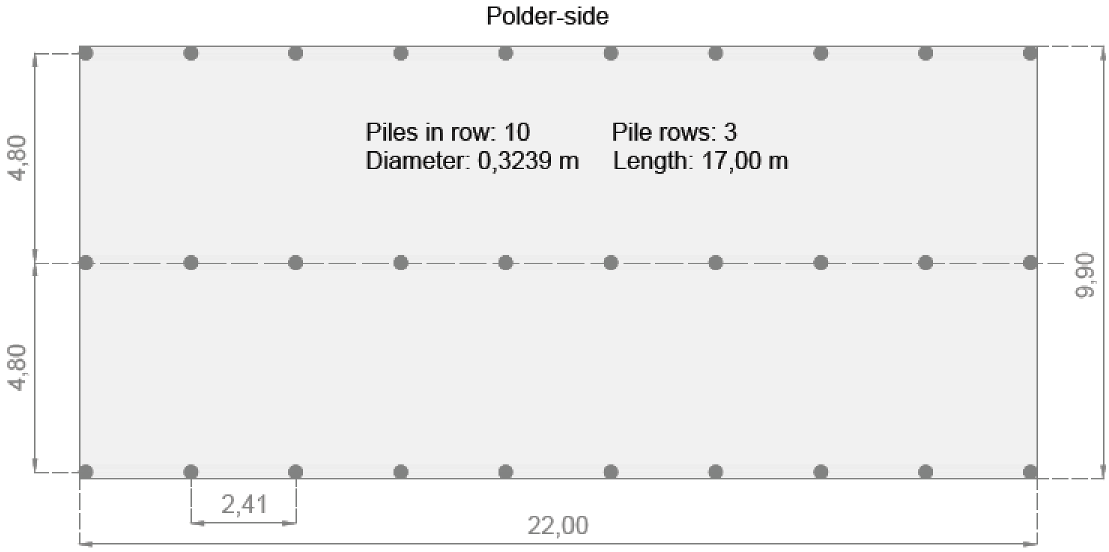


Figure 38: First estimate of dimensions of the pile foundation

The pile foundation has sufficient resistance against uplift if the pile which is subjected to the largest upward directed (tension) force does not fail. Three types of tension failure can be distinguished. One possible failure mechanism is that piles are pulled out of the soil because of insufficient friction between the shaft of the piles and the soil. Another possibility is that the shaft friction is large, but the pile is pulled out together with a clump of soil. In order to prevent this, the total weight of the clump should be larger than the upward directed force on the pile. Lastly, the pile itself can fail if the strength is too low. For correctly design piles, this is usually not the case. Therefore, only the following ultimate limit states are checked:

$F_{s;d} \leq F_{r;shaft;d}$
$F_{s;d} \leq F_{r;clumd;d}$

In which:

$F_{s;d}$	= Design value of the largest tension force that is acting on a single pile	[kN]
$F_{r;shaft;d}$	= Design value of the maximum shaft friction (for tension)	[kN]
$F_{r;clump;d}$	= Design value of the tension capacity according to the clump criterion	[kN]

The design values of the tension resistance of a single pile is calculated in Appendix B.3. The results are shown below:

$F_{r;shaft;d}$ = 46,47 kN
$F_{r;clump;d}$ = 570,84 kN

As can be seen, the tension resistance is determined by the shaft friction as it is much smaller than the tension resistance according to the clump criterion.

The situation with high water gives the largest upward hydraulic pressure, hence it is governing for the tension capacity check of piles. All downward directed loads are favourable and all upward directed loads are unfavourable. Besides vertical loads, moments induced by design loads will also induce pile forces in axial direction. The hydrostatic pressure acting on the waterside of the structure is dominant for the resulting moment. Therefore, this load together with the soil pressure on the waterside are unfavourable loads. The soil pressure on the polder side of the structure is small and not taken into account. This results in a more conservative design. The design loads are computed in Appendix B.3.

The resultant moment that is induced by the design loads causes tension forces on the piles at the waterside of the structure. Therefore, these piles are governing for the tension capacity check. The calculation of the maximum occurring tension force on a single pile is given in Appendix B.3. For the pile scheme shown in Figure 38, this results in the following tension force:

$F_{s;d} = 114{,}79 \text{ kN}$

Since this force is much larger than the maximum shaft friction, the tension capacity is not sufficient. Therefore, the dimensions of the pile foundation must be changed. The maximum tension force on piles can be reduced by increasing the number of piles. Another option is to increase the tension resistance by increasing the pile dimensions. Off course, a combination of both is also possible. Increasing the size of piles is not desired as heavier equipment is needed for construction. Increasing the number of piles will decrease the length of floor spans, thereby reducing internal stresses in the floor. Therefore, the number of piles is increased, without changing the dimensions of single piles. A new pile scheme is set up and shown in Figure 39.

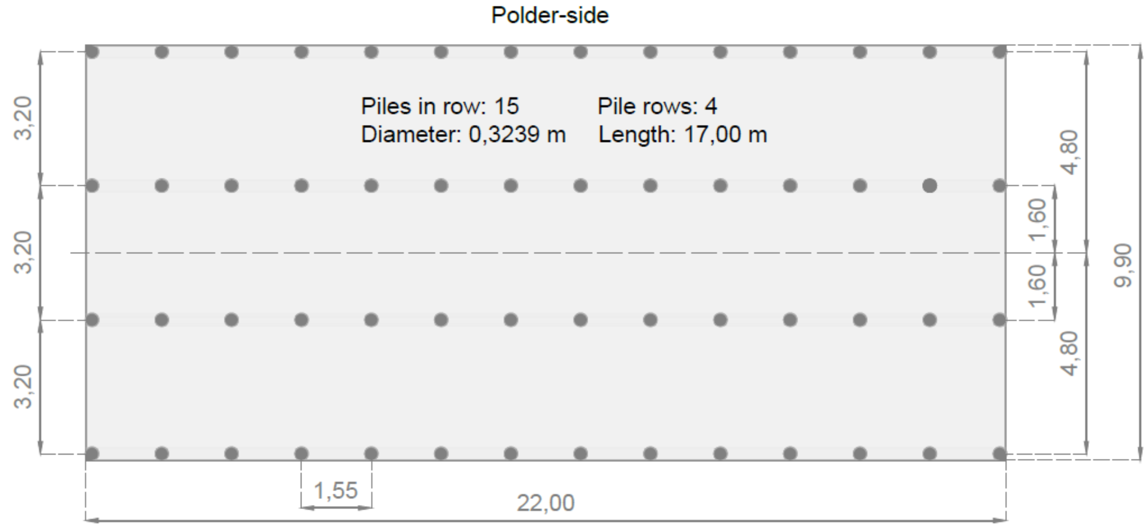


Figure 39: Second estimate of dimensions of the pile foundation

Again, the maximum occurring tension force on a pile at the waterside is calculated. This is also shown in Appendix B.3. Since the dimensions of single piles are unchanged, the tension resistance does not need to be recalculated.

Now, $F_{s;d} = 42{,}46 \text{ kN} < 46{,}47 \text{ kN} = F_{r;shaft;d}$

Hence the design criteria for tension resistance is satisfied.

Pile foundation – compression

The pile foundation has sufficient resistance against compression if the pile which is subjected to the largest downward directed axial (compression) force does not fail. The pile fails if the soil that is surrounding the pile is not able to take over the load. The compression capacity of piles is determined by the sum of shaft friction and tip resistance. However, the soil at the desired location of the glass dike consists entirely of weak and cohesive layers. Therefore, there is no tip resistance, but only shaft friction. Since the soil is preloaded by the (removed) dike, settlements are only expected to occur as a consequence of the positive shaft friction. Therefore, negative shaft friction is not taken into account. The pile foundation that was dimensioned to prevent uplift, is used as input for the pile compression design check. The pile scheme and pile dimensions are shown in Figure 39. The following design check is performed:

$F_{s;d} \leq F_{r;shaft;d}$

In which:

$F_{s;d}$	= Design value of the largest compression force that is acting on a single pile	[kN]
$F_{r;shaft;d}$	= Design value of the maximum shaft friction (for compression)	[kN]

It should be noted that the resistance that can be provided by the shaft friction is much larger for compression forces than for tension forces. The maximum shaft friction is calculated in Appendix B.4. The result is shown here:

$F_{r;shaft;d} = 166{,}43$ kN

The situation with low water gives the smallest upward hydraulic pressure. As a consequence, the downward directed force on the piles is larger. All downward directed loads are unfavourable and all upward directed loads are favourable. For the calculation of compression forces in the piles, a similar approach is used as for the calculation of tension forces (Appendix B.4). The basis of this approach is that pile forces in axial direction are induced by vertical loads and moments that are induced by all loads. For this, horizontal loads are also taken into account. These are unfavourable if directed to the polder-side of the structure. This is elaborated more thoroughly in Appendix B.4. The pile forces in axial direction are also derived in Appendix B.4. The result is shown below:

$F_{s;d} = 112{,}58$ kN

Now, $F_{s;d} = 112{,}58 \text{ kN} < 166{,}43 \text{ kN} = F_{r;shaft;d}$, hence the design criteria for compression resistance is satisfied.

Pile foundation: Horizontal loads

The foundation piles are schematized as pendulums, supported by springs that represent the soil. Since the soil is very weak, the spring constant is low, hence displacements need to be relative large in order to develop resistance against horizontal forces. Since displacements are unwanted, raking piles are applied. For the pile driving method chosen in section 6.4.2 (internal weight drop hammering), the maximum inclination is 1:5. Friction between the floor and soil is not taken into account as horizontal resistance.

To maximize the horizontal resistance of the foundation and distribute horizontal forces over the piles, as many as possible piles are placed under an angle. In order to guarantee the stability of the structure, the pile row at the side of the Noordvliet is placed inclined in opposite direction. A schematization of the inclined piles is given in Figure 40.

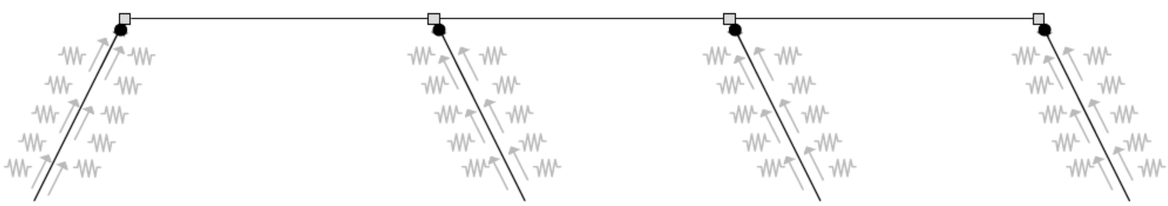


Figure 40: Schematization of raking piles

Applying raking piles has the advantage that piles can provide more resistance against horizontal forces, but at cost of the vertical resistance. However, the maximum horizontal load comes from the water level with a 1 in 100 exceedance probability per year. This water level also corresponds to the situation that compression forces on the piles are minimum. Furthermore, in the pile compression check, it was found that the maximum compression force (at low water) is much smaller the maximum admissible compression force. Therefore, it is not re-checked if the maximum admissible compression force is sufficient.

For the foundation with raking piles, part of the horizontal forces are transferred to the soil trough axial pile forces and part of the horizontal forces are transferred to the soil trough passive soil pressure perpendicular on the piles. The relative contributions are not exactly known and calculating this is rather extensive. Regarding the goal of this research, the pile foundation is not further elaborated.

6.5.2 Strength

Schematization of structural elements

In this section, strength checks are performed for the structural elements. These elements are the floor, wall at the waterside, wall at the polder side, side walls at the transitions with the dike, the intermediate wall and the roof. The strength of the glass elements is not assessed in this chapter.

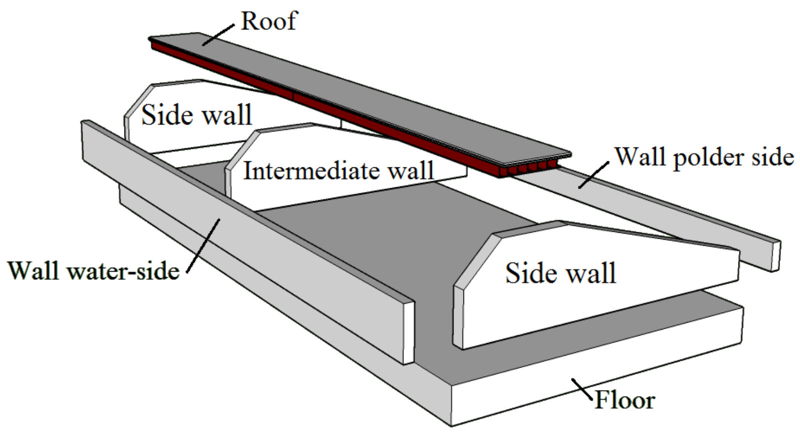


Figure 41: Overview of structural elements, excluding the water retaining glass elements

In section 6.1, some assumptions were made for the distribution of forces over the structural elements. A more detailed schematization of the behaviour of the structure is elaborated in this section. A schematization of the cross-section with structural elements is given in Figure 42.

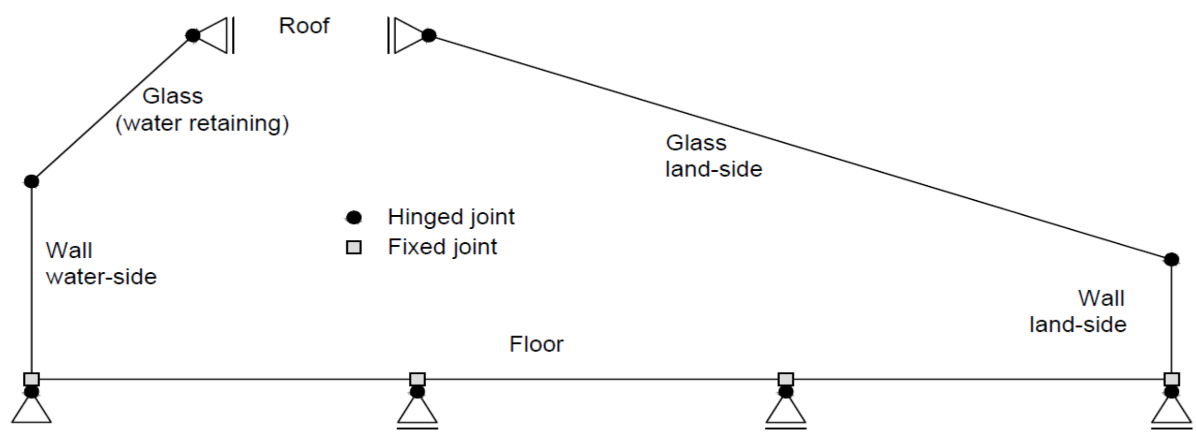


Figure 42: Schematization of structural elements in a cross-section of the glass dike

As can be seen, the roof is not included in the schematization. The reason for this is that the roof is fully supported by the side walls and intermediate wall, which are not in the cross-section but in a length section of the structure. The concrete walls and floor are connected to each other by fixed joints and can therefore be described as one u-shaped element. The piles are schematized as simple hinged supports. This is a conservative assumptions for the strength evaluation of the floor as bending moments are not transferred to the piles. The frame of the glass elements is connected with hinges and is able to move vertically relative to the roof. This way it is prevented that large stresses in the glass elements develop if the roof deforms. A schematization of the structural elements in a length section is given in Figure 43.

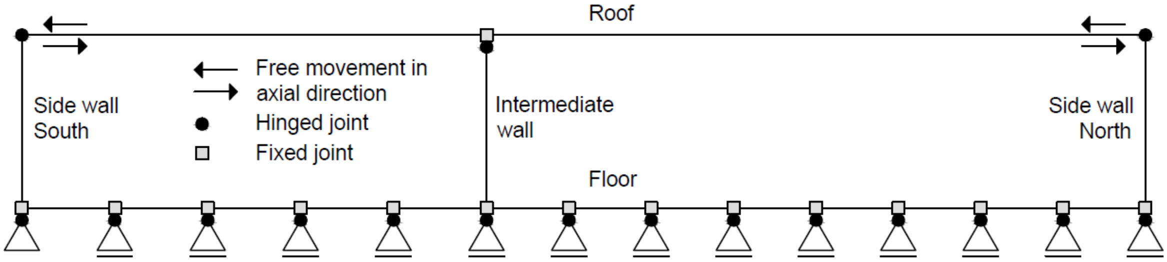


Figure 43: Schematization of structural elements in a length-section of the glass dike

The floor, side walls and intermediate wall are connected to each other with fixed joints. The roof is supported by the walls in vertical direction, but can move freely in horizontal direction. Rotations are also allowed, so no bending moments are transmitted between the roof and the walls.

Relevant strength checks

In section 6.2.2, the possible failure mechanisms of the structural elements were identified. These are shown in Table 13. Since the goal of this chapter is to obtain a first design, only a few design checks are performed. In this section, the most important strength checks that should be performed in this research are determined.

Structural element	Failure mechanism(s)
Glass elements (water retaining)	A, E, G
Wall water-side	A, B
Roof	A, B
Floor	A, B
Piles (if applied)	B, C, D
Side walls	A, B
Intermediate wall	C
Columns (if applied)	C, H

Table 13: Potential failure mechanisms of the structural elements

In which:
A = Bending
B = Shearing
C = Buckling
D = Tension
E = Impact
F = Dynamic fatigue
G = Static fatigue
H = Punch

Columns will only be applied if the roof is not able to carry the loads, without applying very thick roof beams. Therefore, it is relevant to perform a strength check for the roof. The strength of the piles is not assessed in this report. The reason for this is that the soil is very weak and many piles are needed to transfer the loads to the soil, which has the consequence that internal forces in individual piles are relative low. Also the strength of the intermediate wall is not checked in this report as it is not expected to be critical for the design. The glass dike consists of a number of concrete elements (floor and walls), which have a thickness of either 0,30 meter or 0,40 meter. These elements are all assessed by performing only one strength check for each thickness. In order to determine which elements are governing, the distribution of internal forces is investigated in Appendix B.5. In this analysis, it is concluded that strength checks should be performed for the wall at the waterside and the side walls at the transitions with the dike. If the strength of these elements is sufficient, the strength of the other concrete elements are also sufficient as they have the same thickness while internal forces are smaller. It should be noted that only the bending moment resistance of the elements is assessed. This is sufficient for the first design. The strength checks that are performed in this report are summarized in Table 14.

Structural element	Strength check	Material
Wall at the waterside	Bending strength	Concrete (0,30 m)
Side walls	Bending strength	Concrete (0,40 m)
Roof	Bending strength	Steel (HE300B)

Table 14: Strength checks performed in this report

Bending strength of concrete elements

Concrete elements are considered sufficient strong (in bending) if the ultimate bending strength is larger than the maximum bending moment. As stated in Table 14, bending strength checks are performed for the wall at the waterside and for the side walls at the transitions with the dike. Since both the dimensions and loads of the side walls are equal, only one strength check is performed for the side walls. The bending strength is sufficient if following design check is satisfied:

$$M_{E;d} \leq M_{R;d}$$

$M_{E;d}$	= Design value of the maximum bending moment	[kNm/m]
$M_{R;d}$	= Design value of the ultimate bending strength	[kNm/m]

Note that the design checks are performed for a meter length of the elements, hence the unit of the bending moment is kNm/m. The wall at the water-side is equally loaded over its length, hence there is no need to pick a specific location for the strength check. This is not the case for the side walls since both the height and loads vary over its length. The most unfavourable location on the wall is chosen. This is the location where the loads and the height of the wall are at maximum, which results in the largest bending moment. This is clarified in Figure 44.

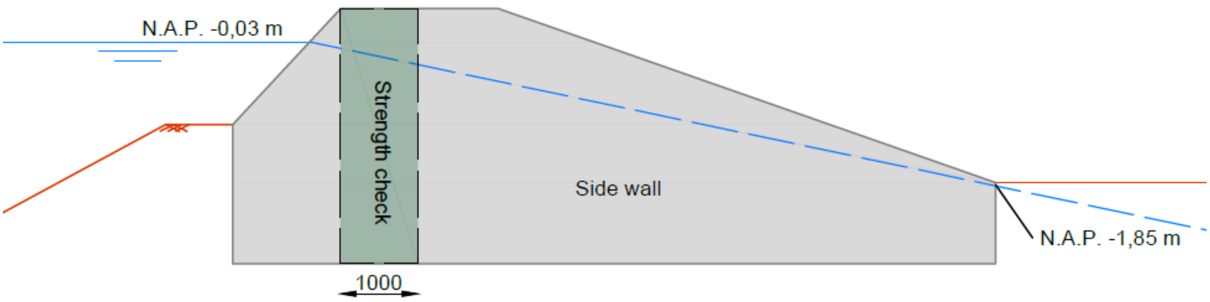


Figure 44: Location on the wall for which the bending strength check is performed

The bending strength checks for the side walls and the wall at the water-side are fully elaborated in Appendix B.5. The results are shown in Table 15.

Element	$M_{E;d}$ [kNm/m]	$M_{R;d}$ [kNm/m]	Design check
Wall water-side	47,20	79,86	Satisfied
Side walls	94,58	161,37	Satisfied

Table 15: Results of the bending strength checks for the wall at the water-side and the side walls

As can be seen, both elements have sufficient bending strength. As elaborated above, these elements are governing for the bending strength of all concrete elements. Therefore, the thickness of the floor and the wall at the polder side is also appropriate and sufficient bending strength can be obtained by applying the right amount of reinforcement.

Looking at the values of the bending moments and bending moment resistance of the elements, one could say that the structure is over-dimensioned. However, reducing the thickness of concrete elements will also reduce the self-weight of the structure. As a consequence, tension forces on the piles will be too large when the water level is high.

Bending of the roof
For the bending strength of the roof, the same design check as for concrete elements should be met:

$$M_{E;d} \leq M_{R;d}$$

In which:

$M_{E;d}$	= Design value of the bending moment	[kNm]
$M_{R;d}$	= Design value of the bending moment resistance	[kNm]

The roof is schematized as a continues beam on three simple supports, which represent the side walls and the intermediate wall. The loads on the roof are the self-weight of the steel beams and additional weight of materials that are used for insulation and to cover the beams. For the weight of the additional materials a conservative estimate is obtained by assuming that the roof is covered by a 0,05 meter thick concrete layer. Off course this is not true in reality. Furthermore, the user imposed load as specified in section 6.3.5 should be applied. The largest bending moment is obtained at the intermediate wall if the user-imposed (variable) load is applied on both spans. The full calculations of the bending moment and bending moment resistance is shown in Appendix B.6. It should be noted that the calculation is performed for single beams with a width of 0,30 meter. The result is shown below:

$$M_{E;d} = 64.04 \text{ kNm} < 202{,}65 \text{ kNm} = M_{R;d}$$

Hence, the roof has sufficient bending strength.

Additional to the bending check in 'Ultimate Limit State', it is checked if the displacement of the roof is small compared to the length of the largest span. This is a 'Serviceability Limit State' that is imposed by the Eurocode. For this, load factors of 1,00 may be used. The maximum displacement of the roof is automatically given when the type of beam is inserted in Matrixframe software and the bending moment is computed. The largest displacement is obtained if the variable load is only applied on the largest roof span. This gives:

$$\frac{w}{L} = \frac{0{,}011}{12.1} = \frac{1}{1100}$$

In which:

w	= Maximum displacement in the roof for the applied loads	[m]
L	= Length of the largest roof span	[m]

The relative displacement is much smaller than the requirement stated in the Eurocode (w/L < 1/500), hence also the displacement requirement in SLS is satisfied.

6.5.3 Transitions

For the connections of the existing dike profile with the glass dike structure, in section 6.2.2, the following failure mechanisms were mentioned to be relevant:

- Macro instability
- Outflanking
- Erosion of the outer slope

Macro instability

The macro stability of the existing dike profile is sufficient as was shown in a previous design study (Bardoel & Leemans, 2013). At the transitions however, the profile of the soil body in a cross-section is locally smaller in order to make a smooth connection between the dike and the glass dike structure. This is illustrated in Figure 36. Since the reduction of the profile is small, it is not likely that the macro stability becomes insufficient. In section 6.4.2, the possibility was described to apply outflanking screens, which are strong and stable enough to serve as soil retaining walls. This way, the potential loss in stability is compensated. This reasoning is not supported by a design check in this report.

Outflanking

Outflanking is eliminated by applying outflanking (seepage) screens. These screens are elaborated in section 6.4.2. Since outflanking screens with a minimum width of 1 meter are always sufficient in clay soil (TAW, Technische Rapport Zandmeevoerende Wellen, 1999), no further design checks are performed in this report. This also applies for piping, although this does not occur at the transitions with the existing dike, but under the structure.

Erosion of the outer slope

The existing dike has no erosion protection. Furthermore, no problems of erosion are reported for flood defences surrounding the Duifpolder. At the connections of hard structures with soil bodies however, erosion could be a problem. At the glass dike, this will mainly be caused by boats that sail past the structure as natural hydraulic loads are very mild in the Noordvliet. It should be checked if currents and waves that are induced by boats could cause erosion at the transitions of the structure and the soil body. If so, some form of protection should be applied. This is however not further elaborated in this research.

6.6 Robustness

In this research, robustness is defined as minimizing consequences in case failure occurs. The Eurocode has a similar definition of robustness and prescribes designers to provide alternative load paths in case some structural element has succumbed, with the final goal to prevent collapse of the total structure. For flood defences, additional consequences of a flooding should be taken into account as well. In the engineering design of the glass dike, robustness is provided by:

- Constructing the roof as one continues beam, thereby supporting the water retaining glass elements even if the intermediate wall has collapsed or removed by an inexperienced construction worker.
- Assuming no friction between the soil and the concrete bottom of the structure for the design calculation of the horizontal stability. If the pile foundation fails, it has residual sliding resistance because of this friction.
- Applying sheet piling as outflanking screens such that soil is retained, even if sliding occurs in part of the soil body.
- Design checks were performed for cross-sections of the structure, thereby not taking into account that structural elements are supported by other structural elements in 3D as well. For example, the wall at the waterside is supported by the side walls and intermediate wall as well, while in the design check it is assumed to be supported by the floor only. The second load path in this case are the side walls and intermediate wall.

For the design of the glass elements, robustness is a very important aspect to consider as well. This will be elaborated in chapter 8.

6.7 Constructability

In section 5.7.3, the following construction related requirements were given for the glass dike:

1. During construction, the flood defence must fulfill its water retaining function without reduction of safety against flooding (strict)
2. In case a pile foundation is applied, a construction method should be applied such that large pore pressures in the dike will not occur. Water board Delfland prescribes the use of foundation techniques which induce no or small pressure waves in the soil (strict)
3. The glass dike should be constructible (strict). Using standard materials, shapes and construction techniques will reduce risks and costs (preferred)

In this section, the construction methods and phases are described. By doing so, it is shown how the third requirement is fulfilled, meaning that the glass dike is constructible. Furthermore, it is shown that the structure is easy to construct and how risks are reduced. It is also shown how the first and second requirements, which are more specific, are satisfied.

Accessibility of the construction site

The desired location of the glass dike is in the Duifpolder. By road, this location is only accessible via a bridge located at the North of the polder. See Figure 45. The use of this bridge is restricted to transport with a maximum total weight of 15 Tons. This restriction could be a serious problem for delivery of building materials and equipment. However, there are a number of other structures located in the Duifpolder, for which building materials are delivered by this same bridge. Possibly, a permit can be retrieved to use the bridge for heavier transport. This possibility is not further investigated in this research. As alternative, materials and equipment can be delivered over water. In this case, they are delivered by road to a nearby quay, where they are loaded onto a boat or pontoons. There are a number of potential locations for this, which are not treated in this report. The construction method described in this section, is based on the case that delivery of heavy materials and equipment is not possible by road through the Duifpolder, hence it is required to work from the water.

Figure 45: Accessibility of the building site

Building pit

The first requirement for constructability of the glass dike is that the flood defence must fully fulfill its water retaining function during construction. Therefore, a temporary flood defence should be realized, before part of the existing dike can be removed. Using soil with natural slopes is not a good solution for this since this soil has to be excavated from the existing dike or should be delivered from an external location. The latter is economical not so attractive. A cofferdam constructed with sheet piles is a good solution. This requires little space and can be placed from the water. This is done with a tracked crane that is placed on pontoons and has a hydraulic sheet pile driver attached to it. Not all of the sheet piles can be placed from the water. Part of the sheet piles are placed with the crane standing on land. Special beams, in Dutch referred to as 'dragline' beams, are used to drive the crane from the pontoons onto the land. These are also used to distribute the load of the crane tracks evenly over the soil. The cofferdam should be stable and easily accessible. Figure 46 gives a proposal for the layout of the cofferdam. With this cofferdam, the requirement of a temporary flood defence is satisfied.

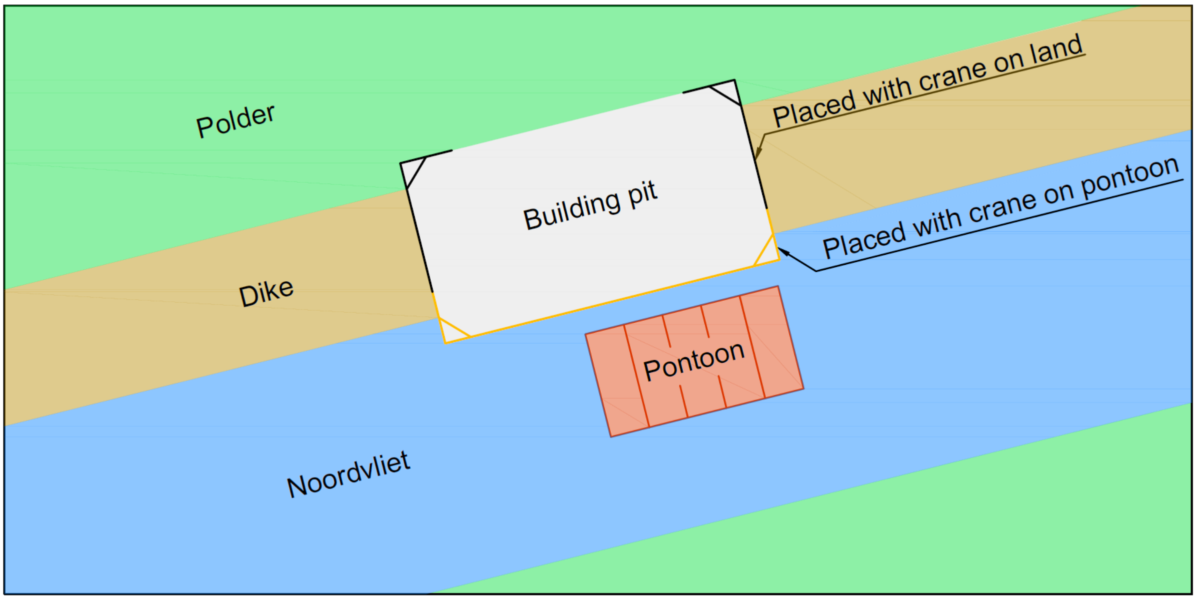


Figure 46: Installation of the sheet pile cofferdam

After placement of the cofferdam, water is pumped out and discharged into the Noordvliet. Then, the part of the existing dike that is situated inside the cofferdam is excavated. The soil below and around the dike profile is excavated to a depth of N.A.P. -3,20 meters. This is 0,10 meter below the bottom of the concrete floor. After excavation, a drainage system is installed. Since the soil is very impermeable, little swell water is expected, so using pumps with a relative small discharge capacity should be sufficient.

<u>Substructure</u>

The substructure consists of foundation piles and seepage screens. The steel tubular piles are placed with the internal weight drop hammering method. This method can be compared to normal weight drop hammering, but the weight is dropped inside the pile instead of externally on top of the pile. For this, a steel plate is welded to the bottom of the pile. The energy of the dropping weight is transferred to this plate. An advantage of this method is that energy is transferred to the soil more directly, thereby reducing pressure waves (and increased pore pressures) in the soil. For this, the second construction related requirement is satisfied. Similar to the tracked crane, pile driving equipment can be driven from the pontoons onto the dike with help of 'dragline' beams. After placement of the piles, they are cut off at a height of N.A.P. -3,15 meter. This is 0,05 meter below the bottom of the floor. Subsequently, reinforcement is placed and the piles are filled with concrete.

As alternative for delivery via roads through the Duifpolder, concrete can be deliver at the water pumping station across the Noordvliet. The roads leading to this water pumping station are also restricted in use, but in this case to a maximum axle load of 4,8 Tons. However, since the water pumping station is also made of concrete, it is assumed that concrete can be delivered at this location. A concrete pumping truck is used to pump the concrete to the other side of the water and to the building site. See Figure 47.

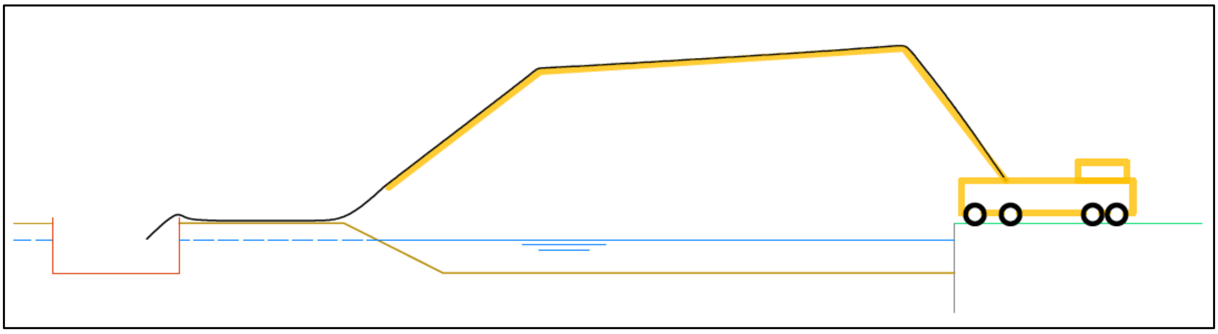


Figure 47: Pumping concrete from the water pumping station across the channel

After placement of the piles, the seepage screens are driven into the soil. The top of piping screens (below the structure) are driven to a height of N.A.P. -3,15 meter. This is 0,05 meter below the bottom of the concrete floor. For this, the same equipment is used as for placement of the cofferdam. The piles and seepage screens will both be casted in the concrete. At last, previous to construction of the upper structure, connections are made for utilities like drinking water, electricity, etcetera.

Upper structure

The upper structure consists mainly of concrete elements. The first element to be casted is the floor, but first, a work floor of approximately 0,05 meter thickness is poured with low quality concrete. Formwork of the floor is placed on this work floor and reinforcement is placed in the formwork. Subsequently, concrete is poured into the formwork. This can be delivered at the same location as explained for the substructure (Figure 47). After some time, the formwork of the floor can be removed and formwork for the walls is placed. Also the reinforcement is placed. To prevent large pressures on the inside of the formwork, concrete for the walls is poured in layers such that a layer is partially hardened before a next layer is poured onto it. Some walls are sloped on the topside. If these are poured in thin layers and a stiff type of concrete is used, there is no need to close the top of the formwork. After sufficient time for hardening of the concrete, the formwork of the walls is removed. Formworks of the floor and walls are made such that both piping- and seepage screens are casted in. All concrete elements have simple shapes. Consequently, placing the formwork is also ‘simple’, thereby reducing construction time and risk.

After sufficient time for hardening, the concrete is strong enough to be loaded. Then, the steel roof beams are placed. These are light enough to be delivered via roads through the Duifpolder and placed with the already present tracked crane. After placement of the roof beams, the frames of the glass elements are mounted to the roof and concrete. These frames should be connected in some way to the concrete elements. These connections can be casted in the concrete, or glued in drilled holes later on. The frames itself are prefabricated and pre-mounted as much as possible, such that they only have to be placed in-situ. This way, construction time and risks are reduced. After placement of the glass frames, the roof is isolated, the structure is made water tight and pavement is placed on top of it. To prevent damage, glass elements are placed as late as possible.

Repair of the dike profile and removal of the building pit

Now that the structure is completed (except for placement of glass elements), the soil levels in front and behind the glass dike are increased to the levels given in the design and the dike is connected to the structure. After this, glass elements are placed and covered to prevent damage. The next step is to slowly fill the space between the glass dike and cofferdam with water and test for water tightness. If the glass dike appears to be water tight, the cofferdam can be removed. To prevent leaks in the dike, the outflanking screens are extended until they reach outside the cofferdam. This is done just after removal of sheet piles from the cofferdam at the polder side of the outflanking screens. After extension of the outflanking screens, the remaining sheet piles of the cofferdam at the waterside are removed. Construction works are now completed and equipment is removed from the construction site.

6.8 Result

In Figure 48, a 3-dimensional impression of the engineering design of the glass dike is shown. The water in the Noordvliet is not shown in the figure, but for clarity, the Noordvliet is situated on the right side of the glass dike.

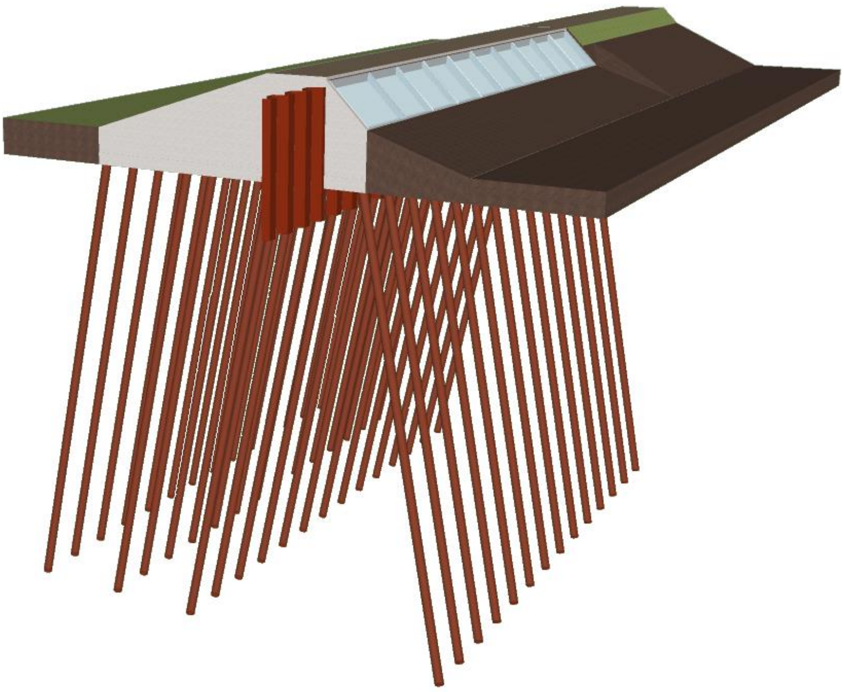


Figure 48: Impression of the engineering design *of the glass dike*

A cross-section of the engineering design is shown in
Figure *49*. Note that reinforcement bars at the connections between the piles and the floor are only drawn to indicate the type of connection. The same holds for the connections of the glass elements with the structure.

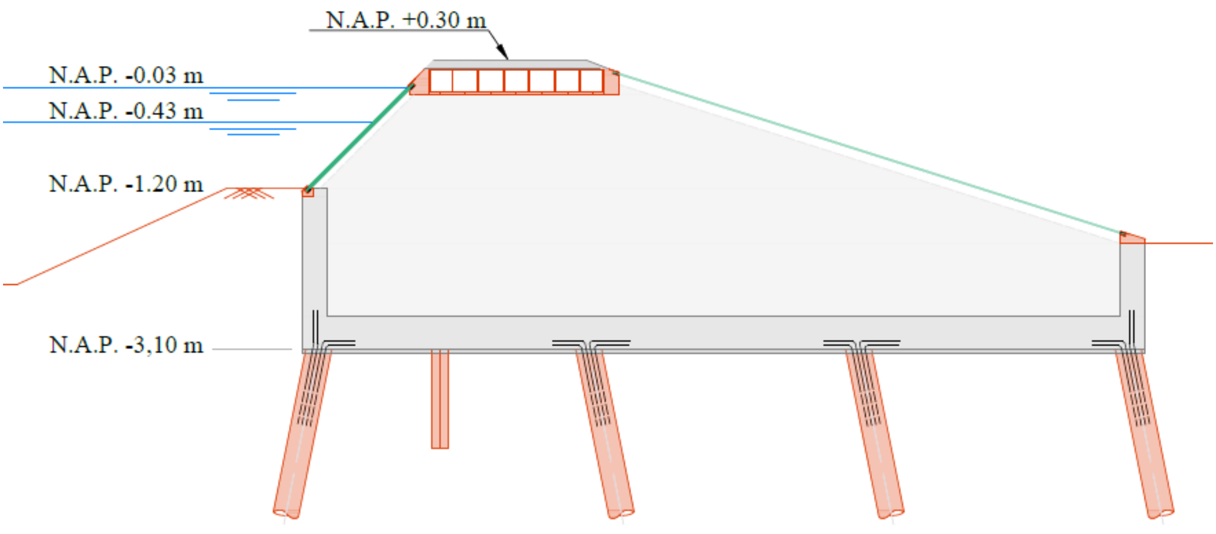


Figure 49: Cross-section of the engineering design of the glass dike

7. Probabilistic reliability evaluation of the glass dike

In section 5.6, the minimum reliability of the glass dike was determined by regarding both the Flood Safety Standard and the Eurocodes. It was concluded that for structural safety of the glass dike, there is an overlap in these requirements. With respect to the Flood Safety Standard, the probability of structural failure of the glass dike may not exceed 10^{-4} per year. The Eurocode on the other hand prescribes that failure probabilities for individual failure mechanisms may not exceed $1{,}3 \cdot 10^{-6}$. In chapter 6, design checks were performed with partial factors corresponding to Eurocode reliability class 2. In this chapter, it is verified whether this leads to a reliability of the total structure (system) that satisfies the reliability as prescribed by the Flood Safety Standard. Furthermore, the failure probability space for the glass elements is defined.

7.1 Approach

Determining the reliability of the glass dike in terms of a yearly failure probability is a very large task. In order to do this, the failure probability must be computed for every possible failure mechanism. Moreover, dependencies between these failure mechanisms must be known as well. This extensive procedure is avoided in this research. Alternatively, estimates are made for the probability of failure mechanisms. Since a lot of experience is available for water retaining structures in flood defences, it is possible to make realistic estimates. Since the probabilities of all failure mechanisms should lead to a total failure probability of the glass dike that satisfies the Flood Safety Standard, the failure space of the glass elements is automatically defined. This is equal to the failure space that is left over, after assigning probabilities to all failure mechanisms that are not related to the glass elements. Before probabilities of failure mechanisms are estimated, failure spaces are defined in section 7.3. This is done with help of LRK2011 (Guideline water retaining hydraulic structures in regional flood defences). This way, it can be checked if estimates of the failure probabilities are on the safe side.

7.2 Schematization of the systems reliability

In section 6.2.2, failure mechanisms that are relevant for the design of the glass dike were identified. Some choices that were made for the first design introduced new failure mechanisms, while other choices eliminated the possibility of certain failure mechanisms to occur. The failure mechanisms that should be taken into account for reliability evaluation are shown in Table 16.

Structural failure	
Insufficient strength	Loss of stability
Upper structure - Roof - Wall at the water side - Intermediate wall - Glass elements	*Vertical instability* - Large scale settlement - Uplift (pile pull out)
Substructure - Foundation piles - Floor	*Undermining* - Piping - Outflanking - Pipe leakage (e.g. drinking water)
Structural transition elements - Side walls - Outflanking screens	*Instability at the transitions* - Outflanking screens (loss of soil retaining function)
	Loss of horizontal stability - Insufficient horizontal resistance of the pile foundation

Table 16: Failure mechanisms that should be taken into account for reliability evaluation of the glass dike

A fault tree that includes all these failure mechanisms is shown in Appendix C. Similar to the subdivision of failure mechanisms in Table 16, the fault tree is split up into a part for structural failure due to insufficient strength of structural elements and a part for structural failure due to insufficient stability of the structure. As can be seen, only OR-ports are present (series system), implying that only one failure mechanism (base event) is needed for structural failure of the glass dike to occur. In total, 18 base events are specified. The probability of structural failure is in between the following boundaries:

$$\max(p_{f;i}) \leq p_f \leq 1-(1-p_{f;1})(1-p_{f;2})\ldots(1-p_{f;n}) \qquad \text{(series system)}$$

$$\text{for } i = 1..n \quad \text{and} \quad n = 18$$

In which:

p_f	= Probability of structural failure	[/year]
$p_{f;i}$	= Probability of occurrence of base event i	[/year]
n	= Total number of base events	

The expression above is only valid for series systems. For full dependency between the failure mechanisms, the probability of the top event (structural failure) is equal to the lower boundary, while for fully independent failure mechanisms, this probability of failure is equal to the upper boundary. Since the fault tree for structural failure of the glass dike is composed of OR-ports only, it can be treated as one large series system. Methods to narrow down the upper- and lower boundaries of the failure probability (e.g. Ditlevsen) exist, but information about the influence of parameters that play a role in individual failure mechanisms is needed. To obtain this information, probabilistic calculations need to be performed for each failure mechanism. This is not done in this thesis. Since dependencies between the base events are unknown, they are assumed to be fully independent. This gives the most conservative estimate of the probability of structural failure.

At this stage, it is not yet clear what failure mechanisms apply to the glass elements. Therefore, failure of the glass elements is schematized as one failure mechanism. In section 8.2, this is further specified.

7.3 Failure probability spaces

In this section, a distribution of failure probability spaces is shown. For this, the LRK2011 (Guideline for water retaining hydraulic structures in regional flood defences) is used. This document gives guidelines to achieve a reliability that satisfies the Flood Safety Standard. As determined in section 5.6, the probability of the top event, which is structural failure of the glass dike, may not be larger than $1{,}0 \cdot 10^{-4}$ per year. In section 7.2, it was assumed that failure mechanisms are fully independent. This conservative assumption is also applied in LRK2011. However, an example in the same guideline shows that failure probability spaces are only defined on the level of subsystems (for instance the upper part of the structure). Maximum failure probabilities of single structural elements are taken equal to the failure probability of the subsystem one level higher in the fault tree, which implies full dependency between failure mechanisms in the considered subsystem. This is clarified in Figure 50.

For the glass dike, an example of the distribution of failure spaces is shown in Figure 50. For the upper part of the structure, the failure probability space is further subdivided, in which a relatively large failure probability is assigned to the glass elements. Is seems odd to assume independency between subsystems on the one hand and to assume full dependency between failure mechanisms on the other hand, especially since the first mentioned is very conservative, while the latter is not. In practice, there is a large dependency between failure mechanisms due to hydraulic loads. For this reason, this approach leads to sufficient safety in the design of hydraulic structures in flood defences.

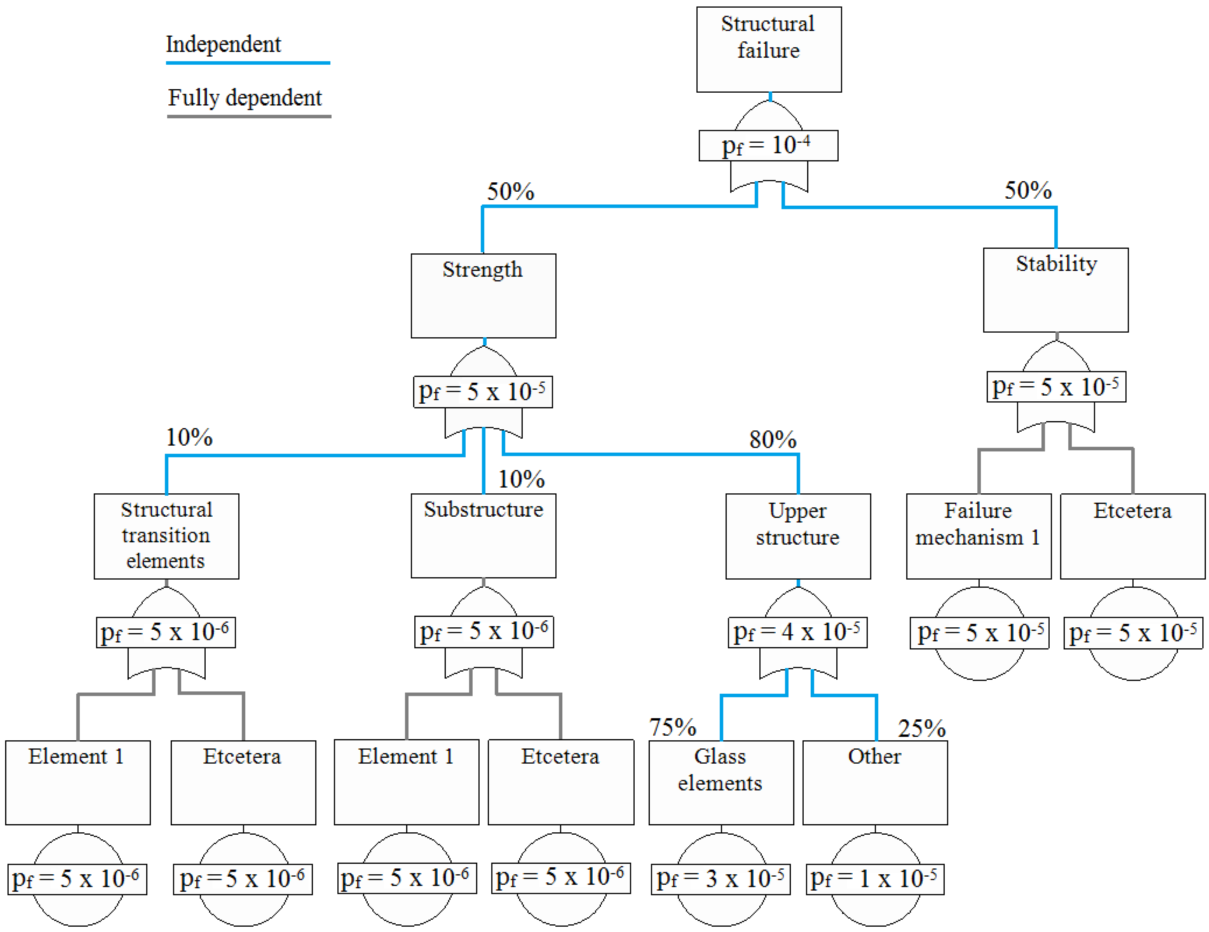


Figure 50: Distribution of failure spaces for structural failure of the glass dike

So based on LRK2011 (Guideline for water retaining hydraulic structures in regional flood defences), the distribution of failure probability spaces, as shown in Figure 50, will lead to a design of the glass dike that satisfies the Flood Safety Standard. However, in order to provide hard evidence that the structural reliability of the glass dike satisfies the Flood Safety Standard, exact dependencies between failure mechanisms and between subsystems should be computed when evaluating the reliability of the system (structure). Alternatively, full independency between all failure mechanisms could be assumed, which gives the most conservative estimate of the failure probability. The latter is done in next section.

7.4 Hypothetical failure probabilities

In order to compute the probability of structural failure of the glass dike, the fault tree as shown in appendix C is used. For most of the failure mechanisms, design checks were performed for the semi-probabilistic design. For these checks, partial factors from Eurocode reliability class 2 were applied. In Eurocode 0, it is stated that these factors are calibrated such, that a reliability index (β) of minimum 4,7 is achieved for single failure mechanisms and for a one year reference period. This corresponds to a maximum failure probability of $1{,}3 \cdot 10^{-6}$ per year. This value is used as input to estimate the probability of structural failure of the glass dike. In Table 17, estimates of failure probabilities are shown for all failure mechanisms.

Type of failure	Element / mechanism	Code	Hypothetical probability
Insufficient strength of structural elements	Roof	STR.1.A	$1{,}3 \cdot 10^{-6}$
	Wall at the water side	STR.1.B	$1{,}3 \cdot 10^{-6}$
	Intermediate wall	STR.1.C	$1{,}3 \cdot 10^{-6}$
	Glass elements	STR.1.D	Not yet determined
	Foundation piles	STR.2.A	$1{,}3 \cdot 10^{-6}$
	Floor	STR.2.B	$1{,}3 \cdot 10^{-6}$
	Side wall North	STR.3.A1	$1{,}3 \cdot 10^{-6}$
	Side wall South	STR.3.A2	$1{,}3 \cdot 10^{-6}$
	Outflanking screen North	STR.3.B1	$1{,}3 \cdot 10^{-6}$
	Outflanking screen South	STR.3.B1	$1{,}3 \cdot 10^{-6}$
Insufficient stability and undermining of the stability	Large scale settlement	STAB.1.A	$1{,}3 \cdot 10^{-6}$
	Uplift (pile pull out)	STAB.1.B	$1{,}3 \cdot 10^{-6}$
	Piping	STAB.2.A	$1 \cdot 10^{-5}$
	Outflanking	STAB.2.B	$1 \cdot 10^{-5}$
	Pipe leakage	STAB.2.C	$1 \cdot 10^{-5}$
	Outflanking screen North	STAB.3.A1	$1{,}3 \cdot 10^{-6}$
	Outflanking screen South	STAB.3.A2	$1{,}3 \cdot 10^{-6}$
	Horizontal stability	STAB.4	$1{,}3 \cdot 10^{-6}$

Table 17:Hypothetical probabilities for individual failure mechanisms

Design checks for piping, outflanking and pipe leakage are not prescribed by the Eurocodes. For these failure mechanisms, it is somewhat more difficult to estimate probabilities. To be on the safe side, the probabilities of these failure mechanisms are estimated to be $1 \cdot 10^{-5}$ per year, which is almost 8 times larger than failure probabilities for the strength of structural elements. Note that these values are hypothetical. The real probabilities should be determined with probabilistic calculations.

Apart from failure of the glass elements, an estimate of the probability has been made for each failure mechanism. With these estimates, hypothetical probabilities of intermediate events (for instance undermining of the stability) and the top event (structural failure of the glass dike) can be computed. Expressions to compute upper boundaries of the probabilities of intermediate events are given in Table 18.

Code	Intermediate events and corresponding expression to compute the failure probability
STR	Insufficient strength of a structural element in the glass dike $1-\big(1-p_f(STR.1)\big)\big(1-p_f(STR.2)\big)\big(1-p_f(STR.3)\big)$
STR.1	Insufficient strength of a structural element in the upper structure $1-\big(1-p_f(STR.1.A)\big)\big(1-p_f(STR.1.B)\big)\big(1-p_f(STR.1.C)\big)\big(1-p_f(STR.1.D)\big)$
STR.2	Insufficient strength of a structural element in the substructure $1-\big(1-p_f(STR.2.A)\big)\big(1-p_f(STR.2.B)\big)$
STR.3	Insufficient strength of a structural element at the transitions $1-\big(1-p_f(STR.3.A)\big)\big(1-p_f(STR.3.B)\big)$
STR.3.A	Insufficient strength of one of the side walls $1-\big(1-p_f(STR.3.A1)\big)\big(1-p_f(STR.3.A2)\big)$
STR.3.B	Insufficient strength of one of the outflanking screens $1-\big(1-p_f(STR.3.B1)\big)\big(1-p_f(STR.3.B2)\big)$

STAB	Insufficient stability of the glass dike
	$1-(1-p_f(\mathrm{STAB.1}))(1-p_f(\mathrm{STAB.2}))(1-p_f(\mathrm{STAB.3}))(1-p_f(\mathrm{STAB.4}))$
STAB.1	Insufficient vertical stability
	$1-(1-p_f(\mathrm{STAB.1.A}))(1-p_f(\mathrm{STAB.1.B}))$
STAB.2	Undermining of the stability
	$1-(1-p_f(\mathrm{STAB.2.A}))(1-p_f(\mathrm{STAB.2.B}))(1-p_f(\mathrm{STAB.2.C}))$
STAB.3	Instability at the transitions (=instability of the outflanking screens)
	$=p_f(\mathrm{STAB.3A})=1-(1-p_f(\mathrm{STAB.3.A1}))(1-p_f(\mathrm{STAB.3.A2}))$

Table 18: Expressions to compute upper boundaries of the probabilities of intermediate events

The upper boundary of the probability of the top event, which is structural failure of the glass dike, is computed with following expression:

$$p_f(\mathrm{STRUC})=1-(1-p_f(\mathrm{STR}))(1-p_f(\mathrm{STAB}))$$

This expression and the expressions shown in Table 18 are programmed in spreadsheets. If the failure probability of the glass elements is assumed to be zero, the result is a probability of structural failure of $4{,}82\cdot10^{-5}$ per year. This is less than half of the maximum probability for structural failure that follows from the Flood Safety Standard. From this, it can be concluded that the hypothetical failure probabilities lead to sufficient structural safety of the glass dike. Moreover, enough failure probability space is left for the glass elements. The latter is treated in next section.

7.5 Failure probability space for the glass elements

Based on hypothetical failure probabilities of individual failure mechanisms and expressions for the upper boundaries of the probabilities of intermediate events, the probability of structural failure of the glass dike can be computed. Regarding the Flood Safety Standard, this probability may not exceed $1{,}0\cdot10^{-4}$ per year. Insufficient strength of the glass elements is the only failure mechanism for which no (hypothetical) failure probability is determined yet. The expressions in Table 18 are programmed in a spreadsheet. With help of this spreadsheet, the failure probability space for the glass elements is defined. It appears that for a failure probability of the glass elements of $5{,}0\cdot10^{-5}$ per year, the probability of structural failure of the glass dike is just below $1{,}0\cdot10^{-4}$ per year, thereby satisfying the Flood Safety Standard.

However, the glass dike should also satisfy requirements from Eurocode reliability class 2. For this class, the probability of failure of individual structural elements may not exceed $1{,}3\cdot10^{-6}$ per year. The failure probability of glass elements should comply to this requirement as well.

Safety standard	Required reliability of glass elements
Eurocode reliability class 2	$1{,}3\cdot10^{-6}$/year
Flood Safety Standard	$5{,}0\cdot10^{-5}$/year

Table 19: Maximum failure probability of the glass elements

Since the most stringent reliability requirement is governing, the maximum failure probability of the glass elements is:

$$p_f(\text{glass elements})=p_f(\mathrm{STAB.1.D})\leq 1{,}3\cdot10^{-6}\ /\text{year}$$

For this failure probability of the glass elements the failure probability of the total structure is:

$$p_f(\text{glass dike})=4{,}95\cdot10^{-5}/\text{year}<\ p_{f;max}(\text{glass dike})=1{,}0\cdot10^{-4}/\text{year}$$

8. Probabilistic design and assessment of the glass elements

8.1 Approach

The most important design criteria for the glass elements is safety. In chapter 7, this is defined in terms of a yearly failure probability. The design of the glass elements should be such that the failure probability is lower than $1{,}3 \cdot 10^{-6}$ per year. For this, all possible failure mechanisms should be taken into account. Therefore, the susceptibility to failure mechanisms must be defined before a first design of the glass elements and the frames can be made. This is done in section 8.2. Subsequently a first design of the elements and the frames is given in section 8.3. Computing the probability of failure for a laminated glass panel for a certain failure mechanism is very complex. In this thesis, the probability of failure is computed only for the failure mechanism 'overloading'. The choice for this failure mechanism is described in section 8.2. Off course, the glass elements are designed to minimize failure probabilities (or prevent) for all glass related failure mechanisms. The probabilistic calculation itself is elaborated in section 8.4. In section 8.5.2, the sensitivity of the failure probability of overloading to variation of parameters like for instance the glass thickness is discussed. Partly based on these sensitivities and on section 8.2, in section 8.5.3, some measures are given which could effectively increase the reliability of the glass dike, with respect to the flood protection function.

8.2 Failure mechanisms

In this section, a short description is given of the failure mechanisms that should be taken into account in the design of the glass elements. These mechanisms were identified based on the literature study on structural glass (chapter 4) and on the loads given in section 6.3. A schematization of the failure mechanisms is given in the form of a fault tree. From all failure mechanism, one is chosen for computation of the corresponding probability of failure.

8.2.1 Schematization of failure mechanisms

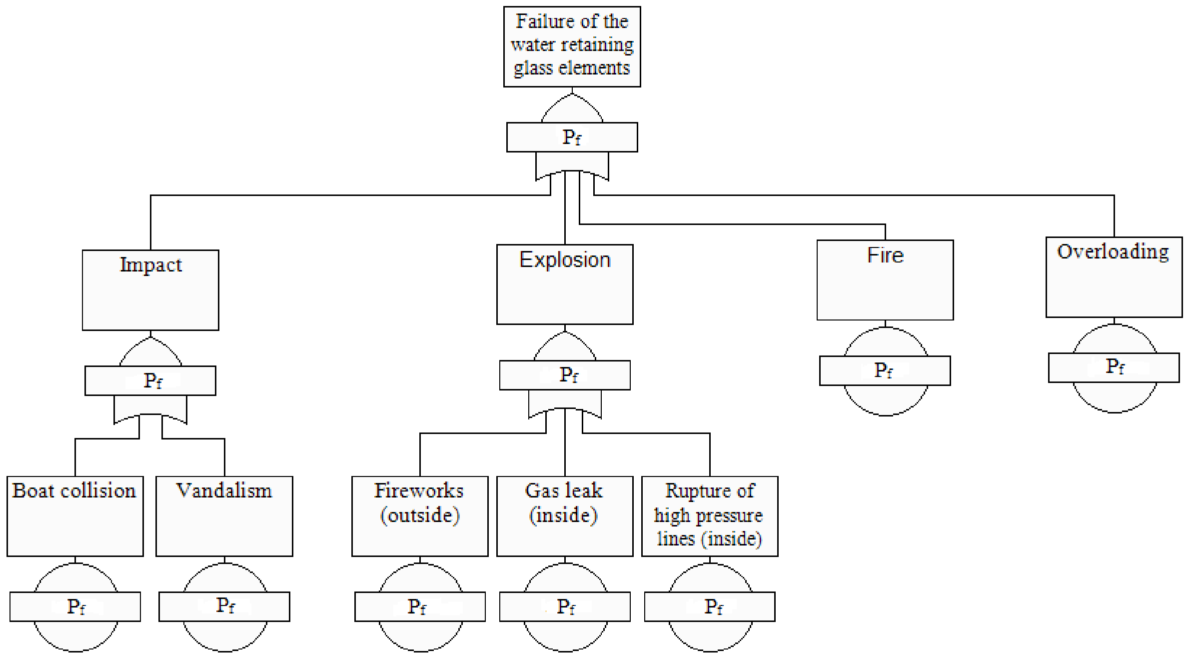


Figure 51:Fault tree for the water retaining glass elements

The most important types of failure of the glass elements are: Overloading, fire, explosions and impact. These are depicted in Figure 51. For impact, vandalism and boat collisions are very important to consider as both have a reasonable chance to occur. Since the load of a boat collision can be very large, this failure mechanism is elaborated more extensively. Explosions can have different causes. At the outside of the glass dike, the most likely cause is fireworks, which is in fact also some form of vandalism. On the inside, explosions are more likely to be caused by gas leakage or rupturing high pressure lines. Overloading governs hydraulic loads as described in section 6.3.

8.2.2 Overloading

For the glass dike, overloading occurs if hydraulic loads become larger than can be resisted by a glass element. The tensile stress at the glass surface opposite to the load will become larger than the practical tensile strength, with the result that the glass ply breaks. When the outer glass ply in the laminated element is broken, the tension stress on the surface of the glass ply next to the broken ply will be even larger. Since the spread in strength of glass is very large, this does not necessarily mean that this ply will break as well. The following load combinations are regarded:

- High water
- High water and waves
- Mean water and a layer of expanding ice

Since waves in the Noordvliet are very small and do not break, they can simply be neglected or added to the water level such that the hydrostatic pressure on the glass is increased. Since the force of an expanding layer of ice is much smaller than the hydrostatic pressure, this can be neglected as well.

Glass elements in the glass dike have to retain a layer of water to a depth of little more than a meter. Since aquaria with glass panels exist that retain water to a depth of several meters, it is very likely that applying thick glass panels will result in a design with a small failure probability for overloading. However, the consequences of a failure of the glass dike are larger than for an aquarium as a very large volume of water can flow into the Duifpolder. Furthermore, overloading can become more relevant if it is chosen to apply glass panels to a larger depth. Note that the force that results from hydrostatic pressure is proportional to the square of water depth.

8.2.3 Fire

As stated in section 4.5, the structural performance of glass is very poor when it comes to fire resistance. In ‘normal’ buildings, windows must stay in place in order to prevent ventilation, which stimulates the fire. For this, several safety standards prescribe a certain amount of time, in the order of a quarter to an hour, during which integrity of the window must be maintained. If a fire occurs inside the glass dike, the glass must remain in place during the whole fire as flooding of the Duifpolder must be prevented. On the other hand, water in the Noordvliet will cool down the glass, thereby increasing the time to failure. Furthermore, the glass elements will be much thicker than in ‘normal’ buildings. This will increase the time to failure as well. Fire performance of glass is usually determined with help of tests. Estimating the fire resistance of glass with help of models is difficult, let alone if the probability of failure should calculated.

So fire is an important but also difficult failure mechanism to consider for the safety of the glass dike. It is best to prevent fire as much as possible and reduce consequences, for instance by applying a sprinkler installation and providing an extra row of water retaining glass elements as backup. This is elaborated in section 8.5.3.

8.2.4 Boat collision

A very important threat to consider is a boat collision. The Noordvliet is a channel that is navigated a lot, especially during summer. Since the glass dike is located next to the intersection with the Bree, where boats have to make a turn of 90 degrees to enter the Noordvliet and where sailboats zig zag back and forth against the direction of the wind, it is not unthinkable that a boat hits one of the glass elements.

The stresses that occur in the glass elements during a boat collision can be obtained by numerical modelling of the impact phenomenon. This is based on complex theory and the result is influence by many factors. An overview of these factors is given below:

- Mass of the boat
- Velocity of the boat
- Angle between the sailing direction and the glass element
- Stiffness of the bow (elastic deformation)
- Strength of the bow (plastic deformation)
- Stiffness of the glass element as function of the impact point on the element
- Stiffness of the supports of the glass element
- Mass of the glass element
- Amount of damage in glass plies on the impact side
- Behaviour of the boat during collision
    - Sliding off
    - Yawing
- Added mass from water moving with the boat
- Damping due to escaping water between the glass and the boat

A very rough estimate of the contact force between the boat and the glass elements during the impact can be obtained with help of following equation (Eurocode 1):

$$F_{max} = V\sqrt{km}$$

In which:

F_{max}	= Maximum contact force during impact	[kN]
V	= Velocity of the boat perpendicular to the structure	[m/s]
k	= Stiffness of the structure	[kN/m]
m	= Mass of the boat	[kg]

With this equation, the impact problem is highly simplified. It is assumed that all kinetic energy of the boat is stored in the structure in elastic deformations. The boat itself is assumed to be infinite stiff and plastic deformations are not taken into account. Models exist, in which energy dissipation by hydraulic damping, and yawing of the boat are taken into account, but these models are more appropriate for calculation of impacts at low velocity, for example during berthing. Since ductile materials can exhibit plastic deformation, exceedance of the strength of structural elements does not necessarily lead to failure, which in the case of a flood defence is loss of the water retaining function. Then, the equation as shown above, can be useful to give a first impression of the impact force that must be dealt with. For brittle materials like glass, exceedance of the strength leads to fracture and in case of laminated glass, the only rest strength of the element is retrieved from the plastic interlayers. Therefore, if it is allowed that boats can hit the glass elements of the glass dike, a more detailed model if the impact problem is required. This can be done with numerical simulations.

For the glass dike, the hydrostatic pressure that acts on the glass elements is rather low. A boat collision will therefore dominate the design of the glass elements. It is therefore questionable if boat collisions should be allowed at all or if it should be prevented by some sort of protection. To answer this question, the force on the glass elements during impact is estimated based on the formula above. Additionally, the stiffness of the bow is taken into account as well. The computation of the impact force is elaborated in Appendix D. For the estimate, two situations are treated, which are a small boat sailing with relative high velocity and a large boat with smaller velocity. As can be seen in the formula, the impact force is proportional to the velocity of the boat, the root of the stiffness (bow and glass elements) and to the root of the mass of the boat. For simplicity and to be more conservative, it is assumed that boats do not yaw or slide off during the impact. The parameters of the glass can be determined quite accurately. The parameters of the boat that is hitting the structure however, shows large variation. An impression of the sensitivity of the impact force to these parameters is obtained by computing the impact force for a number of predefined parameters. This is shown in Table 20. Figure 52 gives an impression of the type of boats that are used for the calculation.

Maximum contact force due to boat collision [kN]					
Small and fast boat m=300 kg		Angle of approach [°]	Velocity of the boat [m/s]		
			3	6	9
Bow stiffness as a percentage of the stiffness of the glass element	1	30	33,46	66,93	100,39
		60	57,97	115,94	173,91
		90	66,96	133,92	200,88
	100	30	237,81	475,62	713,42
		60	411,96	823,92	1.235,88
		90	475,83	951,67	1.427,50
	∞	30	336,31	672,62	1.008,93
		60	582,60	1.165,19	1.747,79
		90	672,93	1.345,86	2.018,79
Large and slow boat m=2000 kg		Angle of approach [°]	Velocity of the boat [m/s]		
			1	3	5
Bow stiffness as a percentage of the stiffness of the glass element	1	30	28,80	86,40	144,01
		60	49,89	149,68	249,47
		90	57,63	172,89	288,15
	100	30	204,67	614,02	1.023,36
		60	354,56	1.063,67	1.772,79
		90	409,53	1.228,60	2.047,67
	∞	30	289,45	868,35	1.447,25
		60	501,42	1.504,26	2.507,10
		90	579,17	1.737,50	2.895,84

Table 20: Sensitivity analysis of parameters for the impact force of a boat collision

Figure 52: Impression of the type of boats used for the impact calculation

As can be seen, the impact force is mainly influenced by the velocity and mass of the boat. The angle of approach also plays a role, but to a lesser extent. If yawing and sliding off is taken into account, this will have a larger (positive) influence. The bow stiffness is taken into account as a percentage of the stiffness of the glass plate. For infinite stiffness, the estimate is equal to the result that is obtained by applying the formula prescribed by Eurocode 1, as shown above. Since the glass element is very stiff, a more realistic, but still conservative estimate of the impact force is obtained if the stiffness of the bow is assumed to be equal to the stiffness of the glass element. Some boats might be equipped with some sort of soft protection to prevent damage. An old car tyre for example. An estimate of the impact force that results from a collision with a boat hat is equipped with this protection, is obtained by assuming the stiffness of the boat to be equal to one percent of the stiffness of the glass elements. The calculation of the glass plate stiffness is shown in Appendix D. For this, a glass element with four plies of 20 mm thickness is used as this is the maximum reasonable thickness for realization. Furthermore, a larger thickness of the glass element leads to larger stiffness and hence, it gives a larger impact force.

As can be seen in Table 20, the force on the glass due to a boat collision can be very large. It should be noted that not all values in this table are realistic. The largest force given for example, results from the situation that a boat with a mass of 2000 kg sails with a velocity of 5 m/s (=18 km/h) perpendicular onto the glass dike. Since the channel is only 20 meters width, it is impossible that a boat with such a mass can hit the glass dike perpendicularly with such a large velocity. But even if a more reasonable situation is regarded, say an impact force of 1000 kN, the bending moment in the glass elements will be extremely large. The maximum tensile stress that results from this bending moment is calculated with SJ-Mepla, which is a commercial finite element program, especially developed to assess the strength of laminated glass. For the element with four plies of 20 mm thickness, this results in a tensile stress of approximately 440 N/mm2, which is much larger than the practical tensile strength of glass. It should be stressed that numerical analysis of the impact force might result in a (much) smaller impact force, since then, other types of energy dissipation (plastic deformation, sliding off of the boat, etcetera) can be taken into account. At last, it is noted that for this analysis, the impact is assumed to be in the centre of the glass element, while impacts on other locations on the plate can lead to larger stresses.

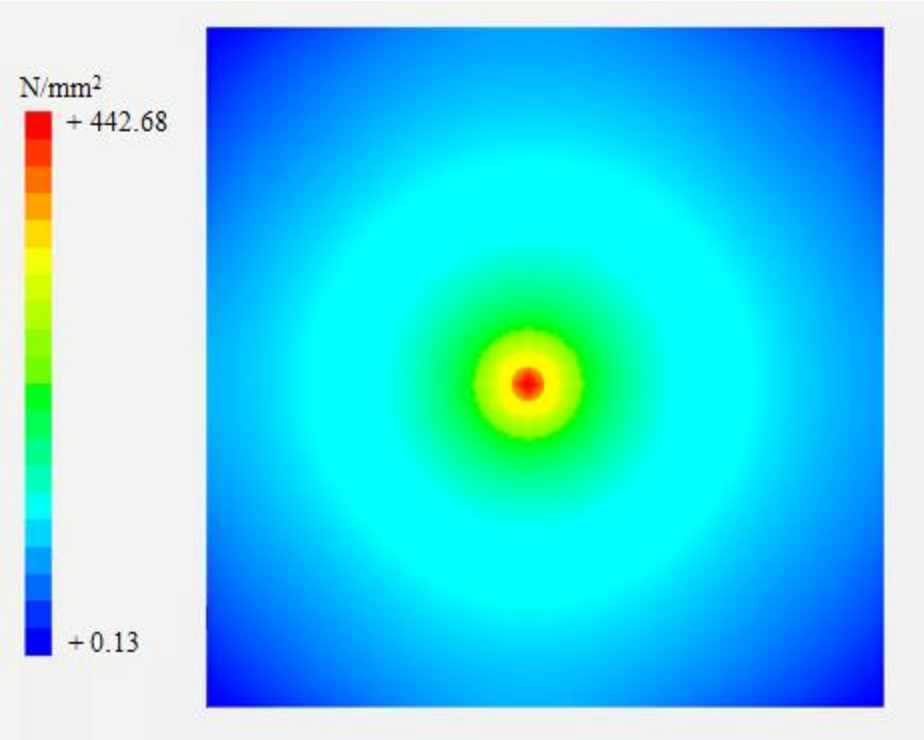


Figure 53: Tensile stresses on the surface of the glass element at the inner side of the glass dike

It is found that boat collisions can lead to extremely large stresses in the glass. Therefore, it is concluded that a protection against boat collisions is required. Another option is to allow the glass element to fail and apply a second row of glass elements that can take over the water retaining function. But then, there still is the practical problem that the damaged element needs to be replaced in some way.

8.2.5 Vandalism (impact)

It is not unthinkable that a person throws an object against the glass elements of the glass dike. Especially during summer when people consume a lot of alcohol and sail past the structure through the Noordvliet. Similar to a boat collision, this is an impact problem. As stated in the previous section, complex models are needed to estimate the resistance of laminated glass elements against this type of load. In contrast to a boat collision, vandalism cannot be prevented without losing the visual function of the glass elements. However, it is possible to simulate the impact with full scale tests. Impact test are prescribed by a number of norms. A lot of experience with these tests is gained in practice, mostly involving much thinner glass panels than will be applied for the glass dike. For glass panels with multiple layers of pre-stressed glass and stiff interlayers, the impact resistance can be very high. Due to the unpredictability of the event of vandalism and the large variety of objects than can be thrown in different angles on different locations on the glass, a probabilistic calculation would give a false indication of the real failure probability. Therefore, impact tests with some governing objects (e.g. a brick or steel ball) are preferred over probabilistic modelling.

8.2.6 Explosion

Possible causes of explosions are fireworks, ignition of leaked gas and rupture of high pressure pipelines. Gas leakage can be prevented by prohibiting utility of gas in the glass dike. Electricity could be a good alternative for cooking and heating. Explosions due to rupture of high pressure pipelines will not occur as these are not present at the structure. There are only drinking water lines exposed to `normal' pressure for which rupture is not followed by a pressure wave. Fireworks could be a realistic hazard. Regarding the location of the water retaining glass elements, exposure by accident is near impossible. Therefore, vandalism should be the criterion to evaluate the probability of occurrence. Probabilistic calculation of stresses induced by the explosion is practically impossible. However, data may be found which include blast tests on laminated glass. This can be used to estimate the probability of glass failure, given the blast. An exact calculation of the failure probability given the blast would give a false impression of accuracy as the probability of a blast with certain energy at a certain distance cannot be determined based on any scientific argumentation.

8.2.7 Choosing a failure mechanism for probabilistic reliability evaluation

From the failure mechanisms described above, the failure probability for only one of these mechanisms is computed with a probabilistic method. Computing the failure probability for a fire, impact or explosion is very complex and could be the subject of a large research project. Simplifying the problem, like is done in section 8.2.4 for a boat collision, leads to extreme large uncertainties, thereby giving a false indication of the failure probability. Therefore, it is better to prevent the load or to perform full scale tests to determine the resistance of the glass elements against these loads. It was concluded that boat collisions should be prevented and tests can be performed for impacts regarding vandalism. Fire and explosions should both be prevented, but since this is not entirely possible, measures to reduce consequences should be applied as well. Measures to increase the reliability of the glass dike are elaborated in section 8.5.3.

Overloading is caused by hydrostatic pressure, which is the only load that is permanently present and cannot be prevented. Furthermore, it is the only failure mechanism that can be modelled with reasonable accuracy given the available time for this research. Although overloading is not likely to occur for the glass dike, calculating the failure probability for this failure mechanism is needed in order to prove that the glass dike satisfies the Eurocode and Flood Safety Standard as no guidelines can be used for this. If it is chosen to retain water with glass elements to a larger depth, the failure mechanism will become more relevant as the stress in the glass is proportional to the square of water depth. Therefore, the probabilistic reliability calculation will be performed for the failure mechanism 'overloading'. This is elaborated in section 8.4. A summation of the failure mechanisms is given in Table 21:

Failure mechanism	Calculation method
Overloading	Probabilistic (section 8.4)
Fire	-
Impact – Boat collision	Deterministic (section 8.2.4)
Impact – Vandalism	-
Explosion	-

Table 21: Applied calculation methods for glass-related failure mechanisms

8.3 Design of the glass elements

In this section, the design of the glass elements and the related frames are described. This is input for the probabilistic design calculation, where the required thickness is determined. This section also gives a first estimate of the thickness. More important, the composition of the elements is chosen. The design of the frames is also very important input for the probabilistic calculation. The type of glass supports influences the distribution of stresses in the glass element. The design given in this section is based on qualitative descriptions of the structural performance of the glass, but also more practical reasons play a role.

8.3.1 Composition of the laminated glass plates

For the design of the glass elements, the requirements stated below must be satisfied. These requirements follow from the literature study on structural glass in chapter 4, the requirements for the entire glass dike as given in section 5.7, but also on the threats for the water retaining glass elements as described in section 8.2 and section 6.6, which describes the robustness of the glass dike.

1. Stress corrosion must be prevented
2. Thermal stress failure must be prevented
3. Failure due to nickel-sulphide inclusions must be prevented
4. Glass elements must be able to resist long term loading
5. Glass elements must be able to resist hard impacts (e.g. due to vandalism)
6. Damage due to hard impacts must be minimized
7. An optimum between failure strength and residual strength must be found

Stress corrosion can be prevented by applying a pre-stress on the glass surface that is higher than tensile stresses induced by the loading. For sufficient thick glass elements, both heat strengthened glass and fully tempered glass can be used for this. Since annealed glass is sensitive to both stress corrosion and thermal stresses, this type of glass is not an option. If heat strengthened or fully tempered glass is used, thermal stress failure will be prevented as well. Since nickel-sulphide inclusions are mainly a problem for fully tempered glass plates, heat strengthened glass is preferred over fully tempered glass. Heat strengthened glass has less tensile strength than fully tempered glass. However, in contrast to fully tempered glass, it has some residual compression strength after breaking. This is desirable with respect to robustness. If it is chosen to clamp the glass at the supports (section 8.3.2), all glass plies in the element will be subjected to both tensile- and compression stresses. Furthermore, heat strengthened glass has a reasonable good resistance against impact loads. So heat strengthened glass has the best structural performance and satisfies all requirements stated above. Therefore, it is concluded that this type of glass should be applied. The failure strength can be improved by machine cutting of the glass plies and subsequently grounding and polishing of the edges.

To obtain enough thickness, multiple glass plies need to be laminated to form an element. Lamination is also desired with respect to robustness. When one ply breaks, the other plies can take over the load. Even when all glass is broken, the foil that is used for bonding of the plies still has some residual strength. The type of interlayer material has a large influence on the structural performance of the laminated glass element. Ionoplast (SentryGlas) will be used as interlayer material. This is stiffer than 'conventional' PVB, it has a larger tensile strength and it is less sensitive to creep. Furthermore, it performs better at elevated temperatures (e.g. due to sunlight). Ionoplast provides good bonding, especially for short term loads. For short term loads, a laminated glass element will almost behave as if it were a monolithic glass plate with the same thickness. This gives good resistance against impact loads.

Robustness in structural glass can be achieved by laminating a number of glass plies. If a ply breaks, the other plies can take over the load. This effect is enhanced if the number of plies in an element is increased. For example: A glass panel of approximately 24 mm thick consisting of 4 plies with a thickness of 6 mm each is more robust than a glass panel with the same thickness of 24 mm that consists of 3 plies with a thickness of 8 mm each. If one ply breaks, less strength is lost for the panel with the larger amount of plies. Hence the residual strength is larger. For the first design of the glass elements, it is estimated that 4 layers of heat strengthened glass will lead to a sufficient low failure probability. If a thickness of 8 mm per ply is applied, the glass element will have a total thickness of 36,56 mm. If this is sufficient will be verified in section 8.5.1. The composition of the glass plies and interlayers in an element is shown in Figure 54.

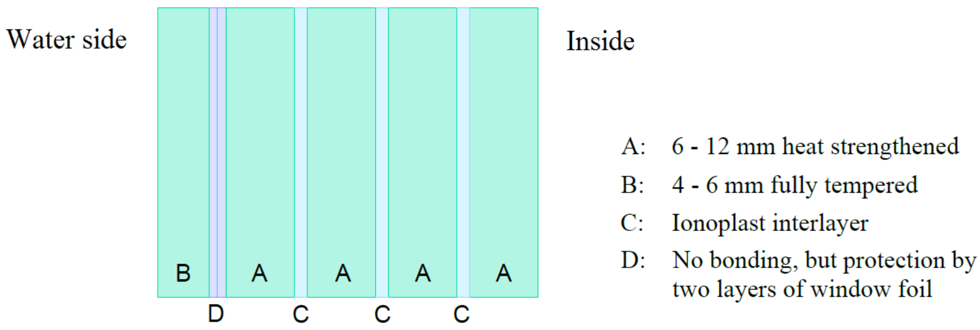


Figure 54: Composition of a glass element

In Figure 54 it can be seen that apart from the four layers of heat strengthened glass, one layer of fully tempered glass is added at the outside (side of the water). This layer serves as an offer layer, which is meant to protect the heat strengthened glass plies. It is not bonded to the other elements, so it can be replaced without the need to replace the entire element.

8.3.2 Design of the frames

For the design of the frames, the requirements stated below must be satisfied. These requirements follow from the literature study on structural glass in chapter 4, the requirements for the entire glass dike as given in section 5.7, but also on the threats for the water retaining glass elements as described in section 8.2 and section 6.6, which describes the robustness of the glass dike.

1. Glass elements must remain attached to the frames, even if fracture has occurred in all glass plies
2. Stress concentrations in the glass must be prevented
3. Stresses at the edges of the glass must be prevented
4. The connections between glass elements and the frames must be water tight
5. The connections between the frames and the glass dike structure must be water tight
6. It must be possible to replace a glass element
7. The roof must be able to move vertically (bending) without stressing the glass elements
8. Corrosion of the frames must be prevented.

Figure 55 and Figure 56 show example details of the glass supports respectively under water at the concrete wall and above water at the roof. As can be seen, the glass elements are clamped in the frames. To prevent damage at the edges, this is done in a controlled environment before delivery on the construction site. Between the u-shaped frame, neoprene is used to prevent stress concentrations in the glass and to provide water tightness. At each side of the glass, two strips of neoprene are placed in order to maximize water tightness. This is further enhanced by adding silicone sealant on the first neoprene strip. By clamping the glass element, it will remain in the frames, even if fracture has occurred in all glass plies. If the u-shaped frames are sufficient deep (approximately > 0,05 meter), large stresses at the edges are prevented. All four sides of the glass element will be clamped in the frame. This way, requirements 1 to 4 are fulfilled.

The connections between the frames and the structure must be water tight as well. Figure 55 shows how this is done for the part of the frame that is supported by the concrete wall. Between the concrete and the steel, water tightness is provided by casting in the steel. Between steel parts, water tightness is provided by welding them together along the entire length. The u-shaped frame is clamped against two strips of neoprene with help of a number of bolts. This way, the glass elements can be detached from the structure. Stainless steel type AISI316TI could be used to produce the glass frames and the supports. This steel type is corrosion proof, even when welded. With this connection, requirements 5, 6 and 8 are satisfied for the glass supports under water.

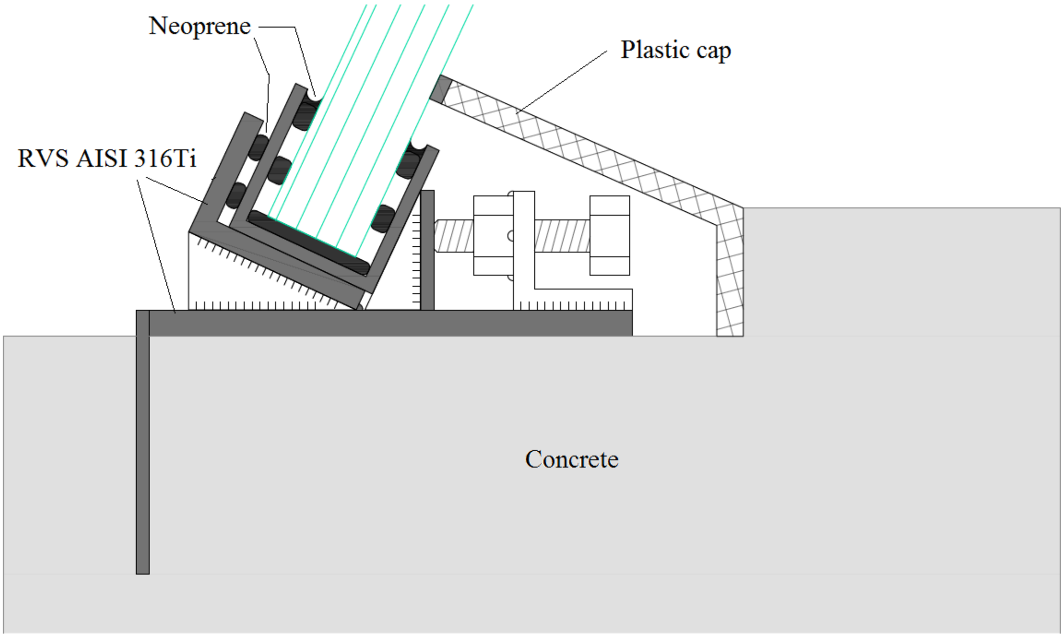


Figure 55: Example detail of the glass frames at the connection with the concrete wall

In contrast to the support at the concrete wall, the glass support at the roof is permanently above the waterline. Therefore, water tightness between the u-shaped frame and the structure is less important. Off course, rainwater and possibly splashes of water from the Noordvliet may not enter the glass dike. This is prevented with help of a plastic cover. The u-shaped frame that clamps the glass element is placed in a u-shaped beam that is bolted to the steel roof. The bolts are placed in elongated holes. Then, vertical movement between the roof and the glass supports is possible by not fully tightening the screw-nut. This way, requirement 7 is satisfied as well.

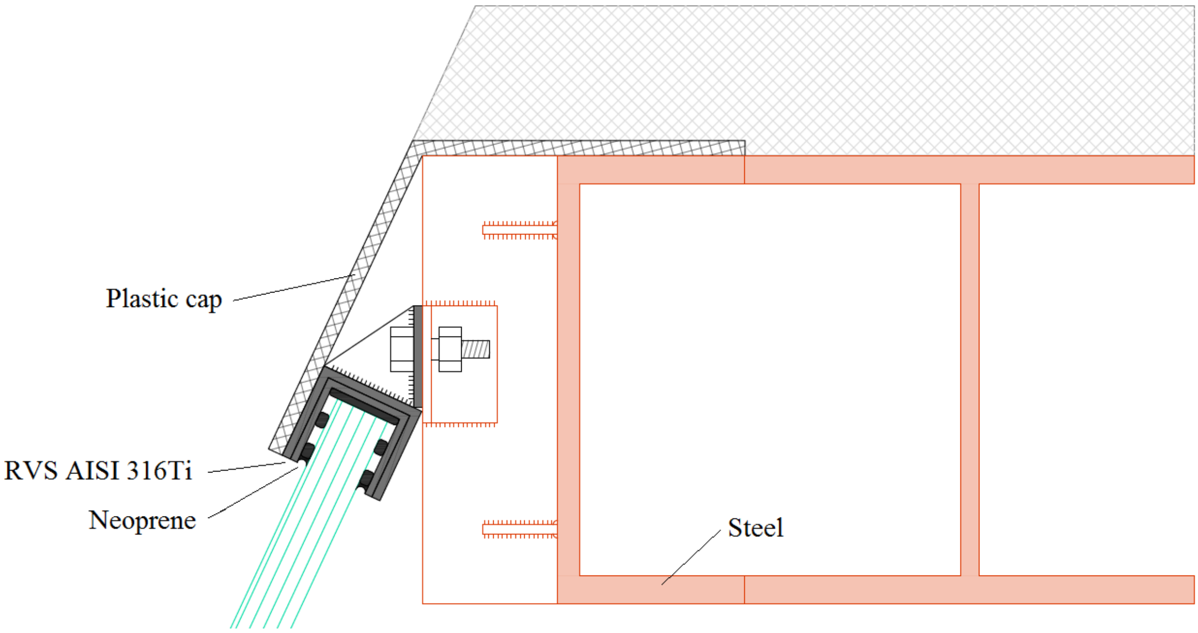


Figure 56: Example detail of the glass frames at the connection with the roof

Note that vertical supports between two glass elements must satisfy above stated requirements as well. This is not elaborated in this report. 3D modelling of the supports may reveal conflicts with the details as shown in above figures.

8.4 Probabilistic model for overloading of the glass elements

8.4.1 Simplified physical model of a glass element loaded by hydrostatic pressure

As described in section 8.3, the glass elements will be supported at all four sides. Each side will be clamped in the frame. The glass elements should thus be schematized as plates loaded by hydrostatic pressure and clamped at all four sides. In this thesis, the glass elements will be schematized as if they are only clamped in the frame at the connections with the concrete wall and at the roof. Hence, the supports at the vertical sides of the elements are disregarded. This simplification has been made for the following reasons:

- The width of a glass element is not yet determined. From an aesthetic point of view, it could be chosen to apply very wide elements. For a larger width/height ratio, the behaviour of the glass elements will become more like that of a beam rather than a plate.
- In section 8.4.2, the probabilistic model for the strength of glass elements is elaborated. It will be explained that glass plies are subdivided in small rectangles. A separate limit state function will be assigned to every rectangle. Modelling the glass elements as plates that are supported on all sides will give a stress field that varies over both height and width. As a consequence, a very large amount of different limit state functions is needed for the probabilistic model of a glass element. This is not desired. This argument will be easier to understand after reading section 8.4.2 and 8.4.3.
- Since tensile stresses in the glass are larger for a plate that is supported at two sides, this is a conservative approach.

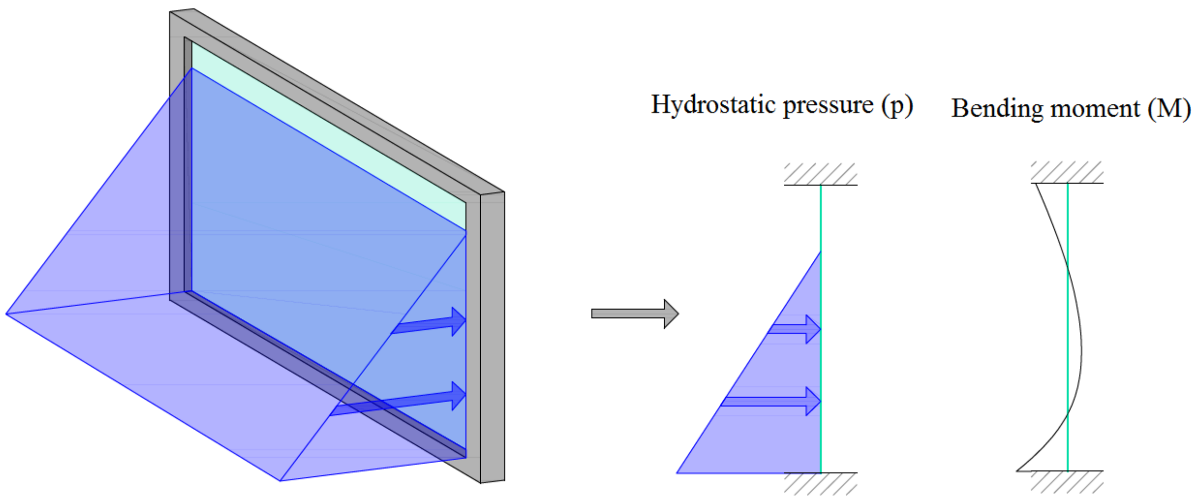


Figure 57: Simplified model for overloading of a glass element and bending moment diagram

Note that the glass element is fixed (clamped) in the frame. The connection between the frame and the structure is not schematized with fixed supports.

8.4.2 Probabilistic model for the strength of a glass element

Usually, when a probabilistic calculation is performed for a structural element in a hydraulic structure, the strength is only checked at the point in the element where the stress is maximum. For ductile materials like steel, which show very little spread in strength over the length and width of the element, this method gives accurate results. For a brittle material like glass however, the strength is determined by potential damages at the surface. The location of such a damage is random. Therefore, fracture in a glass ply can be initiated at a point in the glass where the tensile stress is much lower than the tensile stress at the point where the bending moment is maximum. Moreover, for larger glass plies it is more likely that some critical damage is present. As a result, the average strength of a glass ply decreases with increasing size. Therefore, statistical descriptors for the strength of glass cannot be used without taking into account this size- and location effect. In section 4.3, the results of strength tests that were performed for different types of glass and the factors that influence this strength are discussed (Veer, Louter, & Bos, 2009). Statistical descriptors that result from these tests are used for the probabilistic calculation of the glass elements. Below, it is explained how correlations between strength, location and size are taken into account.

From test specimen to glass ply

The size effect (decreasing average strength with increasing surface area) is solved by subdividing glass plies in small rectangles with a size that fits the specimens that were used for the strength tests. These were 1,0 meter long, 0,1 meter width and 0,01 meter thick. Each rectangular section is assigned a separate stochastic variable for the strength. A four point bending machine was used for the tests. Figure 58 gives an illustration of the bending moment diagram that corresponds to an element loaded in four point bending.

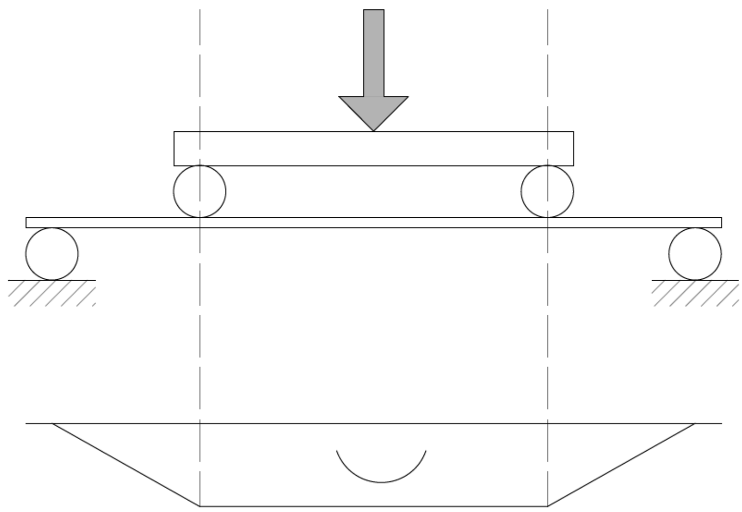


Figure 58: Four point bending test and corresponding bending moment diagram

The size of rectangular sections in a glass ply are not exactly equal to the size of test specimens. Only the part of the specimen that is fully stressed is taken into account, which covers half of the length. Since stresses in this part of the specimen are constant, all critical damages at the surface may lead to failure, thereby excluding the correlation between failure stress and the location(s) of critical damage(s). Thus, glass plies should be subdivided in rectangles with a length of 0,5 meter and a width of 0,1 meter. The length of a glass element is approximately 1,5 meter. Then, it makes sense to split up this length in three parts of 0,5 meter. Since the bending moment as shown in Figure 57 has a maximum at three locations, this is very convenient as each rectangle covers one of these maxima. For the simplified physical model as described in section 8.4.1, stresses at the glass surface are constant in horizontal direction. Therefore, the width of the glass plies can be split up in parts of 0,1 meter without consequences. For glass elements with a width of 2,0 meters, this is illustrated in Figure 59.

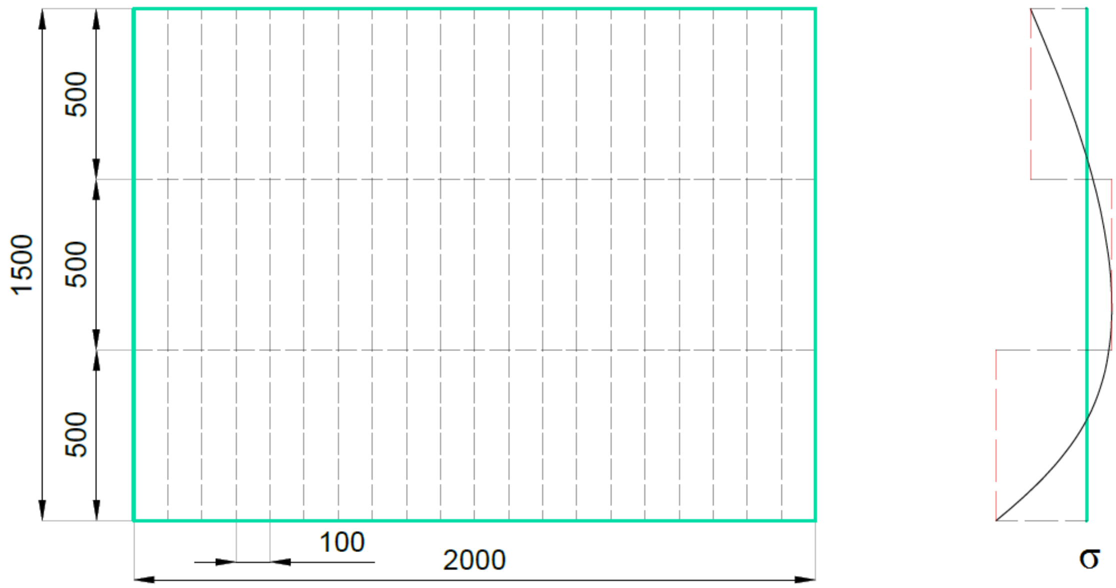


Figure 59: Subdivision of rectangles in a glass ply, each assigned a separate stochastic strength variable

For every rectangular section in a glass ply, the maximum stress in that particular section is used as input for a probabilistic calculation. However, statistical descriptors for the strength of glass are based on tests with specimens that were subjected to a stress that is constant over the surface of the specimen, while the maximum stress in a rectangular glass section in the glass dike is only present at a small part of the section. This is illustrated in the right part of Figure 59. As a result, the failure probability is overestimated.

In summary, the approach as described above is conservative for the following reasons:

- Glass plies are subdivided in rectangular sections that are smaller than the test specimens that were used to obtain statistical descriptors of the strength.
- The strength of glass is largely correlated to the quality of the edges. Statistical descriptors of the strength were derived from tests in which specimens were used with a large relative edge length (ratio of edge length over surface area). This relative edge length is much smaller for the glass elements in the glass dike.
- Stresses in a rectangular glass section are assumed to be constant at every point in the rectangle and equal to the maximum stress in the real stress distribution.

It should be noted that only tensile stresses lead to fracture. A negative value for the stress indicates compression. However, compression on one side of a glass panel implies tension with an equal absolute value on the other side of the panel.

Failure definition

For the glass dike, laminated glass panels will be applied. Tension stresses induced by bending moments will be maximum at the surfaces of the panels. If a ply is fractured, it still has some residual strength in the parts that are loaded in compression, but all tension strength is lost. For simplicity of the probabilistic model, it is assumed that fractured plies do not contribute to the resistance of the laminated element at all. Moreover, the offer layer as described in section 8.3.1, which is not bonded to the other plies, is not taken into account as load bearing ply. If fracture has occurred in one of the outer plies, stresses caused by the bending moment are redistributed over the remaining intact plies. This is illustrated in Figure 60.

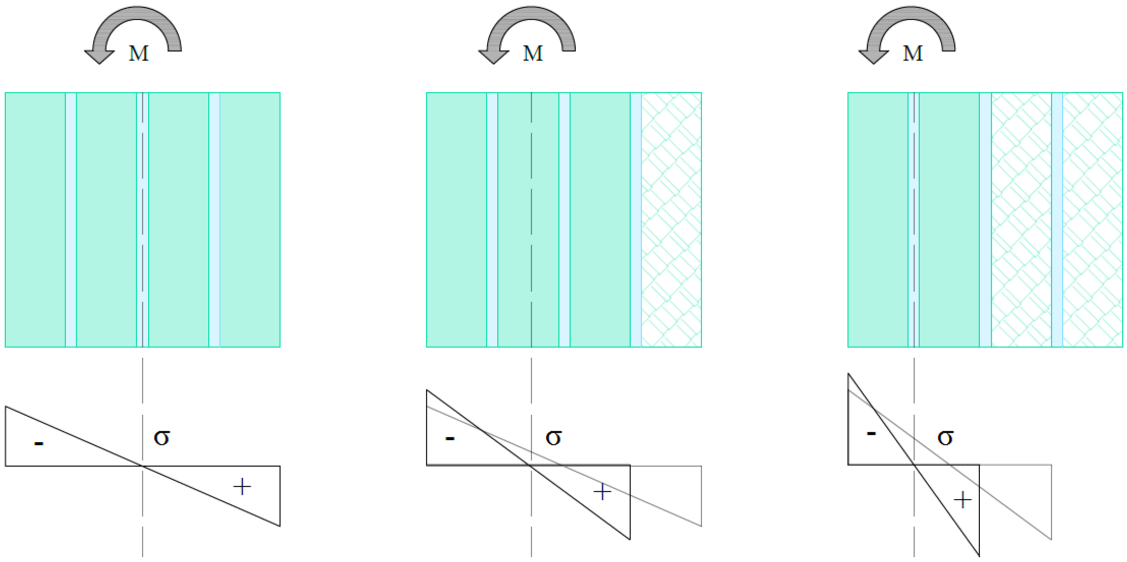


Figure 60: Stress distributions in damaged laminated glass panels

Note that in Figure 60, the effect of interlayers on the stress distribution is not taken into account. This should be taken into account by assigning different stiffness parameters to these interlayers, which results in a discontinuous stress distribution. A finite element method can be used to compute this. For probabilistic calculations however, this leads to very complex models that require a lot of computational power. This is not done in this research. Alternatively, the effect of the interlayers is taken into account by means of the equivalent thickness as was explained in section 4.2.

As can be seen in Figure 60, tension and compression stresses become larger if a glass ply is fractured. This stress is inversely proportional to the square of the thickness of the panel. So if half of the glass plies are broken, the maximum tension stress is quadrupled. Since the spread in glass strength is very large, fracture of one glass ply does not necessarily mean failure of the element. In other words, if a glass ply breaks due to a tensile stress that is larger than the strength of that particular ply, the ply next to it may stay intact, even if it is subjected to a tensile stress that is much larger than the tensile stress that caused fracture of the first ply. Glass panels in the glass dike will consist of four plies. If two of these plies are fractured, the stress increase will be so large that fracture of the remaining plies is very likely. Therefore, it is assumed that failure of a glass element has occurred if two glass plies are fractured.

In next section, it is explained how the considerations above are implemented in the probabilistic model for overloading of the glass elements.

8.4.3 Probabilistic model for overloading of glass elements

In Figure 61, a fault tree for overloading of glass elements is shown. The top event is failure due to overloading of one or more glass elements. As can be seen, the base events that can lead to failure are:

- Fracture of an outer glass ply (ply 1) at the submerged support
- Fracture of an outer glass ply (ply 1) in the field of the ply
- Fracture of an outer glass ply (ply 1) at the support at the roof

In combination with one of the subsequent events:

- Fracture of an glass ply next to the outer ply (ply 2) at the submerged support
- Fracture of an glass ply next to the outer ply (ply 2) in the field of the ply
- Fracture of an glass ply next to the outer ply (ply 2) at the support at the roof

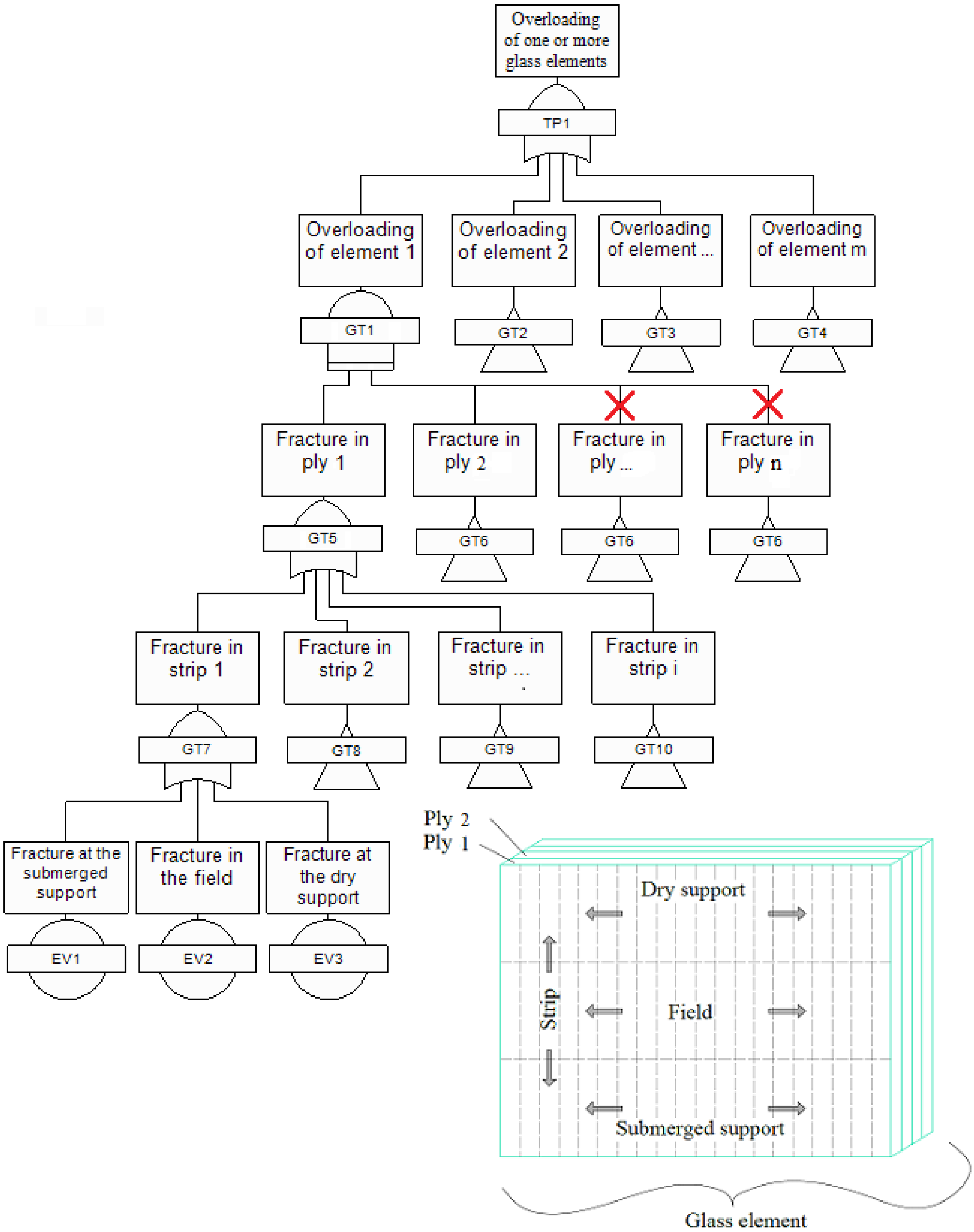


Figure 61: Fault tree for overloading of glass elements

Each base event is assigned its own limit state function. For glass elements of 2 meters width, the glass dike will be equipped with 11 elements. Each ply is subdivided into 60 rectangular sections, each with its own stochastic variable for the strength. Since failure has occurred if fracture occurs in more than one ply in an element, the total number of base events in the system is in between 660 (11 x 60) and 1320 (2 x 11 x 60). The failure probability of the top event, overloading of one or more glass elements, can be computed by computing the failure probabilities for the 6 base events only once. The failure probability of intermediate events and the top event can be estimated by means of upper and lower boundaries, corresponding to events that are respectively independent and fully dependent. For systems with a large number of events, as is the case for the glass elements, this method is not very accurate. The upper and lower bounds can be narrowed down by applying the method of Ditlevsen. This method takes into account correlations between the events by means of correlation coefficients. An even better estimate can be obtained by applying the method of Hohenbichler and Rackwitz. However, this method requires more calculation capacity. For both the Ditlevsen as for the Hohenbichler and Rackwits methods, influence coefficients (α-values) of all stochastic variables in the limit state functions of the base events should be known. These can be obtained by performing a First Order Reliability Method (FORM analysis) for each base event. The advantage of FORM analysis is that insight is obtained into the influence of the statistical properties of variables on the failure probability. (CUR-publicatie 190, 1997)

However, it is possible to compute the exact failure probability. This can be done by means of a Monte Carlo simulation. For MC simulations, a large amount of calculation cycles is performed, using new values for stochastic variables in each cycle. The failure probability is obtained by counting the number of failures and comparing this to the total number of cycles. With this method, all correlations between the events are automatically taken into account. For overloading of the glass elements, this results in a large model that requires a lot of calculation capacity. A disadvantage of performing a Monte Carlo simulation is that less insight is obtained into the influence of the variables on the failure probability. (CUR-publicatie 190, 1997)

The aim of this section is to obtain an accurate estimate of the failure probability for overloading of the glass elements. Therefore, it is chosen to perform a Monte Carlo analysis, rather than applying upper and lower bounds. Additionally, FORM analysis are performed for three (types of) base events: fracture in a glass section in the outer glass ply at the dry support, in the field and at the submerged support. This way, α-values are obtained as well. These can be valuable when proposing measures to increase the reliability.

Limit state functions

As mentioned before, the total number of base events in the Monte Carlo simulation is 1320. Half of these base events concern rectangular glass sections in glass plies with an exposed surface, while the other half concern rectangular glass sections in glass plies that do not have an exposed surface (not an outer ply). The latter become relevant after fracture of an outer ply. A large part of these base events states is computed with identical limit state functions. In fact, only three limit state functions are required, concerning:

- Fracture in a rectangular glass section at the submerged support
- Fracture in a rectangular glass section in the field
- Fracture in a rectangular glass section at the dry support

The derivation of these limit state functions is shown in Appendix E.1. Since the full functions are very comprehensive, they are presented in parts. The expressions from which the three limit state functions are composed, are given in Table 22.

Expression	Submerged support	Field	Dry support
General limit state function	$Z = f_g - \sigma_s$		
Stress at the glass surface	$\sigma_s(x) = \left\lvert \frac{6M(x)}{b \cdot t_{eq}{}^2} \right\rvert = \sqrt{\left(\frac{6M(x)}{b \cdot t_{eq}{}^2}\right)^2}$		
Bending moment	$M_1(x) = \frac{px^3}{6a} - \frac{px^2}{2} - EI(C_1x + C_2)$		$M_2(x) = -EI(C_5x + C_6)$
Location of maximum stress	$x = 0$	$x = -\frac{-ap + \sqrt{2C_1EIap + a^2p^2}}{p}$	$x = L$
Integration constant 1	$C_1 = -\frac{ap(10L^3 - 5La^2 + 2a^3)}{20L^3EI}$		-
Integration constant 2	$C_2 = \frac{pa^2(10L^2 - 10La + 3a^2)}{60L^2EI}$		-

Integration constant 5	-	$C_5 = \frac{pa^3(5L-2a)}{20L^3EI}$
Integration constant 6	-	$C_6 = -\frac{pa^3(10L-3a)}{60L^2EI}$
Bending stiffness	$EI = E\left(\frac{1}{12} \cdot b \cdot t_{eq}{}^3\right)$	
Equivalent glass thickness	$t_{eq} = r \cdot t$	
Real glass thickness	$t = \sum_{i=1}^{n} t_{p;i} + \sum_{j=1}^{m} t_{f;j}$	
Max. hydrostatic pressure	$p = \gamma_w(h-z)b$	
Submerged length of the glass	$a = (h-z)/\sin(\beta)$	

Table 22: Expressions from which the limit state functions are composed

For simplicity, the width of a glass element is set to one meter. Although the elements will be wider in reality, this has no consequences for the stresses that are computed. However, note that the hydrostatic pressure should also be computed for one meter width. Also note that absolute stress values are computed. The reason for this is that tensile stresses are indicated by a positive sign and for the above given schematization of the glass, compressive stress on one side of the element implies tensile stress on the other side of the element. By computing absolute values, tensile stresses in both outer glass plies in the elements are taken into account instead of just one. Since compression on one side of the panel implies tension on the other side of the panel, the total in tension loaded surface area of the two outer glass plies are exactly equal to the surface area of one glass ply. Therefore, this approach will not lead to an over- or underestimate of the failure probability.

The limit state functions are obtained by substituting the expressions in Table 22 step by step in upward direction, starting from the lowest row in the table (note that most expressions are used in all three limit state functions). This results in three functions, which are composed of the variables shown in Table 23.

Stochastic variable	Symbol	Unit	Statistical function	Parameter 1	Parameter 2
Tensile strength of glass	f_g	MPa	Weibull	u = 114,76	k = 3,93
Young's modulus of glass	E_g	MPa	Normal	μ = 70.000	σ = 100
Conversion factor to obtain the equivalent thickness	r	-	Normal	μ = 0,6	σ = 0,03
Height of the water level relative to N.A.P.	h	m	Shifted exponential	λ = 13	ε = -0,38
Height of the bottom of the glass plate relative to N.A.P.	z	m	Normal	μ = -1,2	σ = 0,005
Specific weight of water	γ_w	MN/m^3	Normal	μ = 0,01	σ = 0,00005
Length (/height) of a glass element	L	m	Normal	μ = 1,5	σ = 0,001
Width of a glass element (Only in calculation)	b	m	Deterministic	μ = 1,0	-
Thickness of a glass ply	$t_{p;i}$	m	Normal	μ = 0,01	σ = 0,00015
Thickness of a foil layer	$t_{f;j}$	m	Normal	μ = 0,00152	σ = 0,00002
Angle of the glass with respect to the horizontal axis	β	Rad	Normal	μ = 1,107	σ = 0,01107

Table 23: Statistical parameters of stochastic variables in the limit state functions

The derivation of statistical descriptors of the stochastic variables is elaborated in Appendix E.2. In Appendix E.3, it is shown how (parts of) the limit state functions are validated for correctness.

Monte Carlo simulation

As mentioned before, 660 limit states need to be computed for the outer glass plies. If fracture occurs in one or more of these plies, 60 limit states (per ply) need to be computed for the glass plies situated behind the fractured plies. All limit states and stochastic variables are programmed in a spreadsheet. The protocol of one calculation cycle in the Monte Carlo simulation is shown in Figure 62.

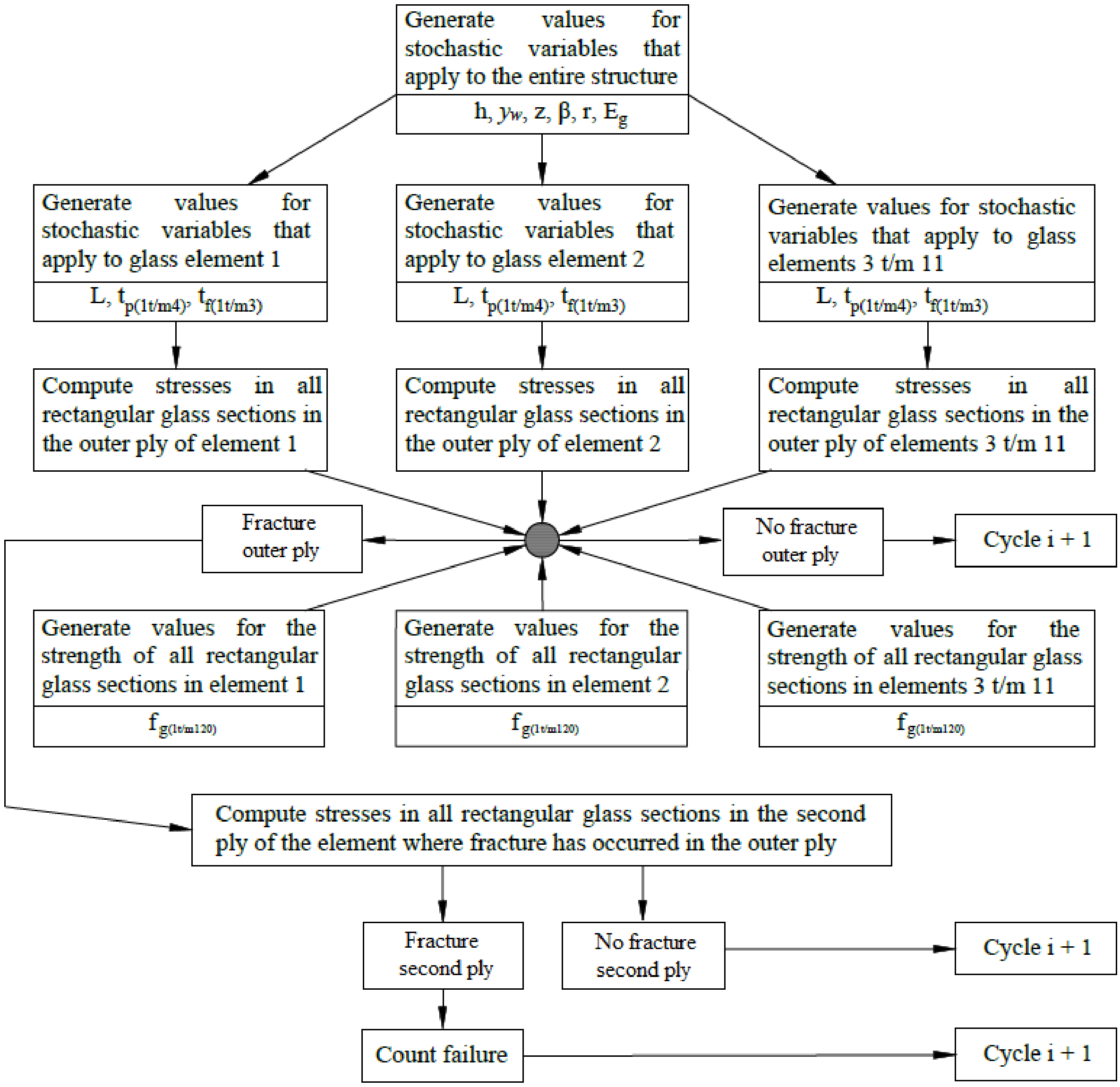


Figure 62: Monte Carlo simulation: flow chart for one calculation cycle

For some stochastic variables, only one value needs to be generated per calculation cycle. These are the water level, specific weight of water, height of the bottom of the glass elements, angle of the glass elements with respect to the horizontal axis, the conversion factor to obtain the equivalent thickness and the Young's modulus of glass. The length of a glass element can differ per element. Therefore, a value for the length has to be generated for each element separately. The same holds for the thickness of glass plies. Values for the strength need to be generated for each rectangular glass section separately. As explained before, this leads to minimum 660 values and maximum 1320 values for the glass strength per calculation cycle.

In order to obtain an accurate value of the failure probability, the number of calculation cycles that is needed is determined with help of the following rule of thumb:

$$\text{Number of calculation cycles} = 100/\text{failure probability}$$

Since the failure probability space for overloading of the glass elements is $3{,}25\cdot 10^{-7}$ per year (section 8.5.1), more than 300 million cycles are needed. On a fast personal computer, this corresponds to approximately 900 hours of computation. In order to obtain a first estimate of the failure probability for overloading, the number of calculation cycles is restricted to 10^{7}. The results are presented in section 8.5.1..

FORM analysis
The main advantage of performing FORM analysis is that insight is obtained into the influence of statistical properties of variables on the failure probability. In this thesis, this method is not applied to compute the total failure probability for overloading of glass elements as this is done by means of a Monte Carlo simulation. However, it is applied to evaluate a number of base events (limit states) in order to obtain information about the failure process for individual rectangular glass sections. The FORM analysis were executed with the software 'Prob2B'. The results are discussed in section 8.5.2.

8.5 Design optimization

8.5.1 Glass thickness

Target reliability for overloading
In chapter 7, failure probability spaces were assigned to all failure mechanisms of the glass dike. The total failure probability of glass elements may not exceed $1{,}3\cdot 10^{-6}$/year. This value should be further subdivided over the failure mechanisms that apply to the glass elements. In Figure 51, four types of failure are shown. At this stage, it is not known what would be a good distribution of failure spaces. Therefore, maximum failure probabilities for different types of failure in the glass elements are simply set equal. Then, the maximum failure probability for overloading becomes:

$$p_{f;max} = \frac{1{,}3\cdot 10^{-6}}{4} = 3{,}25\cdot 10^{-7}/\text{year}$$

Required glass thickness
In section 8.3.1, the composition of glass plies in an element was defined. The thickness of glass plies that is required to obtain a sufficient low failure probability was estimated to be 8 mm per ply. This appeared to be insufficient. Therefore, a Monte Carlo simulation was also performed for glass elements with ply thicknesses of 10 mm. This resulted in a failure probability of 1,3·10-6 per year for overloading, which is insufficient as well. At last, the probabilistic calculation was repeated for glass elements with glass plies of 12 mm thick. This resulted in a failure probability for overloading of 2·10-7 per year. Since this is below the target probability, this is sufficient. In summary, glass elements with a composition as shown in Table 24 should be applied.

Glass type	Heat-strengthened
Number of glass plies	4 per element
Thickness per ply	12 mm
Type of interlayer material	SentryGlas
Thickness of an interlayer	1.52 mm
Total thickness of a glass element	52.56 mm
Failure probability for overloading	$2\cdot 10^{-7}$ per year

Table 24: Composition of a glass element

Note that the offer layer of fully tempered glass, as described in section 8.3.1, is not shown in Table 24. The reason for this, is that the offer layer is not taken into account in the probabilistic calculation.

8.5.2 Sensitivity analysis

In this section, the sensitivity of the failure probability for overloading of glass elements to a number of parameters is examined. First, the influence of statistical properties of stochastic variables is regarded. If further optimization of the failure probability is desired, this can be used to determine for which variables extra research is most effective. Furthermore, the effect of the thickness of glass plies is investigated. After that, the effect of the surface area of glass elements is examined. This could be relevant if, for some reason, it is chosen to increase the width of the glass dike. At last, the consequence of the applied failure definition for glass elements on the failure probability is examined.

__Influence of statistical properties of stochastic variables on the failure probability__

The influence of statistical properties of stochastic variables on the failure probability is examined by means of FORM analysis. For each stochastic variable, the design point and α-value are determined. When failure has occurred, the value of a stochastic variable is most likely to be near the design point. The α-value of a stochastic variable gives the relative contribution of the variables uncertainty on the failure probability. These characteristics are specified for the cases shown in Table 25.

Case	Submerged support	Field	Dry support	Ply	Water level
1	x			1	Variable
2		x		1	Variable
3			x	1	Variable
4	x			2	-0,03
5		x		2	-0,03
6			x	2	-0,03

Table 25: Cases for which FORM analysis was performed

Case 1, 2 and 3 correspond to limit states for fracture of the exposed glass ply. Case 4, 5 and 6 correspond to limit states for fracture of the second ply, given that the first ply already fractured. Since fracture of the outer ply is expected to occur at high water, a deterministic value equal to the design water level of -0,03 meter N.A.P. was applied. The results are shown in Table 33, in which design points are indicated with X. A histogram with α-values for the different cases is shown by Figure 63.

	Case 1		Case 2		Case 3		Case 4		Case 5		Case 6	
Variable	α	X	α	X	α	X	α	X	α	X	α	X
E_g	0,000E+00	7,000E+04	-1,388E-15	7,000E+04	2,522E-16	7,000E+04	-6,570E-16	7,000E+04	-4,879E-15	7,000E+04	-1,893E-15	7,000E+04
L	-4,555E-04	1,500E+00	-2,254E-04	1,500E+00	7,788E-05	1,500E+00	-8,102E-04	1,500E+00	-4,728E-04	1,500E+00	-5,733E-05	1,500E+00
b	0,000E+00	1,000E+00	0,000E+00	1,000E+00	0,000E+00	1,000E+00	0,000E+00	1,000E+00	0,000E+00	1,000E+00	0,000E+00	1,000E+00
β	4,563E-03	1,107E+00	5,043E-03	1,107E+00	7,730E-03	1,107E+00	4,779E-03	1,107E+00	4,916E-03	1,107E+00	9,183E-03	1,107E+00
f_g	9,785E-01	6,733E+00	9,776E-01	2,737E+00	9,464E-01	4,115E+00	9,944E-01	1,878E+01	9,967E-01	7,783E+00	9,960E-01	1,098E+01
h	-1,915E-01	-2,587E-01	-2,005E-01	-2,371E-01	-3,161E-01	-1,662E-01	0,000E+00	-3,000E-02	0,000E+00	-3,000E-02	0,000E+00	-3,000E-02
r	7,500E-02	5,904E-01	6,215E-02	5,906E-01	6,415E-02	5,906E-01	1,034E-01	5,902E-01	7,946E-02	5,903E-01	8,696E-02	5,902E-01
t_{f1}	6,781E-04	1,520E-03	5,662E-04	1,520E-03	5,805E-04	1,520E-03	1,260E-03	1,520E-03	9,687E-04	1,520E-03	1,060E-03	1,520E-03
t_{f2}	6,781E-04	1,520E-03	5,662E-04	1,520E-03	5,805E-04	1,520E-03	1,260E-03	1,520E-03	9,687E-04	1,520E-03	1,060E-03	1,520E-03
t_{f3}	6,781E-04	1,520E-03	5,662E-04	1,520E-03	5,805E-04	1,520E-03	0,000E+00	0,000E+00	0,000E+00	0,000E+00	0,000E+00	0,000E+00
t_{p1}	5,079E-03	9,997E-03	4,211E-03	9,997E-03	4,348E-03	9,997E-03	9,434E-03	9,996E-03	7,252E-03	9,996E-03	7,936E-03	9,996E-03
t_{p2}	5,079E-03	9,997E-03	4,211E-03	9,997E-03	4,348E-03	9,997E-03	9,434E-03	9,996E-03	7,252E-03	9,996E-03	7,936E-03	9,996E-03
t_{p3}	5,079E-03	9,997E-03	4,211E-03	9,997E-03	4,348E-03	9,997E-03	9,434E-03	9,996E-03	7,252E-03	9,996E-03	7,936E-03	9,996E-03
t_{p4}	5,079E-03	9,997E-03	4,211E-03	9,997E-03	4,348E-03	9,997E-03	0,000E+00	0,000E+00	0,000E+00	0,000E+00	0,000E+00	0,000E+00
γ_w	-3,776E-03	1,000E-02	-3,131E-03	1,000E-02	-3,233E-03	1,000E-02	-5,203E-03	1,000E-02	-4,000E-03	1,000E-02	-4,377E-03	1,000E-02
z	8,450E-03	-1,200E+00	8,070E-03	-1,200E+00	1,020E-02	-1,200E+00	8,146E-03	-1,200E+00	7,223E-03	-1,200E+00	1,085E-02	-1,200E+00

Table 26: α-values and design points (X) of stochastic variables

Since the α-value for the tensile strength of glass is much larger than α-values of the other stochastic variables, failure of the glass elements is heavily dominated by uncertainty in the strength of glass. This is supported by the fact that the design point of the strength varies from approximately 2 MPa up to 11 MPa for the different limit states, while the average strength is 104 MPa. A tensile strength of 2 MPa seems unrealistic low. No literature exists in which such a low strength is reported. This stresses the need for good statistical descriptors for the strength of glass. Apart from the strength of glass, it is observed that the uncertainty of the water level and uncertainty of the conversion factor (bonding) both have a reasonable influence on the failure probability. The influence of the other stochastic variables is negligible.

Note that α-values only give information about the influence of stochastic variables in a probabilistic calculation. This influence is mainly determined by uncertainty, usually expressed by the standard deviation. This information could for instance be used to determine what research has to be performed in order to effectively decrease uncertainty. However, α-values give little information about the sensitivity of the failure probability to changes of mean values of stochastic variables. If for example the thickness of each glass ply is increased from 10 to 12 mm, the failure probability reduces significantly. This effect is not observed from the α-values for the thickness of glass plies as these are close to zero.

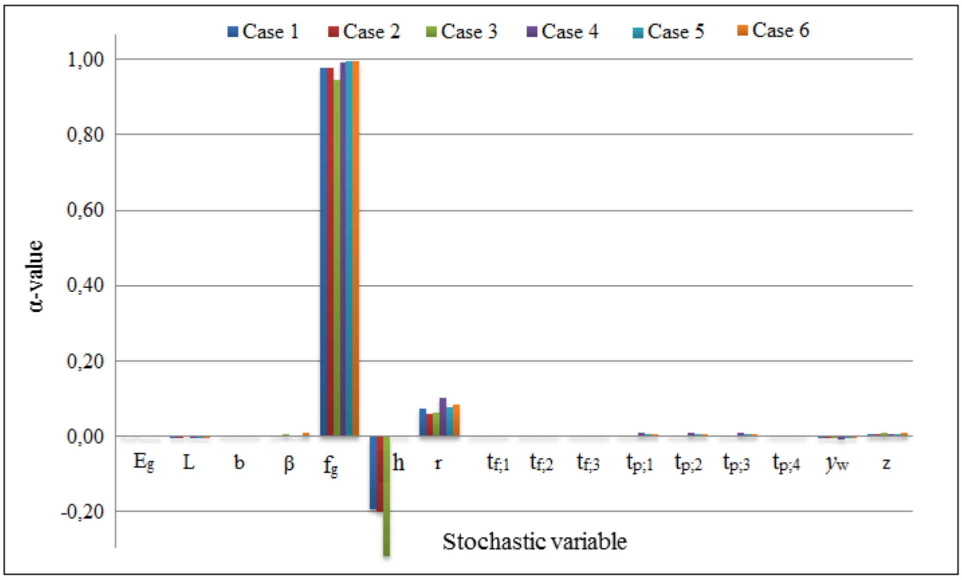
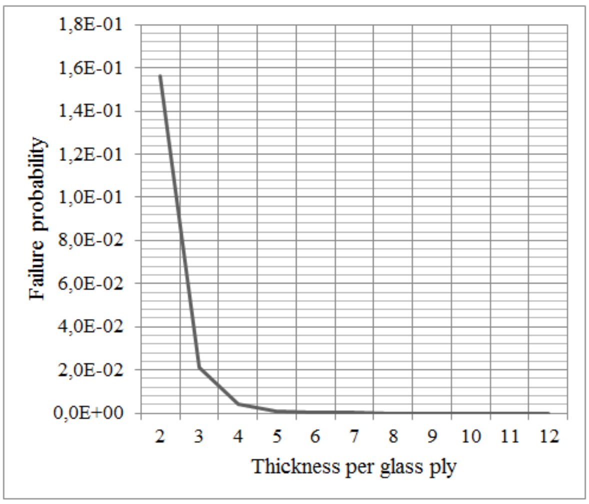


Figure 63: Alpha values of stochastic variables

Thickness

Stresses at the surface of a glass element are inversely proportional to the thickness of glass plies in the element. Consequently, the same holds for the failure probability for overloading. The effect of the thickness on the failure probability is examined by performing FORM calculations for varying thicknesses of the glass plies. In each calculation, all four glass plies are equally thick, so an increase of 1 mm per glass ply results in an increase of 4 mm of the glass element. The results are shown in Figure 64.

Note that values of the failure probabilities shown in Figure 64 correspond to fracture in a rectangular glass section in an outer glass ply at the submerged support. These values are not representative for failure of an entire glass element, but are only used to obtain qualitative insight into the effect of the glass thickness on the failure probability. As can be seen, the failure probability decreases exponentially for increasing glass thickness.



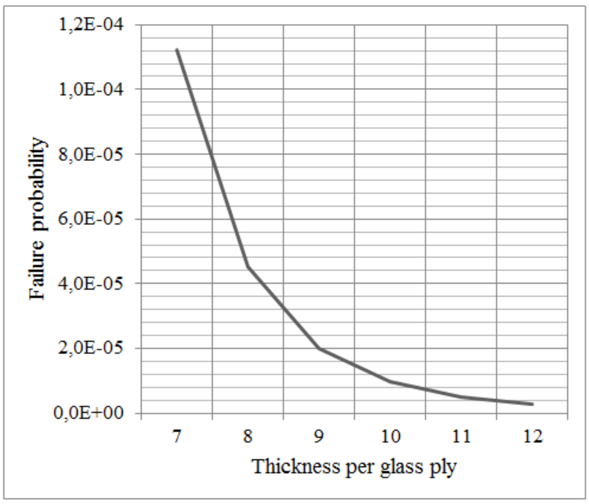


Figure 64: Effect of the thickness of glass plies on the fracture probability of a rectangular glass section

Surface area

The strength of glass depends on the presence of damages at the surface and at the edges. For a larger surface area of glass elements, the probability of the presence of some critical damage is also larger. Consequently the probability of failure due to overloading increases as well. The effect of the surface area has been examined by performing a number of Monte Carlo simulations, each time for a different number of glass elements in the glass dike. The result is shown in Figure 65.

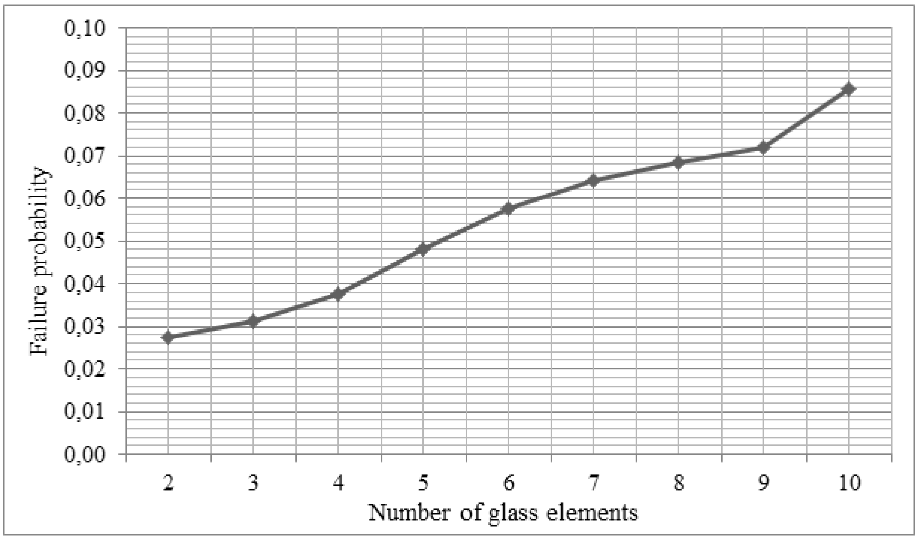


Figure 65: Effect of the number of glass elements on the total failure probability for overloading (with reduced glass thickness)

Failure probabilities as shown in Figure 65 are based on glass elements that consist of four plies with each a thickness of 4 mm. This has been done to reduce the required number of calculation cycles in the Monte Carlo Simulations. So Figure 65 does not give information about the real failure probability, but only gives qualitative insight into the sensitivity of the failure probability to the surface area of glass elements. As can be seen, the failure probability is linear proportional the number of glass elements (width of the glass dike). In this example, the failure probability is tripled when the number of glass elements is multiplied by five. This can be explained by the large spread in glass strength. Due to this spread, the failure probability is dominated by weak spots in the glass.

Failure definition

The water retaining function of a glass element is lost if all glass plies are fractured and all interlayers are ripped. However, a more conservative definition of failure was defined in section 8.4.2. It is assumed that failure has occurred if two glass plies in a glass element are fractured. Residual strength of the other plies and interlayers is not taken into account. Alternatively, it could also be chosen to allow fracture in only one glass ply. Failure probabilities for both failure definitions are shown in Figure 66.

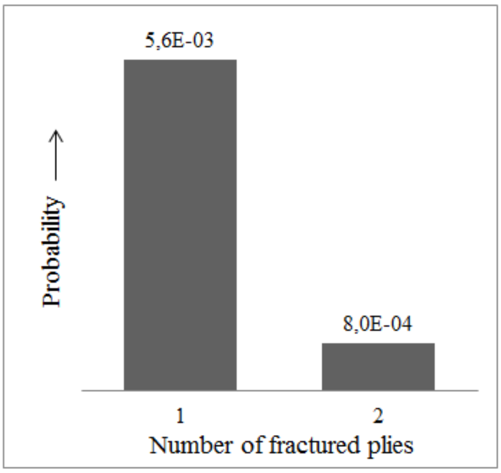


Figure 66: Failure probability vs. number of fractured glass plies in an element (with reduced thickness)

Again, the thickness of glass plies in glass elements have been reduced to decrease the required number of calculation cycles in the Monte Carlo simulations, so failure probabilities shown in Figure 66 are not representative for overloading failure of glass elements in the glass dike. In the figure, it can be seen that the failure probability is reduced significantly if more glass plies are allowed to fracture. For glass elements consisting of four glass plies, it is not wise to increase the number of plies that are allowed to fracture. However, if glass elements consisting of more than four plies are applied, the failure definition could be changed to optimize the design and prevent over dimensioning. This is a design choice and has no consequences for the 'real' failure probability.

8.5.3 Options to increase the reliability

So far, it has been established to dimension the glass elements such that the probability of overloading of glass elements is low. However, probabilities of other failure mechanisms that apply to the glass should satisfy (be lower than) the same target failure probability. Unfortunately, the exact probabilities of these failure mechanisms could not be quantified in this thesis. In this section, a number of additional design options are given, which are intended to increase the reliability of the glass dike. Based on the knowledge that is gained so far, the effect of these options on the failure probability is qualitatively described for each failure mechanism. These failure mechanisms are shown in the fault tree in Figure 51 and can be subdivided into four main groups: Overloading, Impact, Explosion and Fire.

Option A: additional row of glass elements

By adding an extra row of glass elements behind the first (permanently) water retaining row, a breach can only occur if glass elements in both rows have failed. Since overloading failure is dominated by weak spots in the glass, the additional row of glass elements will reduce the failure probability significantly.

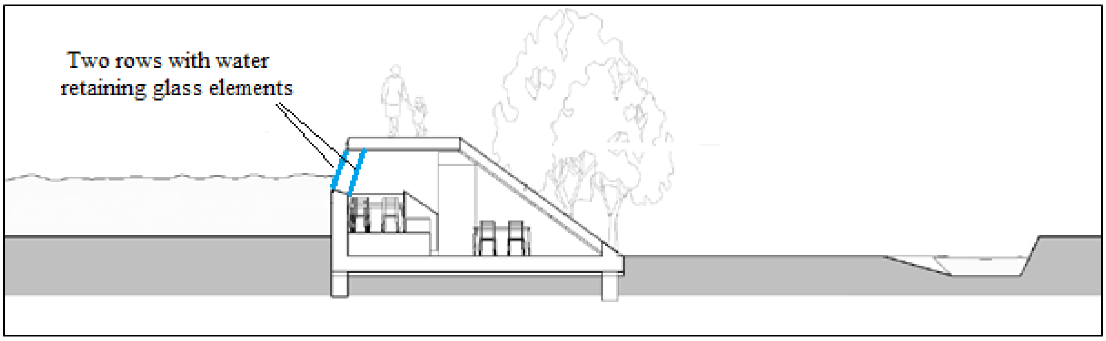


Figure 67: Additional row of glass elements

The probability of failure due to a boat collision is reduced as well, but this depends on the distance between the glass elements and the amount of collision energy that is dissipated by the first row. The same holds for an explosion and for fire, although both require more research. In Figure 67, the rows with water retaining glass panels are indicated with blue lines.

Option B: Water retaining function of glass elements at the polder side

An alternative for application of a second row of glass elements just behind the first row, is to design the sloped glass wall at the polder-side of the glass dike in such a way that it can retain water as well. In Figure 68, the rows with water retaining glass panels are indicated with blue lines.

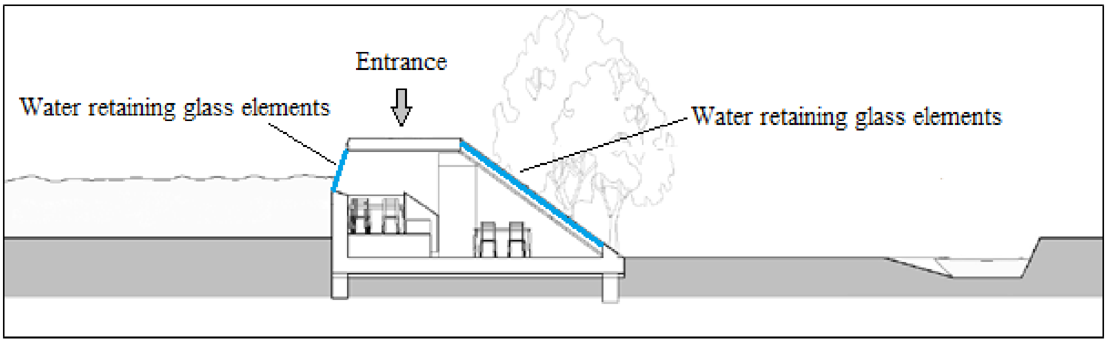


Figure 68: Water retaining functions of glass panels at the polder-side of the glass dike

From an aesthetical point of view, this option is more attractive than option A. Another advantage is that glass elements at the polder-side cannot fail due to a boat collision. There are also disadvantages. First of all, the structure should be water tight at both sides, so no openings are allowed at a height that is lower than the water level. Access to the glass dike should be provided by an entrance at the crest of the dike. Another disadvantage is that glass elements at the polder-side will be subjected to larger stresses as it has to retain water to a larger depth. At last, the row of glass elements that is parmenently retaining water from the Noordvliet is less protected against fire and explosions inside the structure as is the case for option A.

Option C: Boat collision protection

In section 8.2.4, it was concluded that boat collisions represent a large threat on the water retaining function of the glass dike. This threat is strongly reduced by applying option A or B. However, the glass dike itself may still incur large damages as some glass elements need to be replaced and water may enter the structure. This can be prevented by placement of a protective structure in front of the glass dike.

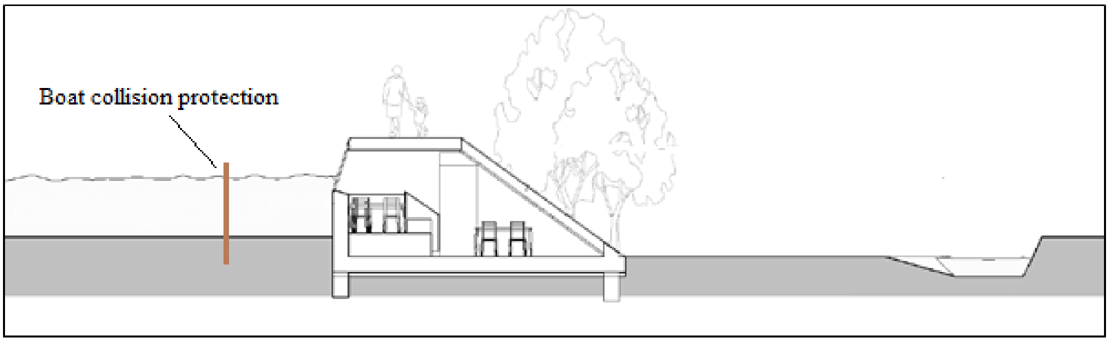


Figure 69: Boat collision protection

Placing a structure in front of the glass dike in order to prevent boats to sail against the glass elements, has no influence on the failure probability of other failure mechanisms like overloading, fire and explosions. It should also be noted that the protective structure has a certain failure probability as well, meaning that there still is a small probability that a boat collides against the glass dike. Furthermore, other impacts, for instance due to vandalism, remain a threat.

Option D: Prevention of fire and explosions

Option D is not really one option, but rather a number of suggestions to prevent fire and explosions and/or to reduce consequences. A summation of these suggestions is given below:

- Install sprinkler system
- Use electricity for cooking instead of gas
- Use electricity for heating instead of gas
- Install rolling shutters that protect the glass elements against fire

Effectiveness of design options

In Table 27, a qualitative overview is given of the effect of above described design options on the susceptibility to failure mechanisms that apply to the glass elements. The choice for one or more of these options is not just a matter of safety, but also financial, aesthetical and practical considerations play a role.

Design option	Failure mechanism			
	Overloading	Impact	Explosion	Fire
Option A	+++	++	++	++
Option B	++	+++	+	+
Option C	n.e.	++	n.e.	n.e.
Option D	n.e.	n.e.	++	++
n.e.: no effect +: reduced failure probability				

Table 27: Effect of design options on failure mechanisms that apply to the glass elements

8.6 Periodic assessment of the glass elements

The safety level of flood defences in the Netherlands is assessed once every 12 years. Rules for assessment of structural elements in hydraulic structures are given in LRK2011. For the glass elements, it is attempted to affiliate with this guideline as much as possible. A flowchart, with four methods for the assessment of structural elements, is given in Figure 70.

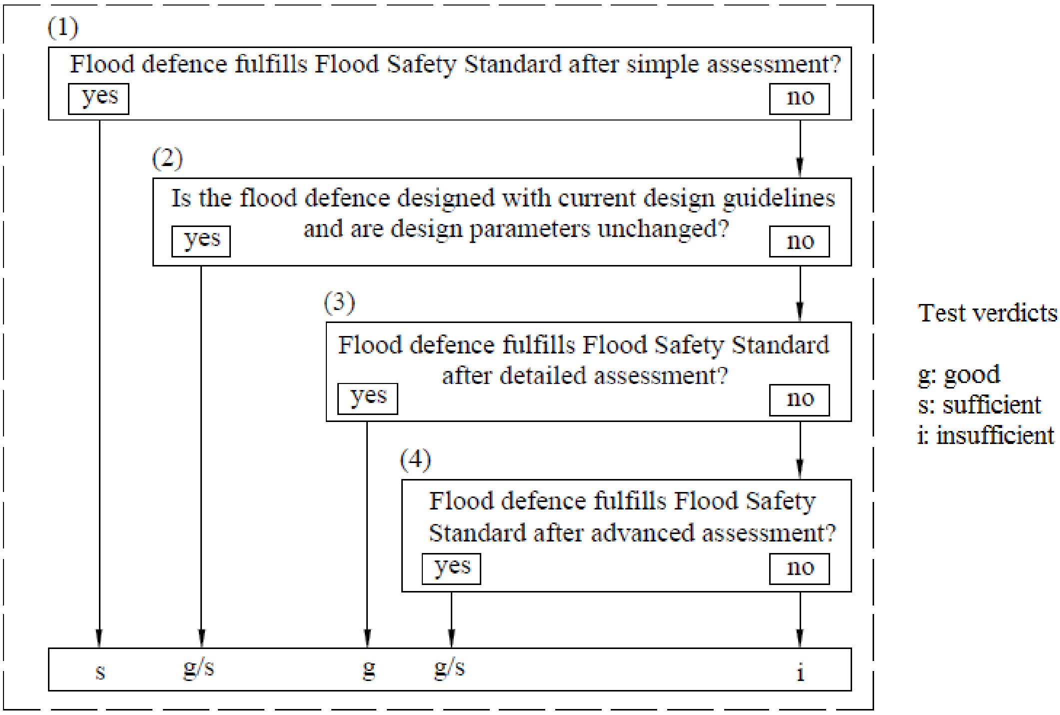


Figure 70: Flowchart with assessment methods for structural elements in a flood defence (LRK2011)

The four assessment methods are:

1. Simple assessment
2. Assessment of design calculations
3. Detailed assessment
4. Advanced assessment

For innovative flood defences, simple assessment (1) is not possible as practical experience is required. In the most favourable situation, the glass elements can be assessed based on the original design calculations (2). However, the glass elements are designed with an advanced (probabilistic) method. Approval of the glass elements, based on the probabilistic design, is possible if the following conditions are met:

- The strength of the glass elements may not be affected because of damage
- Actual boundary conditions may not be less favourable than design boundary conditions
- Current safety requirements may not be stricter than design safety requirements
- The design lifetime of the glass elements may not be exceeded

Based on the flowchart in Figure 70, a new procedure for assessment of the glass elements is proposed. This procedure is explained by the flowchart in Figure 71.

<u>Strength of the glass elements</u>

If one of the load bearing glass plies in a glass element is fractured or critically damaged, it cannot be approved. Therefore, damaged glass elements will get the verdict 'insufficient' and replacement is required. This only holds for glass plies that are taken into account in strength checks that were performed for the design. If an offer (protection) layer is fractured, approval is still possible.

Boundary conditions

In probabilistic design calculations, the water level is represented by a statistical distribution function. Parameters in this distribution function may not have become less favourable, for instance due to climate change or alterations in the water system. If boundary conditions have become less favourable, advanced assessment is required. For this, the probabilistic design calculations can be used, but with altered statistical parameters of the water level.

Design lifetime

Glass is a very durable material. However, bonding between the glass plies (and thereby the total load bearing capacity) will decrease with increasing load duration. This strength decrease is taken into account in the design. However, when the design lifetime (100 years) is exceeded, it should be checked whether the load bearing capacity of the glass elements is still sufficient.

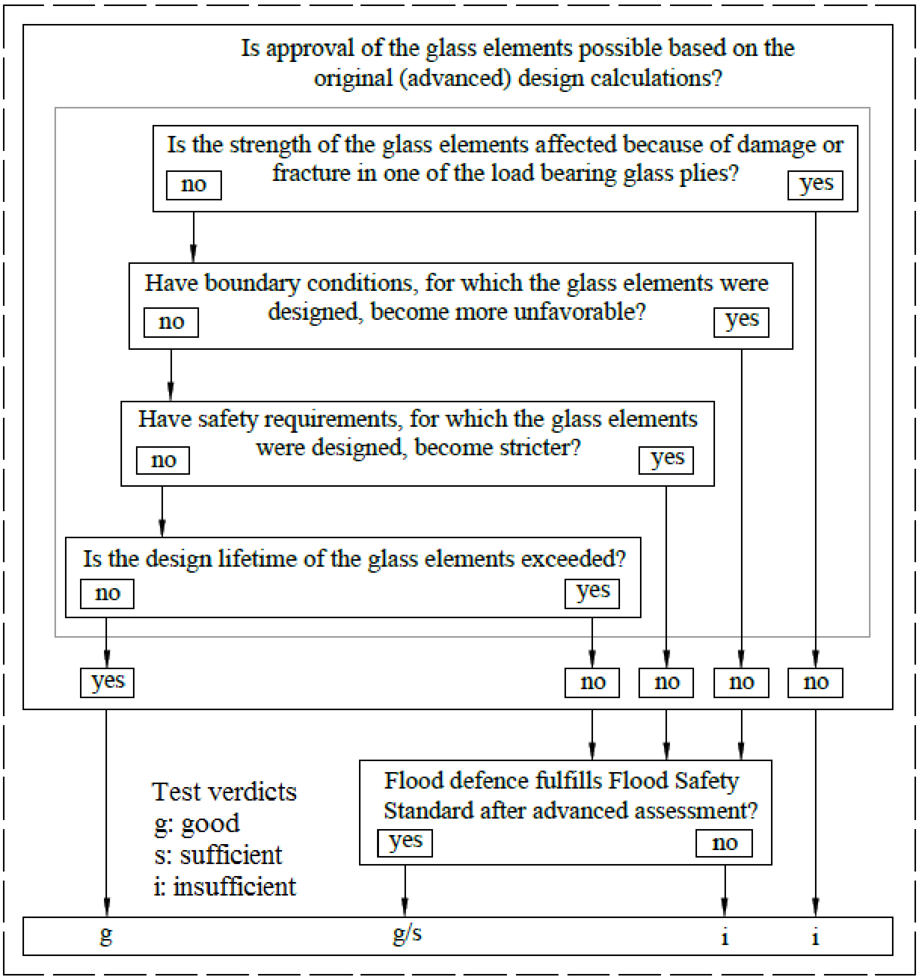


Figure 71: Flowchart for assessment of the glass elements

Designing for assessment

A negative judgement of the glass elements can be prevented by taking the above described considerations into account in the design. The following advices are given for this:

- Provide replaceable (glass) offer layers on the water side of the glass elements. This way, damage to glass plies that contribute to the load bearing function is prevented.
- Use conservative values for statistical parameters of the water level. If safety requirements have become more strict, boundary conditions have become less favourable or if the design lifetime of the glass elements is exceeded, disapproval of the glass elements based on an advanced assessment is less likely.
- Design the glass elements for a lifetime of 100 years.

8.7 Discussion

In this chapter, the glass elements and frames were designed and a probabilistic model for overloading of the glass is proposed. During the process, some assumptions were made. Consequences of these assumptions on the results of probabilistic calculations are explained in the corresponding sections, but a summation is given below. Furthermore, some points of attention that require more research are given. At last, the approach of the probabilistic method in general evaluated and compared with alternative methods.

Consequences of assumptions

- The strength of glass is largely correlated to the quality of the edges. Statistical descriptors of the strength were derived from tests, in which specimens were used with a large ratio of edge length over surface area. This 'relative edge length' is much smaller for glass plies in the glass dike. Therefore, the failure probability is overestimated.
- Glass plies are subdivided in rectangular sections that are smaller than the test specimens that were used to obtain statistical descriptors of the glass strength. As a result, the probability of overloading of glass elements is overestimated.
- Stresses in rectangular glass sections are assumed to be constant in the entire section and equal to the maximum stress in the particular section, while in reality, stresses are lower in large parts of the section. Since the strength of glass is largely location dependent, the failure probability is overestimated.
- Stresses in the glass elements are computed with a simplified (beam)model. If the glass elements are schematized as plates supported at all four edges, a smaller (and more realistic) failure probability will be obtained.
- Glass elements are protected by a thin ply of fully tempered glass. This 'offer layer' is not taken into account in the probabilistic calculation, but does contribute to the strength of glass elements.

Points of attention

- In order to obtain a more accurate value of the failure probability for overloading, more cycles in the Monte Carlo simulation should be performed.
- A Weibull distribution is applied to generate values for the strength of glass. For this distribution, a very conservative value was used for the variance. In the probabilistic calculation, fracture occurred at tensile stresses in the order of 5 MPa, while the average tensile strength of heat-strengthened glass is approximately 104 MPa. Although the strength of glass has a large spread, it should be questioned if above mentioned value for the fracture stress is realistic.
- Statistical properties of the conversion factor (for bonding) are estimated. The amount of bonding between glass plies depends on temperature and duration of loading. The conversion factor should accurately include this relation. Alternatively, it could be assumed that there is no bonding between glass plies at all.
- For overloading of the glass elements, only hydrostatic pressure is taken into account. Waves were not taken into account as these are very small and cause pressure only locally (waves arrive at an angle). Whether this simplification is permissible should be examined more thoroughly.

Evaluation of the probabilistic calculation approach

An abbreviated fault tree that includes the base events and intermediate events that lead to overloading of the glass elements is shown in Figure 61. For the probabilistic calculation, glass plies were subdivided into small rectangular sections. Each base event is defined as fracture in a specific glass section. As a result, the fault tree consists of 1320 base events (rectangular sections). In order to accurately compute the probability of the top event (overloading), dependencies between base events and intermediate events should be fully taken into account. This is done by performing Monte Carlo simulations, which is a rather impractical task for such a large fault tree. Furthermore, a high level of computational capacity is required. Another disadvantage is that influence coefficients of stochastic variables are not obtained.

A promising alternative to the proposed approach in this thesis, is to apply probabilistic fracture mechanics, which is especially useful for brittle materials like glass. This method includes the relation between maximum admissible stresses and the location and size of damages, but also incorporates statistical uncertainties (Rahman, 2001). However, damage at the glass surface is just one of the strength criteria. Typical for glass, is that its strength is largely influenced by the quality of the production process, especially for pre-stressing. To date, it has not been established to represent this influence with statistical functions. As long as this is the case, it has no use to apply probabilistic fracture mechanics. At the time, the only solution is to apply strength statistics in which all parameters are implicitly taken into account as is done in this thesis.

8.8 Conclusion

In this chapter, the glass elements and frames were designed and a probabilistic model for overloading of the glass is proposed. Other failure mechanisms that apply to the glass elements are impact due to a boat collision or vandalism, explosions and fire. For these failure mechanisms, no probabilistic calculations were performed, but the threat of these failure mechanisms were qualitatively assessed in section 8.2. It was concluded that additional measures are needed in order to prevent boat collisions or to reduce corresponding consequences. This can be done by respectively placing a protective structure in front of the glass dike that stops boats from sailing against the glass elements or by applying an additional row of glass elements as back-up. Options to increase the reliability of the glass dike are elaborated in section 8.5. For vandalism, tests can be performed to check if the glass elements are able to resists impact forces that correspond to this threat. For fire and explosions, more research is needed in order to be able to accurately determine the corresponding failure probability of the glass elements. As long as this is not possible, sufficient measures should be taken to prevent these events or the reduce consequences.

In section 8.5.1, it was determined that for each failure mechanism that applies to the glass elements, the failure probability should be lower than $3{,}25 \cdot 10^{-7}$ per year. For overloading of the glass elements, a number of Monte Carlo simulations were performed, each time for a different glass thickness. It followed that the probability of overloading is sufficient low for the composition of a glass element as shown in Table 28.

Glass type	Heat-strengthened
Number of glass plies	4 per element
Thickness per ply	12 mm
Type of interlayer material	SentryGlas
Thickness of an interlayer	1.52 mm
Total thickness of a glass element	52.56 mm
Failure probability for overloading	$2 \cdot 10^{-7}$ per year

Table 28: Composition of a glass element

The water retaining function of a glass element is lost if all glass plies are fractured and all interlayers are ripped. However, a more conservative definition of failure was defined in section 8.4.2. It is assumed that failure has occurred if two glass plies in a glass element are fractured. Apart from this failure definition, more assumptions were made in order to simplify the probabilistic model. For safety, only conservative assumptions were made. A summation of these assumptions is given in 8.6. The result is an overestimated probability for overloading of the glass elements.

The sensitivity of the failure probability for overloading of glass elements to a number of parameters is examined in section 8.5.2. The main findings are summarized below:

- Failure of the glass elements is heavily dominated by uncertainty in the glass strength.
- As a result from the point above, the failure probability increases linear for increasing surface area of the glass elements.
- The failure probability decreases exponentially for increasing thickness of glass plies.

Part III

Generalization

9. Generalization

Although it is not usual to apply glass in a flood defence, it is conceivable that more situations will arise in which transparency of the flood defence is desired, especially in urban areas. If the glass dike is realized, this will be the first transparent flood defence that is equipped with permanently water retaining glass elements. Therefore, the glass dike could be seen as a pilot project. As for all innovations, (many) problems are encountered, which have to be solved eventually. For the glass dike, a great deal of these problems are identified in this thesis. In this chapter, the lessons that were learned will be used to say something about the considerations that play a role for glass flood defences in general. First, some technical issues will be discussed in section 9.1. After this, legislations with respect to the reliability of flood defences are discussed in section 9.2. This is done for primary flood defences and regional flood defences separately. At last, the feasibility of glass flood defences in general will be discussed in section 9.3.

9.1 Technical considerations of water retaining glass as part of a flood defence

At the moment, the number of structural application of glass is increasing. Some examples were given in section 4.6. Although the load types in these examples are all different, they have one thing in common: load durations are relatively short. These durations are in the range of less than a second to a couple of years. The bearing capacity of laminated glass decreases significantly with increasing load duration. Hydraulic structures in flood defences are designed for a lifetime of up to 100 years. If the glass elements permanently retain water, this has consequences for the required initial bearing capacity of the laminated glass. One could argue that a glass flood defence can be best compared to a large aquarium. Technically, this is not entirely correct. First of all, if a glass flood defence fails, consequences are much larger compared to broken aquaria. Secondly, loads are different as well. Where an aquarium only has to resist hydrostatic pressure, a flood defence also has to resist (slamming) wave forces, impacts loads, ice loads, etcetera.

Threats

The brittle failure behaviour of glass makes it extra susceptible to impact loads such as a boat collision, vandalism, but also floating objects like the trunk of a tree can be a serious threat. For unprotected glass elements, these impacts can be governing for the design. This also holds for explosions, which cause a large over pressure on the glass. Another important threat is fire as glass is a poor material when it comes to fire resistance compared to steel or concrete. It should be noted that for most flood defences, a fire is very unlike to occur. Moreover, if there is no potential fire source, it is not needed to take this into account at all. In the case of the glass dike, hydrostatic pressure plays an inferior role. This is because the glass dike has to retain water to a limited depth. Since stresses in the glass increase exponentially with increasing retaining depth, hydrostatic pressure may be the governing load for glass elements in other flood defences.

Robustness

In contrast to steel and reinforced concrete, glass is not able to deform plastically. If the maximum admissible stress is exceeded, glass plates will break completely. For a flood defence, this is a large disadvantage. This can partially be solved by laminating a number of glass plies to form one panel. If fracture occurs in one ply, the other plies can take over the load. Unfortunately, interlayer materials in laminated glass are less stiff and exhibit creep. This has consequences for the durability.

Durability

The load bearing capacity of glass decreases with the duration of loading. This is caused by stress corrosion in the glass (not for pre-stressed glass) and creep in the interlayer material. In most cases, flood defences are permanently retaining water for large periods of time, up to 100 years. As a result, the load bearing capacity will decrease tremendously. This needs to be taken into account in the design. For pre-stressed glass, the load bearing capacity will not decrease further as soon as there is no bonding between the glass plies anymore. Another issue that should be considered is that scratches and little damages on the glass, which are not always visible, also affect the strength. If the glass is not protected, the number of scratches will increase during the lifetime.

Probabilistic calculations
In order to prove the reliability of a glass flood defence, probabilistic calculation must be performed. For glass, this is very complicated since the strength of glass depends on a very large amount of factors. Amongst others, these are the length/width ratio, type of glass, quality of the edges, quality of the production process, pre-stressing method and amount of (invisible) damages on the surface. As a result, there are still no good statistical distribution functions to use in probabilistic calculation. This can only be solved by making very conservative assumptions for statistical descriptors. For laminated glass, probabilistic calculations are even more difficult to perform as additional temperature and load duration effects are introduced.

9.2 Flood Safety Standard versus Eurocodes

In part II of this thesis, the required reliability of the glass dike was defined by regarding the Flood Safety Standard and the Eurocodes. An overlap in these requirements was found. This is described in a general way in this section.

Water retaining hydraulic structures, which are part of a flood defence, should always comply to Eurocodes. With respect to structural reliability, the Eurocode prescribes three different classes. Primary flood defences should always satisfy the minimum reliability that corresponds to reliability class 3, which is the highest class. Regional flood defences must satisfy reliability class 1 or 2, depending on the consequences of a breach. For conventional hydraulic structures, design guidelines prescribe which Eurocode reliability class has to be applied.

In section 5.6, it was explained that the required reliability that follows from the Eurocode is assessed in a different way than the required reliability that follows from the Flood Safety Standard. The latter demands an integral reliability for the water retaining structure as a whole, while the Eurocode sets reliability requirements to individual failure mechanisms (strength of structural elements and stability of the structure). Applying the Eurocode to achieve sufficient low failure probabilities for individual failure mechanisms may lead to a reliability of the total structure that does not satisfy the Flood Safety Standard. For each water retaining hydraulic structure, it should be checked which requirement is governing.

9.2.1 Regional flood defences
When designing innovative types of flood defences, probabilistic methods must be applied to prove the reliability. For regional flood defences, the Flood Safety Standard prescribes a water level with a certain exceedance probability to which the flood defence must be designed. In order to verify if the flood defence is sufficient reliable with respect to the Flood Safety Standard, a translation needs to be made from this exceedance probability of the design water level to a maximum failure probability. For hydraulic structures, LRK2011 prescribes how this should be done. For structural failure, a maximum probability of 1/100 times the exceedance probability, given that the design level is not exceeded, is prescribed. The note ‘given that the design water level is not exceeded’ is a bit odd. This implies that in probabilistic calculations, the probability density- and distribution functions should be cut off at the design water level. It is more practical (and safer) to also take into account water levels higher than the design water level.

Another thing that should be noted for regional flood defences, is that apart from the requirement for structural reliability, separate requirements apply to the reliability of moving elements (like the doors of a lock) and to the height of the flood defence. Summation of the maximum probabilities corresponding to the different types of failure leads to a total failure probability larger than the water level exceedance probability. One could interpret this as non-satisfaction of the Flood Safety Standard. However, the water level exceedance probability should not be confused with a maximum failure probability.

For a multifunctional regional flood defence like the glass dike, it is required to apply a higher Eurocode reliability class than for a conventional hydraulic structure in the same flood defence. The reason for this, is that consequences of failure are larger. If a glass element in the glass dike fails, people might get trapped inside the structure and drown, while flooding of the polder would be the only consequence for failure of a conventional water retaining structure. Primary flood defences should always satisfy the highest Eurocode reliability class, regardless whether or not it is a multifunctional flood defence.

9.2.2 Primary flood defences

In 2017, the Flood Safety Standard for primary flood defences will be changed. Instead of a design water level with a prescribed exceedance probability, flood defences must be designed such that a prescribed maximum probability of flooding is satisfied. In contrast to the current Flood Safety Standard, all failure mechanisms are taken into account in this integral failure probability. An advantage of this requirement is that it can be compared to the results of probabilistic calculations more directly.

The maximum probability of flooding as prescribed by the new Flood Safety Standard applies to dike trajectories. If part of the trajectory is replaced by a glass flood defence, not only the failure probability of the glass flood defence must be computed, but the probability of a breach in the total dike trajectory should be computed in order to verify if the flood defence still satisfies the Flood Safety Standard. Vice versa, the maximum allowed failure probability of the glass flood defence can be deduced from the maximum probability of flooding due to a breach in the dike trajectory. The latter is done for the practical example shown below.

Practical example

A glass flood defence is realized in a dike trajectory that is part of primary flood defence, which protects the Flevopolder against flooding from the Markermeer. For this example, it is assumed that the trajectory is composed of dikes and three water retaining structures. In preparation for WTI2017 (new Flood Safety Standard), reliability requirements similar to those in WTI2017 are prescribed by OI2014. For the trajectory in this example, the maximum probability of flooding is 1/10.000 per year. Guidelines for the distribution of failure probability spaces in the dike trajectory are also given in OI2014. For water retaining hydraulic structures, the total failure probability space is equal to 8% of the maximum allowed probability of flooding. This failure probability space is further distributed over the existing structures and the new glass flood defence. As can be seen in Figure 72, this results in a maximum allowed failure probability of the glass flood defence equal to $2 \cdot 10^{-6}$ per year ($\beta = 4{,}61$).

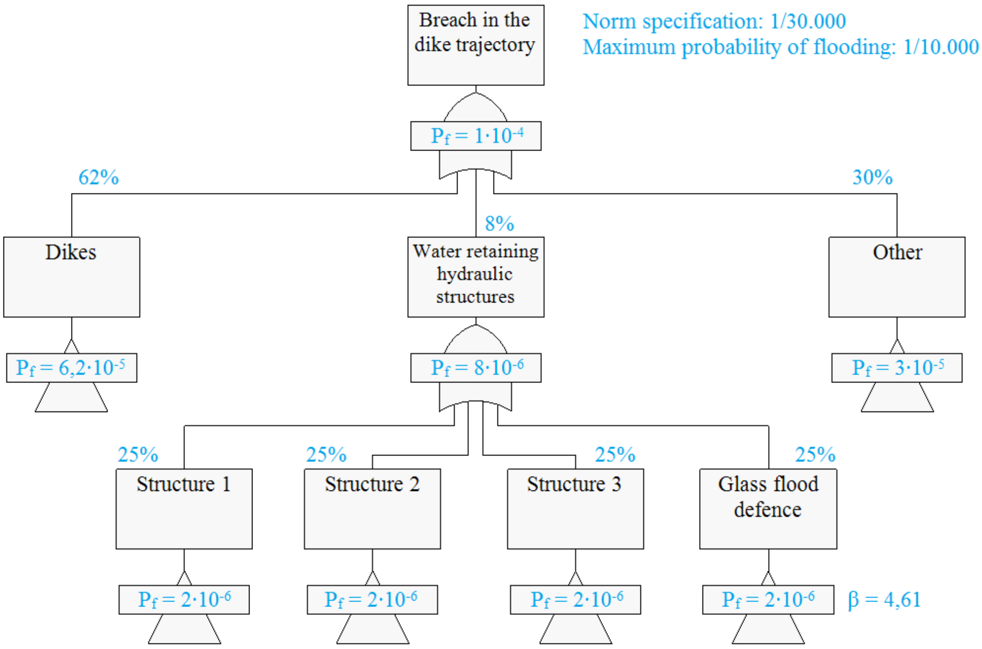


Figure 72: Failure probability space distribution of a dike trajectory in a primary flood defence

For the glass dike, a failure probability space distribution on structure level is given in section 7.3. Failure of the glass elements is assigned a failure probability space equal to 30% of the total failure probability of the structure. If the same distribution is applied to the glass flood defence in this practical example (thereby implicitly assuming a structure with similar failure mechanisms), the result is a maximum allowed yearly failure probability of $6 \cdot 10^{-7}$ ($\beta = 4{,}86$) for the glass elements. Structural elements in primary flood defences should always satisfy Eurocode reliability class 3, which corresponds to a β-value of 5,2 for a one year reference period. Hence, in this practical example, the Eurocode is governing for the reliability of the glass elements.

For both the glass dike as for the practical example shown above, the Eurocode is governing for the required reliability of the glass elements. In order to investigate at what point the Flood Safety Standard becomes governing, a number of situations is addressed. Apart from varying reliability requirements as prescribed by the Flood Safety Standard, also the number of water retaining structures in the trajectory is varied. As can be seen in Table 29, the reliability requirement for the glass elements that is deduced from the Flood Safety Standard is seldom stricter than prescribed by the Eurocode.

Norm specification (OI2014)	1/10.000			1/30.000			1/100.000		
Max. probability of flooding due to a breach in the dike trajectory	3,00E-04			1,00E-04			3,00E-05		
Relative failure probability space for structural failure (OI2014)	8%								
Number of water retaining hydraulic structures in the dike trajectory	2	4	8	2	4	8	2	4	8
Failure probability space of the glass flood defence	1,20E-05	6,00E-06	3,00E-06	4,00E-06	2,00E-06	1,00E-06	1,20E-06	6,00E-07	3,00E-07
β-value for the glass flood defence	4,22	4,38	4,53	4,47	4,61	4,75	4,72	4,86	4,99
Failure probability space of the glass elements	3,60E-06	1,80E-06	9,00E-07	1,20E-06	6,00E-07	3,00E-07	3,60E-07	1,80E-07	9,00E-08
β-value for the glass elements	4,49	4,63	4,77	4,72	4,86	4,99	4,96	5,09	5,22
β-value required by Eurocode RC3	5,2								
Eurocode governing	yes	yes	yes	yes	yes	yes	yes	yes	**no**

Table 29: Reliability requirements (1-year reference period) for glass elements in primary flood defences

9.3 Feasibility of a glass flood defence

The most important motivation to realize a glass flood defence is its aesthetically attractiveness. Although it is technically possible to design a glass flood defence with an extremely low failure probability, at some point the glass becomes so thick that its visual quality is lost. For the same reason, additional measures, like a boat collision protection or an additional row of glass (backup) elements are not desired. For the glass dike, it was found that sufficient reliability can be achieved with reasonable dimensions of the glass (approximately 5 cm thick) and reasonable additional measures.

In general, the required dimensions of the glass and additional (protective) measures that should be applied are strongly related to the reliability that should be achieved. For the glass dike, glass elements were designed to satisfy Eurocode reliability class 2. For primary flood defences, Eurocode reliability class 3 should be satisfied, for which the maximum failure probability is more than ten times smaller. As a result, the required glass thickness is much larger for a primary flood defence, even if the loads are exactly the same as for a regional flood defence. Similar to the Eurocode, reliability differentiation is also given by the Flood Safety Standard. For some primary flood defences, this leads to reliability requirements that are stricter than the highest class in the Eurocode.

Boundary conditions at the location of the flood defence also play a large role. The glass dike has to be designed for relative low loads. For most primary flood defences (and some regional flood defences), loads are much larger. Ice loads, slamming waves and wind set-up can have large consequences for the design of the glass elements. Boat collisions should be prevented at all times. Alternatively, consequences can be limited by providing a second row of water retaining elements. The magnitude of hydrostatic pressure that has to be resisted by the glass depends on the depth of the water that has the be retained. This depth can be chosen by the designer. However, since stresses in the glass increase exponentially with increasing retaining depth, the design of the glass elements is strongly influenced by this depth.

Realizing a glass flood defence is technically possible, regardless of what boundary conditions apply and no matter what reliability requirement should be satisfied. For some (primary) flood defences however, concessions need to be made for the aesthetic quality in order to achieve the desired level of safety.

10. Evaluation

10.1 Conclusions

The main goal of this research is to develop an engineering design of the glass dike that satisfies applicable safety standards. As a side product, there should be an evaluation of impediments in the engineering design process and advices on how to handle these in similar projects. These goals were achieved by answering the two main research questions:

(1) *Based on an architectural design, what engineering design of the glass dike satisfies applicable safety standards?*
(2) *What are the problems that are encountered when technical guidelines for flood defences and building codes are used to design a multifunctional glass flood defence and what advices can be given to overcome these problems?*

In order to be able to properly answer these question, a large amount of literature was reviewed. The results of this literature study are given in the answers of sub questions 1 to 3. Since the answer of the first main research question is a design of the glass dike, this answer cannot simply be given, but the main lines are given in the answers of sub questions 4 and 5. The answer to the second main research question is completely given in the answer of sub question 6. The answers to these sub questions are given below.

(1) What are the different types of flood defences? Which laws, standards and technical guidelines for the design and reliability evaluation are applicable and what future changes can be expected? In the Netherlands, each flood defence is classified as either a primary- or regional flood defence. Within these two classifications, differentiations are made in reliability requirements that are imposed by the Flood Safety Standard. At the moment, both primary- and regional flood defences must be designed to safely withstand a water level with a certain exceedance probability. For primary flood defences, the Flood Safety Standard will be changed in 2017. From that moment, these flood defences have to be designed to satisfy a prescribed maximum probability of flooding. Apart from the Flood Safety Standard, structures that are part of a flood defence system need to comply to the Eurocode, which also gives reliability requirements.

The glass dike is part of a regional flood defence. For conventional structures in regional flood defences, guidelines to achieve a reliability that satisfies the Flood Safety Standard are given in LRK2011. In some parts, this guideline also refers to the Eurocode.

(2) What method(s) can be applied to evaluate the reliability of an innovative flood defences? If the innovative technology is a new type of structural material that is part of the water retaining function, it is not known what partial factors should be applied in order to achieve the desired reliability. The solution to this problem is to perform probabilistic calculations (level II or level III) to quantify the reliability.

(3) What consideration play a role when using glass as structural elements and what knowledge is still missing? With respect to structural applications, glass is not a very attractive material. In contrast to steel and reinforced concrete, it exhibits brittle failure. Although its theoretical tensile strength is in the order of 21.000 MPa, small imperfections at the surface of the glass can cause stress concentrations larger than the theoretical strength. As a result, fracture of the glass starts at much lower stresses. This is called the practical tensile strength. The structural performance can be improved by applying heat treatments, which cause compressive pre-stresses at the surface. Additionally, a number of glass plies can be laminated to obtain more strength and to introduce some robustness. Interlayer materials in laminated glass exhibit creep and the stiffness decreases sharply with increasing temperature, hence the load bearing capacity of a laminated glass panel depends on load duration and temperature. In this research, three types of float glass were considered: Annealed-, heat-strengthened- and fully tempered glass. Annealed glass is float glass for which no heat treatment is applied. This type of glass has the lowest strength and is sensitive to stress corrosion and thermal failure. Heat-strengthened and fully tempered glass have more strength and are not sensitive to stress corrosion and thermal failure. Although fully hardened glass has the largest strength, it can fracture spontaneously due to stony inclusions. In contrast to heat-strengthened and annealed glass, fully tempered glass shatters completely once it fractures. As a result, it has no residual compressive strength after fracture. Note that residual compressive strength can only be utilized if the glass is laminated.

For structural applications of glass, a lot of knowledge is still missing. The practical tensile strength depends mainly on the quality of the production process. Pre-stresses vary per location on the glass plate, but also per direction. Furthermore, the strength is strongly influenced by the quality of the edges. As mentioned before, fracture is induced by stress concentration around imperfections at the surface. These imperfections vary in size and location. For a large glass plate, the probability of the presence of some critical damage is larger than for a small plate. As a result, the average strength of large glass plates is smaller. To date, no good statistical descriptors for the strength of glass are available.

(4) What technical additions and changes to the architectural design of the glass dike should be implemented in order to prevent failure mechanisms and/or reduce failure probabilities? In chapter 6, a first engineering design of the glass dike was proposed. For this, relevant failure mechanisms were identified, design solutions were proposed and dimensions of structural elements were determined by performing semi-probabilistic calculation. A summation of most important design changes is given below:

- Because of the weak soil, a pile foundation is applied. Since piles should be able to resist both compression and tension loads, steel tubular piles with concrete fill are applied. In order to resist the large horizontal hydrostatic pressure load, piles are placed in inclined position (raking piles).
- Piping is prevented by placing seepage screens under- and at the sides of the structure. Due to the impermeability of the soil, a length of one meter under the structure is sufficient. The width of seepage screens at the sides of the glass dike is three meters. These also function as reinforcement of the stability of adjacent dike profiles.
- The glass dike is a hollow structure. Uplift is prevented through a combination of tension piles and sufficient weight of the structure. Therefore, the floor and walls are made of concrete.
- The roof is supported by the side walls (at the transitions with the adjacent dike) and one intermediate wall. Since the span of the roof is rather large and a large thickness is undesired, it is constructed with steel HE300B beams. If for some reason the intermediate wall has failed, the roof will bend, but is still able to provide horizontal support to the water retaining glass elements.
- The top of the glass dike is situated at a height of + 0,30 m N.A.P. This is equal to the crest level of adjacent dike sections just after improvement and 0,20 m higher than minimum required by the Flood Safety Standard.

The design of hydraulic structures is strongly influenced by construction related boundary conditions. For the glass dike, some important construction related aspects are described below:

- Steel tubular piles are driven by hammering internally, thereby preventing large overpressures in the pores of the soil body of the dike.
- Part of the construction activities will be performed from the water (Noordvliet). This will not necessarily cause hindrance to boats sailing through the Noordvliet.
- Heavy building materials can only be delivered by transport over water.

It was found that the glass dike is not so different from conventional water retaining hydraulic structures that are part of a flood defence. The glass panels should be simply regarded as structural elements that fulfil part of the water retaining function. However, the glass elements cannot be designed with a semi-probabilistic method. Therefore, the design of the glass elements was not part of chapter 6.

(5) What optimizations should be implemented in order to obtain a design that satisfies applicable safety standards? In section 5.6, reliability requirements from the Eurocodes and the Flood Safety Standard were compared. An overlap was found for reliability requirements related to structural failure. However, the Eurocode gives requirements for individual failure mechanisms, while an integral maximum allowed probability of structural failure is deduced from the Flood Safety Standard. The glass dike must satisfy both.

Safety standard	Part of the structure	Reliability requirement ($p_{f;max}$)
Eurocode (RC2)	Structural elements / failure mechanisms	$1{,}3 \cdot 10^{-6}$/year
Flood Safety Standard	Total structure	$1{,}0 \cdot 10^{-4}$/year

Table 30: Reliability requirements for structural failure of the glass dike

The reliability of the total structure (system) of the glass dike was evaluated in chapter 7. Instead of computing all failure probabilities for the failure mechanisms included in the fault tree, conservative estimates were made. It appeared that, with these ‘hypothetical failure probabilities’, the glass dike satisfies the Flood Safety Standard. Moreover, considerable failure probability space is left over for the glass elements. However, the maximum allowed failure probability that follows from the Eurocode is more strict.

Safety standard	Reliability requirement for the glass elements ($p_{f,max}$)
Eurocode (RC2)	$1{,}3 \cdot 10^{-6}$/year
Flood Safety Standard	$5{,}0 \cdot 10^{-5}$/year

Table 31: Reliability requirements for the glass elements

For the glass elements, four failure mechanisms were taken into account: Overloading, impact, explosions and fire. A failure probability space of $3{,}25{\cdot}10^{-7}$ per year was assigned to each failure mechanism. A probabilistic model was proposed for overloading of the glass elements. A sufficient low probability of failure due to overloading was found for the composition of glass elements as shown in Table 34.

Glass type	Heat-strengthened
Number of glass plies	4 per element
Thickness per ply	12 mm
Type of interlayer material	SentryGlas
Total thickness of a glass element	52.56 mm
Failure probability for overloading	$2{\cdot}10^{-7}$ per year

Table 32: Composition of a glass element

Additional to the composition as shown in Table 32, a thin ply of fully tempered glass is attached to the glass elements at the water side in order to protect it from small impacts. For impact, explosions and fire, no probabilistic calculations are performed as only very complex models can provide answers with reasonable accuracy. However, a deterministic calculation was performed for impact due to a boat collision. It was found that a second row of glass elements that can take over the water retaining function, in case the first row of glass elements has failed, is required. Moreover, it is desired to prevent boat collisions by means of a protective structure in front of the glass dike. For explosions, fire and impact due to vandalism, a qualitative analysis is performed. Again, it is found that a second row of glass elements is required. Since glass is a poor structural material with respect to fire, measures to prevent this and to reduce consequences are desired. More research should be performed on this subject.

(6) Based on the glass dike, what considerations play an important role when designing- and evaluating the reliability of glass flood defences in general? The lessons that were learned from the case study about the glass dike were used to say something about the considerations that play a role for glass flood defences in general. A summation of the most important conclusions is shown below:

- The required thickness of glass elements is strongly related to reliability requirements that follow from the Flood Safety Standard and Eurocodes.
- If a regional flood defence is multifunctional in the sense that it is accessible to people, it is required to apply a higher Eurocode reliability class than prescribed by LRK2011. Primary flood defences must always satisfy the highest Eurocode reliability class (RC3).
- Reliability requirements that follow from the Eurocode are almost always governing for the design of the glass elements. As a result, possibilities to adjust failure probability spaces and to optimize the design with respect to the Flood Safety Standard are limited.
- The failure probability for overloading of glass elements is strongly related to the size of the loaded surface area. Therefore, the so called ‘length effect’ is very large for glass flood defences.
- Ship collisions should be prevented by means of a protective structure in front of the glass panels.
- Alternatively, a second row of (glass) elements (or second flood defence) that can take over the water retaining function should be provided. With this solution, failure probabilities due to overloading, explosions and fire will be reduced as well.

10.2 Discussion

In this section, some comments are given on problems that were encountered and choices that were made, which still require attention or for which care should be taken. These comments are not sorted by relevance.

- It is often argued that applying Eurocode partial factors on a water level with an already very low exceedance probability gives a too conservative design of the flood defence. For primary flood defences, this problem is solved as soon as the new Flood Safety Standard takes effect. This does not hold for regional flood defences. However, since prescribed exceedance probabilities of the water level are lower for regional flood defences, these are less over dimensioned when Eurocode partial factors are applied.

- For failure of water retaining structures in regional flood defences, a maximum allowed probability, given that the design water level is not exceeded, is prescribed. The note 'given that the design water level is not exceeded' is not very logical. This implies that, in probabilistic calculations, the probability density- and distribution functions of the water level should be cut off at the design water level. It is safer and more practical, to also take into account water levels higher than the design water level. Therefore, this has been done in this thesis.

- In chapter 7, it was determined that Eurocode reliability requirements are governing for the design of the glass elements. As a result, possibilities to adjust (increase) failure probability spaces and to optimize the design with respect to the Flood Safety Standard are limited.

- In this thesis, a probabilistic calculation is performed to verify if the glass elements (together) satisfy the minimum reliability as prescribed by Eurocode reliability class 2. However, since it is allowed to apply this requirement on the level of structural elements, it is officially allowed to apply this requirement to individual glass elements. This way, the length effect is not taken into account. Furthermore, the Eurocode reliability requirement becomes less strict than the required reliability that follows from the Flood Safety Standard. This gives more room to optimize failure probability spaces with respect to the integral failure probability of the structure as a whole. For this integral failure probability, length effects are taken into account.

- In probabilistic calculations of steel structural elements, failure is assumed if the steel starts to yield. In reality, it still has some residual load bearing capacity, thereby introducing robustness in the design. In fact, the Eurocode gives requirements for the robustness of structures. One of these requirements is that alternative load paths should be provided. On the level of a glass elements, this is done by laminating a number of glass plies together. If fracture occurs in one glass ply, the other plies can take over the load. In this thesis, failure was assumed if only two out of four plies have fractured. If failure is only assumed for fracture of all glass plies in an element, there is no residual strength, leading to a 'real' reliability that is lower than steel structural elements with an equal value for the computed failure probability.

- A Weibull distribution is applied to generate values for the strength of glass. For this distribution, a very conservative value was used for the variance. In the probabilistic calculation, fracture occurred at tensile stresses in the order of 5 MPa, while the average tensile strength of heat-strengthened glass is approximately 104 MPa. Although the strength of glass has a large spread, it should be questioned if above mentioned value for the fracture stress is realistic.

- Under normal conditions, the glass elements are only subjected to hydraulic loads. Impacts, fire and explosions are all incidental loads. In chapter 8, the maximum allowed failure probability of the glass elements that follows from the Eurocode is distributed over the failure mechanisms that apply to the glass, which also includes these incidents. This may give a too conservative (small) failure probability space for overloading. In practice, maximum failure probabilities that follow from the Eurocode are not distributed over different types of failure, but are applied to each failure mechanism separately. With respect to the Flood Safety Standard, it is common practice to distribute failure spaces. However, the maximum allowed failure probability of the glass elements that follows from the Flood Safety Standard is much larger.

10.3 Recommendations

In order to realize the glass dike or a glass flood defence in general, a broad spectrum of subjects need to be considered. In this master thesis, the engineering design and reliability are addressed. This section lists a number of recommendations for the glass dike in particular and for further research related to glass flood defences in general. Note that these recommendations are not sorted by relevance.

Glass dike

- The foundation of the glass dike was dimensioned by making use of very conservative soil parameters. More soil investigations should be performed in order to be able to optimize the design of the pile foundation.
- The engineering design proposed in this thesis, is mainly focussed on achieving a certain structural reliability. Other technical aspects like functionality, sustainability and safety during construction are only partly taken into account. These aspects should be addressed more elaborately.
- Non-technical challenges that are very important to address (governance, juridical, financial and spatial) are summarized in section 2.1, but only a small part is considered in this thesis. These subjects require more attention.
- Periodic assessment of the glass elements is briefly discussed in this thesis. However, a plan for more frequent inspections, maintenance and monitoring should be made as well.
- An important question that should be answered is: What should happen with the glass dike, in case the Flood Safety Standard is changed, such that the glass dike does not satisfy its requirements anymore? Is it possible to adjust the glass dike?
- For the glass elements, failure due to incidents (like a boat collision) should be prevented when possible. If prevention is not possible (like fire), the threat should be reduced to a minimum. At first instance, a small number of tests with laminated glass elements can be performed to verify if the threats are relevant at all.
- Glass elements should be designed such, that a negative assessment judgement is prevented. This can be done by taking into account a lifetime of 100 years in glass strength calculations and by providing protection layers on the glass elements to prevent damage in load bearing glass plies.

Probabilistic reliability evaluation of a glass flood defence

- Because of interactions between the soil and hard structures, the reliability (stability) of a dike is influenced by the presence of a glass flood defence. This influence can be quantified by combining a finite element model for the stability of the dike with a probabilistic method.
- The effect of load duration and temperature on properties of the interlayer material was taken into account by means of a conservative estimate of the amount of bonding. It would be more accurate to include the full relation between bonding, load duration and temperature.
- In the probabilistic calculation for overloading of the glass elements, a large number of conservative simplifications and assumptions is made. Since the strength of glass is largely location depended, a good alternative approach to compute the failure probability is to combine a finite element model with a probabilistic method. However, more research should also be performed on statistical descriptors for compressive pre-stresses in pre-stressed float glass.
- In the probabilistic model for overloading, only hydrostatic pressure is taken into account. For other flood defences, it might be needed to also include loads from (slamming) waves.
- Threats of incidents like fire, explosions and impact should be quantified with a failure probability. The simplest way is to estimate the probability of occurrence of these incidents. If less conservative values of the failure probability are desired, probabilistic models should be made in which the resistance of the glass elements against these incidental loads is included.
- The need for good statistical descriptors for the strength of glass was stressed a couple of times in this report. At the moment, many researches are carried out to obtain these statistical properties. This should be continued.

Bibliography

Bardoel, J., & Leemans, P. (2013). *Geotechnische onderbouwing kadeverbeteringen Duifpolder.* Oranjewoud. Delft: Hoogheemraadschap van Delfland.

Böhtlingk, E., & Robinson, J. (2015). *Realisatie Glazen Dijk*. Maasland.

Bos, F. P. (2007). *Towards a combined probabilistic/consequence-based safety approach of structural glass members.* HERON-ENGLISH EDITION.

Button , D., Colvin, J., Cunliffe Joan, Inman , C., Jackson, K., Lightfoot, G., . . . Waldron, B. (1993). *Glass in building: A guide to modern architectural glass performance.* Butterworth Architecture.

Centrum voor Onderzoek Waterkeringen. (n.d.). *Onderzoek naar de veiligheid van de boezemkade van de Duifpolder.*

CUR-publicatie 166. (1997). *Damwandconstructies.* Gouda: Stichting CUR.

CUR-publicatie 190. (1997). *Probability in Civil Engineering, PART 1: The theory of probabilistic design.* Gouda: Stichting CUR.

De Architect. (2014, 05 06). Glazen dijk voor midden-Delfland. Retrieved June 2015, from http://www.dearchitect.nl/nieuws/2014/05/02/glazen-dijk-midden-delfland/glazen-dijk-midden-delfland.html

De Groot, I. (2015). *Projectplan natte kunstwerken voor de toekomst.* Deltares,TNO,MARIN.

Deltaprogram. (2014). Ministerie van Infrastructuur en Milieu & Ministerie van Economische Zaken.

Don, K., & Karunarathna, L. (2013). *Low-velocity impact analysis of monolithic and laminated glass using finite element method (FEM).* University of Birmingham.

Doulkari, K. (2013). *The Transparent Facade of the Future.* Delft: TU Delft.

Ellen, G., Tromp, E., Borgers , H., Rengers , J., Boers, M., Knoeff, H., . . . Wessels, J. (2011). *Multifunctioneel gebruik van de waterkering.* Deltares.

European committe for Standardization. (2011). *NEN-EN 1992-1-1+C2 - Design of concrete structures.*

European Committee for Standardization. (2003). *NEN-EN 1991-1-3+C1 - General Actions - Snow loads.*

European Committee for Standardization. (2011). *NEN-EN 1990+A1+A1/C2 - Basis of structural design.*

European Committee for standardization. (2011). *NEN-EN 1991-1-1+C1 - General actions -Densities, self-weight, imposed loads for buildings.*

European Committee for Standardization. (2011). *NEN-EN 1991-2+C1 - Traffic loads on bridges.*

European Committee for Standardization. (2012). *NEN-EN 1997-1+C1 - Geotechnical design - general rules.*

Ford, M. (2001, January 22). *How is tempered glass made?* . Retrieved 2015, from SCIENTIFIC AMERICAN: http://www.scientificamerican.com/article/how-is-tempered-glass-mad/

Fröling, M. (2013). *Strength design methods for laminated glass.* Lund University, Department of Construction Sciences, Structural Mechanics.

Haldimann, M., Luible, A., & Overend, M. (2008). *structural use of glass* (Vol. 10). Zurich, Switzerland: IABSE.

Het recreatieschap Midden-Delfland. (2015). *Vaarnetwerk Midden-Delfland en Westland.*

Housing Act. (1991, 08 29). Retrieved June 2015, from http://wetten.overheid.nl/BWBR0005181/

IBS . (2016). *Glass walls for flood defence*. Retrieved March 22, 2016, from http://www.ibsengineeredproducts.co.uk/en/Flood-Protection-Systems/Glass-Walls.php

Inspectie Verkeer en Waterstaat. (2011). *Derde toets primaire waterkeringen.*

Jansen, H., Janssen, G., & Muskens, M. (2005). *Jellema deel 2 - Onderbouw.* Utrecht/Zutphen: ThiemeMeulenhoff.

JRC EC. (2015). *EN Eurocode Parts.* Retrieved June 2015, from http://eurocodes.jrc.ec.europa.eu/showpage.php?id=13

Kasper, A. (1999). Advances in Testing Tempered Glasses in Heat Soak Ovens. *Glass Processing Days 1999*, 71-75.

Klijn, F., & Bos, M. (2010). *Deltadijken: ruimtelijke implicaties. Effecten en kansen van het doorbraakvrij maken van primaire waterkeringen.* Deltares, Delft.

Knoeff, H., van der Meer, M., van nieuwenhuijzen, L., Tromp, E., Woning, M., Leeuwdrent, W., & Bizzarri, A. (2013). *Handreiking Innovaties Waterkeringen.* Deltares.

Knoeff, H., Van der Meer, M., Van Nieuwenhuijzen, L., Tromp, E., Woning, M., Leeuwdrent, W., & Bizzarri, A. (2013). *Handreiking Innovaties Waterkeringen.* Delft: Deltares.

Kogelvrij.nl. (2016). *Kogelwerend glas*. Retrieved March 22, 2016, from http://www.kogelvrij.nl/Producten/Kogelwerend_glas.html

Kuraray. (2015). *Sentryglas Data Sheet.* Opgeroepen op Januari 2016, van sentryglas.com.

Le Bourhis, E. (2008). *Glass-Mechanics and Technology.* Weinheim, Germany: Wiley-VHC.

Legger Regionale Waterkeringen. (2015). Retrieved July 2015, from http://hhdelfland.maps.arcgis.com/

Pilkington. (2016). *Step into the void*. Retrieved March 22, 2016, from http://projects.pilkington.com/Netherlands/show/5669/Step-into-the-void.aspx

Rahman, S. (2001). Probabilistic fracture mechanics: J-estimation and finite element methods. *Engineering Fracture Mechanics, 68(1)*, 107-125.

Rijksoverheid. (2015b). *WTI2017 in ontwikkeling*. Retrieved 06 09, 2015, from Helpdesk Water: www.helpdeskwater.nl

Rijkswaterstaat. (2013). *Ruimte voor innovatie.* Corporate innovatieprogramma Rijkswaterstaat.

Schiereck, G., & Verhagen, H. (2012). *Introduction to bed, bank and shore protection 2nd edition.* Delft: VSSD.

Staaltabellen.nl. (2015). Retrieved August 2015, from www.staaltabellen.nl/balkstaal

STOWA. (2011). *LRK2011 - Leidraad waterkerende kunstwerken in regionale waterkeringen.* Amersfoort.

TAW. (1998). *Grondslagen voor Waterkeren.* Rotterdam.

TAW. (1999). *Technische Rapport Zandmeevoerende Wellen.* Delft.

TAW. (2001). *Technisch Rapport Waterkerende Grondconstructies.* Delft.

TAW. (2003). *LK2003 - Leidraad Kunstwerken.* Delft.

Veer, F., & Rodicheff, y. (not yet published). Variations in pre-stress in annealed, heat strengthened and fully tempered glass.

Veer, F., & Rodichev, Y. (2011). The strength of water jet cut glass. *Glass performance days 2011*, 434-438.

Veer, F., Louter, P., & Bos, F. (2009). The strength of annealed, heat-strenghtened and fully tempered float glass. *Fatigue & Fracture of Engineering Materials & Structures, 32*, 18-25.

Volker Staal en Funderingen. (2015). *Palen*. Opgeroepen op October 2015, van http://www.vsf.nl/nl/funderingen/palen/

Vrijling, J., Bezuyen, K., Kuijper, H., Molenaar, W., Van Baars, S., & Voorendt, M. (2015). *Manual Hydraulic Structures.* Delft: Delft University of Technology.

Vroom Funderingstechnieken. (2015). *Products*. Opgeroepen op October 2015, van http://www.vroom.nl/nl/products/

Wabo. (2008, 11 06). Retrieved June 2015, from http://wetten.overheid.nl/BWBR0024779/

Water Act. (2009, 01 29). Retrieved June 2015, from http://wetten.overheid.nl/BWBR0025458/

Waterverordening Zuid-Holland. (2015, 02 27). Retrieved July 2015, from http://decentrale.regelgeving.overheid.nl/

Weijers, J., & Tonneijck, M. (2009). *Lecture notes CT5314 - Flood Defences.* Delft: TU Delft.

Wikipedia. (2016). *Georgia Aquarium*. Opgeroepen op March 22, 2016, van https://en.wikipedia.org/wiki/Georgia_Aquarium

Xu, J., Li, Y., Lu, G., & Zhou, W. (2009). Reconstruction model of vehicle impact speed in pedestrian-vehicle accident. *International Journal of Impact Engineering, 36(6)*, 783-788.

Appendix A: Boundary conditions

Appendix A.1: Hydraulic boundary conditions

The Noordvliet is a polder discharge channel with a controllable water level. This means that the governing water level not only depends on external influences like precipitation and wind set-up, but also on water management strategies. In fact, the governing water level is determined by the water board. The same holds for the average water level and the water levels inside the Duifpolder. The low water level is not known, but a conservative assumption is made. These water levels are shown below (Bardoel & Leemans, 2013):

Water levels Noordvliet

Governing high water level:	N.A.P. -0,03 m
Average water level:	N.A.P. -0,43 m
Low water level:	N.A.P. -0,63 m

Water levels in the Duifpolder

Average water level:	N.A.P. -3,15 m
High water level:	N.A.P. -3,00 m

The groundwater levels inside the Duifpolder are assumed to be equal to the water levels in the polder channels. This assumption has no consequences for the design of the glass dike, since the groundwater level in and near the dike (phreatic line) depends on the water level in the Noordvliet and the small channel behind the dike.

The height of wind waves is determined by the fetch length, water depth, wind velocity and duration of the wind. Since the Noordvliet is a relative small and shallow channel, no significant wind driven waves are expected. Therefore, a maximum significant wave height of 0,10 m is taken into account for wind waves. The guideline 'Water retaining hydraulic structures in regional flood defences (2011)' gives tables for the wave height at the structure, based on calculations with the formula of Brettschneider. At wind speeds higher than 20 m/s, the wave heights can become slightly larger than 0,10 m (Figure 73). If the waves are directed at the structure depends on the wind direction. The combination of high wind velocity and wind direction towards the glass dike is not likely to occur. Especially not in combination with very high water levels. For this reason, the governing wind wave height is chosen to be not larger than 0,10 m.

Waves can also be induced by boats sailing through the Noordvliet. A distinction is made between primary waves and secondary waves. Primary waves can be described as a temporary depression and raise of the water level. Primary waves are the largest for larger boats. However, at extreme high water levels, large boats cannot sail on the Noordvliet due to the limited clearance height of bridges near the Noordvliet. Therefore, primary waves are not regarded. Secondary waves are the largest for small (fast) boats. These waves always approach the flood defence under an angle, so pressures are only locally increased. Therefore, secondary waves can be neglected for stability calculations. For the strength, boat collision impacts are expected to be dominant. For this reason secondary waves are also not regarded for the first design of the glass dike.

Currents can be characterized by their cause. The most important type of current is the average flow velocity related to the water discharge function of the channel. Boats also induce currents. These are return currents and propeller wash. Return currents are caused by the water displacement of the boat, while the propeller wash is directly caused by the propeller of the boat. The average current and return currents are expected to be unimportant for the design of the glass dike. The propeller wash can be relevant if boats come to a stop very near the glass dike and then start sailing again. In that case, a bottom protection should be applied.

Wind snelheid	Water diepte	Breedte water	Golfhoogte bij kunstwerk	Golf periode	Overslag hoogte Q = 0,1l	Overslag hoogte Q = 10l
[m/s]	[m]	[m]	[m]	[s]	[m]	[m]
24	7,5	10	0,10	0,91	0,20	0,01
24	7,5	15	0,12	1,00	0,24	0,02
24	7,5	20	0,13	1,08	0,28	0,03
24	7,5	25	0,14	1,14	0,31	0,04
24	7,5	50	0,19	1,35	0,45	0,08
24	7,5	75	0,23	1,49	0,55	0,11
24	7,5	100	0,26	1,59	0,63	0,14
24	7,5	250	0,38	1,98	1,00	0,28
24	7,5	500	0,50	2,33	1,39	0,44
24	7,5	1000	0,65	2,72	1,91	0,67
24	7,5	2500	0,92	3,32	2,84	1,09
24	7,5	5000	1,15	3,81	3,70	1,51
24	10	10	0,10	0,91	0,20	0,01
24	10	15	0,12	1,00	0,24	0,02
24	10	20	0,13	1,08	0,28	0,03
24	10	25	0,14	1,14	0,31	0,04
24	10	50	0,19	1,35	0,45	0,08
24	10	75	0,23	1,49	0,55	0,11
24	10	100	0,26	1,60	0,64	0,14
24	10	250	0,38	1,99	1,00	0,28
24	10	500	0,50	2,34	1,40	0,45
24	10	1000	0,66	2,75	1,95	0,68
24	10	2500	0,95	3,37	2,94	1,14
24	10	5000	1,21	3,89	3,91	1,61
22	2	10	0,09	0,86	0,17	0,00
22	2	15	0,10	0,95	0,21	0,01
22	2	20	0,12	1,02	0,24	0,02
22	2	25	0,13	1,07	0,27	0,03
22	2	50	0,17	1,26	0,39	0,06
22	2	75	0,20	1,39	0,47	0,09
22	2	100	0,23	1,48	0,54	0,11
22	2	250	0,32	1,82	0,81	0,21
22	2	500	0,40	2,11	1,07	0,31
22	2	1000	0,49	2,42	1,35	0,43
22	2	2500	0,59	2,84	1,68	0,57
22	2	5000	0,63	3,15	1,84	0,64
22	3	10	0,09	0,86	0,17	0,00
22	3	15	0,11	0,95	0,21	0,01
22	3	20	0,12	1,02	0,25	0,02
22	3	25	0,13	1,08	0,28	0,03
22	3	50	0,17	1,27	0,39	0,06
22	3	75	0,20	1,40	0,48	0,09
22	3	100	0,23	1,50	0,55	0,11
22	3	250	0,33	1,85	0,85	0,22
22	3	500	0,43	2,16	1,15	0,34
22	3	1000	0,54	2,49	1,52	0,50
22	3	2500	0,69	2,97	2,04	0,72
22	3	5000	0,79	3,34	2,38	0,88

Figure 73: Wind wave heights (LRK2011)

Appendix A.2: Geo-technical boundary conditions

In 2008, soil investigations were performed for the soil body at the desired location of the glass dike. By means of cone penetration tests and taking samples, the parameters of the soil were determined for the locations given in Figure 74. The corresponding results are given in the following figures.

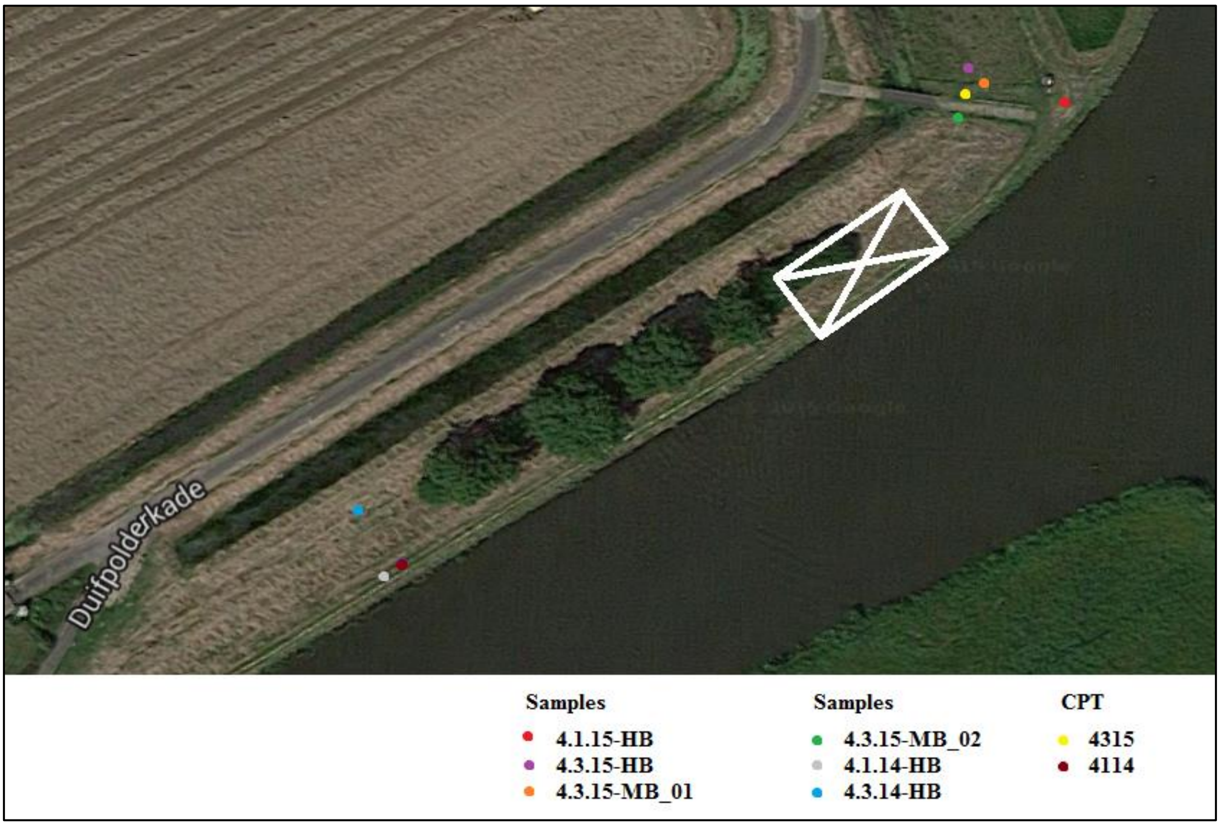
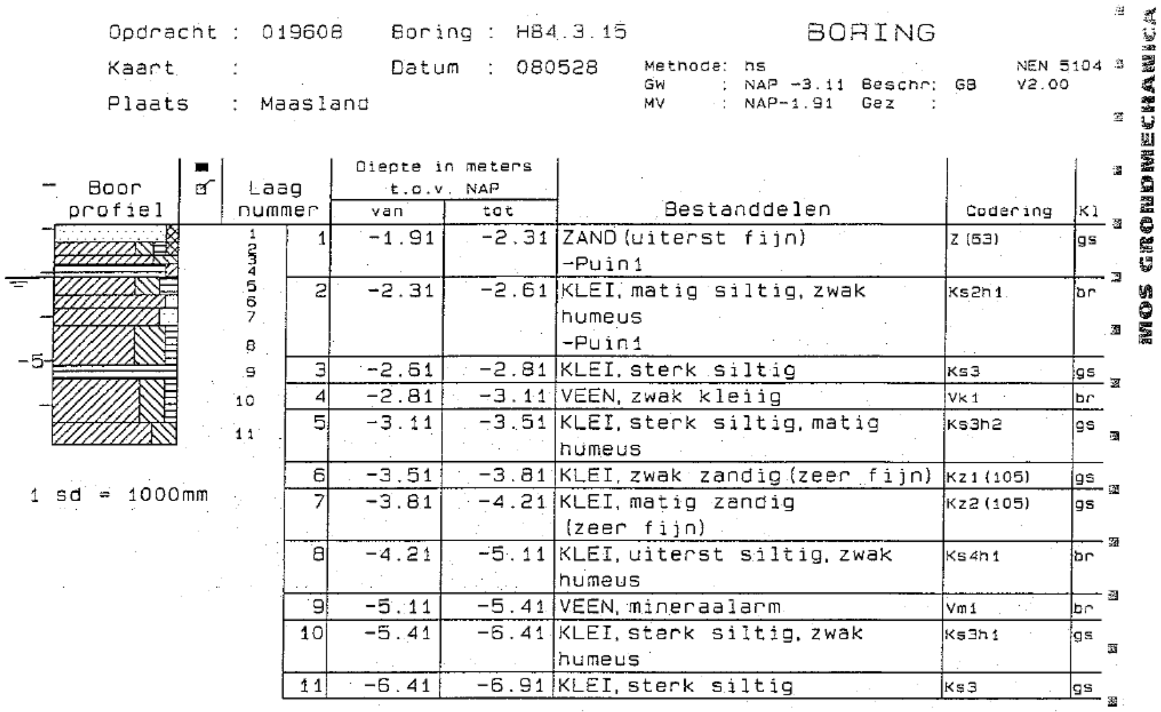


Figure 74: Locations of soil investigation



Opdracht : 019608 Boring : HB4.3.15 BORING

Kaart : Datum : 080528 Methode: hs NEN 5104

GW : NAP -3.11 Beschr: GB V2.00

Plaats : Maasland MV : NAP-1.91 Gez :

Laag nummer		Diepte in meters t.o.v. NAP van	tot	Bestanddelen	Codering	Kl
1 2 3 4	1	-1.91	-2.31	ZAND (uiterst fijn) -Puin1	Z (63)	gs
5 6 7 8	2	-2.31	-2.61	KLEI, matig siltig, zwak humeus -Puin1	Ks2h1	br
9	3	-2.61	-2.81	KLEI, sterk siltig	Ks3	gs
10	4	-2.81	-3.11	VEEN, zwak kleiig	Vk1	br
11	5	-3.11	-3.51	KLEI, sterk siltig, matig humeus	Ks3h2	gs
	6	-3.51	-3.81	KLEI, zwak zandig (zeer fijn)	Kz1 (105)	gs
	7	-3.81	-4.21	KLEI, matig zandig (zeer fijn)	Kz2 (105)	gs
	8	-4.21	-5.11	KLEI, uiterst siltig, zwak humeus	Ks4h1	br
	9	-5.11	-5.41	VEEN, mineraalarm	Vm1	br
	10	-5.41	-6.41	KLEI, sterk siltig, zwak humeus	Ks3h1	gs
	11	-6.41	-6.91	KLEI, sterk siltig	Ks3	gs

Figure 75: Soil sample 4.3.15-HB

Opdracht : 019608 Boring : M84.3.15 BORING
Kaart : Datum : 080618 Methode: Puls NEN 5104
GW : NAP -3.06 Beschr: V2.00
Plaats : Maasland MV : NAP-1.98 Gez :

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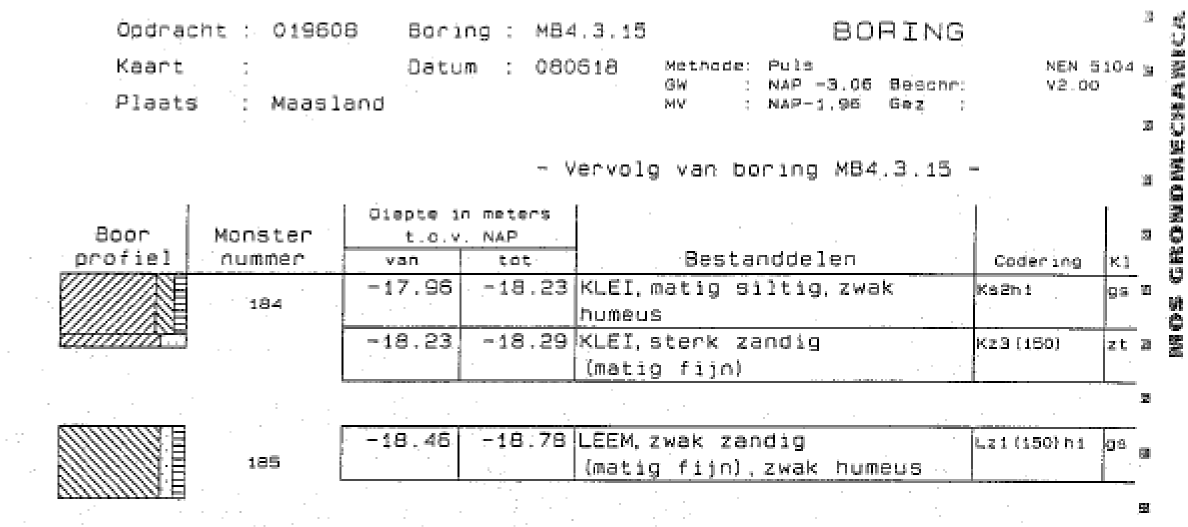
- Vervolg van boring M84.3.15 -

Boor profiel	Monster nummer	Diepte in meters t.o.v. NAP van	tot	Bestanddelen	Codering	Kl
	174	-6.46	-6.86	KLEI, uiterst siltig	Ks4	gs
	175	-7.46	-7.61	KLEI, sterk siltig, matig humeus	Ks3h2	br
		-7.61	-7.86	KLEI, uiterst siltig, zwak humeus	Ks4h1	gs
	176	-8.46	-8.80	KLEI, sterk siltig	Ks3	gs
	177	-9.46	-9.60	KLEI, matig siltig, matig humeus	Ks2h2	gs
		-9.60	-9.77	VEEN, mineraalarm	Vm1	br
	178	-9.96	-10.04	VEEN, mineraalarm	Vm1	br
		-10.04	-10.32	KLEI, matig siltig, matig humeus	Ks2h2	gs
	179	-10.96	-11.31	KLEI, sterk siltig, zwak humeus	Ks3h1	gs
	180	-11.96	-12.35	KLEI, sterk siltig, zwak humeus	Ks3h1	gs
	181	-13.46	-13.82	KLEI, zwak zandig (zeer fijn)	Kz1 (105)	gs
	182	-14.96	-15.28	KLEI, matig siltig -Schelpen1	Ks2	gs
	183	-16.46	-16.76	KLEI, sterk siltig	Ks3	gs
		-16.76	-16.86	KLEI, matig siltig, zwak humeus	Ks2h1	gs

Figure 76: Soil sample 4.3.15-MB_01



Opdracht : 019608 Boring : MB4.3.15 **BORING**

Kaart : Datum : 080618 Methode: Puls NEN 5104

GW : NAP -3.06 Beschr: V2.00

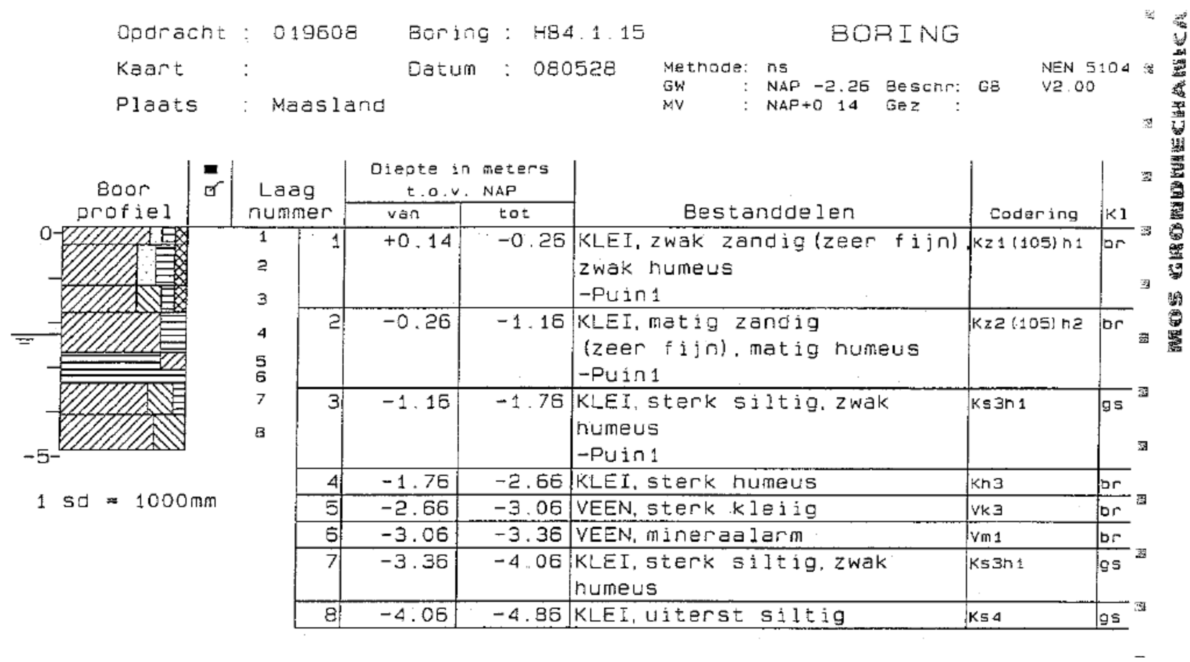
Plaats : Maasland MV : NAP-1.96 Gez :

- Vervolg van boring MB4.3.15 -

Boor profiel	Monster nummer	Diepte in meters t.o.v. NAP van	tot	Bestanddelen	Codering	Kl
	184	-17.96	-18.23	KLEI, matig siltig, zwak humeus	Ks2h1	gs
		-18.23	-18.29	KLEI, sterk zandig (matig fijn)	Kz3(150)	zt
	185	-18.46	-18.78	LEEM, zwak zandig (matig fijn), zwak humeus	Lz1(150)h1	gs

MOS GRONDMECHANICA

Figure 77: Soil sample 4.3.15-MB_02



Opdracht : 019608 Boring : HB4.1.15 **BORING**

Kaart : Datum : 080528 Methode: ns NEN 5104

GW : NAP -2.26 Beschr: GB V2.00

Plaats : Maasland MV : NAP+0 14 Gez :

0 -5 1 sd = 1000mm

Boor profiel	Laag nummer		Diepte in meters t.o.v. NAP van	tot	Bestanddelen	Codering	Kl
	1 2 3	1	+0.14	-0.25	KLEI, zwak zandig (zeer fijn) zwak humeus -Puin1	Kz1(105)h1	br
	4 5 6	2	-0.26	-1.16	KLEI, matig zandig (zeer fijn), matig humeus -Puin1	Kz2(105)h2	br
	7 8	3	-1.16	-1.76	KLEI, sterk siltig, zwak humeus -Puin1	Ks3h1	gs
		4	-1.76	-2.66	KLEI, sterk humeus	Kh3	br
		5	-2.66	-3.06	VEEN, sterk kleiig	Vk3	br
		6	-3.06	-3.36	VEEN, mineraalarm	Vm1	br
		7	-3.36	-4.06	KLEI, sterk siltig, zwak humeus	Ks3h1	gs
		8	-4.06	-4.86	KLEI, uiterst siltig	Ks4	gs

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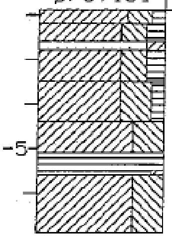
Figure 78: Soil sample 4.1.15-HB

Opdracht : 019608 Boring : HB4.3.14 BORING

Kaart : Datum : 080528 Methode: hs NEN 5104

Plaats : Maasland GW : Beschr: GB V2.00

MV : NAP-1.89 Gez :



1 sd = 1000mm

Laag nummer	Diepte in meters t.o.v. NAP van	tot	Bestanddelen	Codering	Kl
1	-1.89	-2.19	KLEI, sterk siltig, zwak humeus	Ks3h1	br
2	-2.19	-2.59	KLEI, sterk siltig, matig humeus	Ks3h2	br
3	-2.59	-2.79	ZAND (zeer fijn), matig kleiig	Z(105)k2	gs
4	-2.79	-3.49	KLEI, sterk siltig, matig humeus	Ks3h2	gs
5	-3.49	-4.39	KLEI, uiterst siltig, zwak humeus	Ks4h1	gs
6	-4.39	-5.09	KLEI, uiterst siltig	Ks4	gs
7	-5.09	-5.49	VEEN, mineraalarm	Vm1	br
8	-5.49	-5.59	VEEN, zwak kleiig	Vk1	br
9	-5.59	-6.89	KLEI, uiterst siltig	Ks4	gs

MOS GRONDMECHANICA

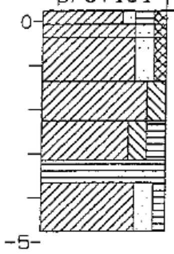
Figure 79: Soil sample 4.3.14-HB

Opdracht : 019608 Boring : HB4.1 14 BORING

Kaart : Datum : 080527 Methode: hs NEN 5104

Plaats : Maasland GW : Beschr: GB V2.00

MV : NAP+0 25 Gez :



1 sd = 1000mm

Laag nummer	Diepte in meters t.o.v. NAP van	tot	Bestanddelen	Codering	Kl
1	+0.25	-0.05	KLEI, zwak zandig (zeer fijn) matig humeus -Puin1	Kz1(105)h2	br
2	-0.05	-0.35	KLEI, matig zandig (zeer fijn) -Puin1	Kz2(105)	br
3	-0.35	-1.35	KLEI, matig zandig (zeer fijn) -Puin1	Kz2(105)	br
4	-1.35	-2.25	KLEI, matig siltig	Ks2	gs
5	-2.25	-3.15	KLEI, matig siltig, matig humeus	Ks2h2	br
6	-3.15	-3.65	VEEN, mineraalarm	Vm1	br
7	-3.65	-4.75	KLEI, matig zandig (zeer fijn), zwak humeus	Kz2(105)h1	gs

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Figure 80: Soil sample 4.1.14-HB

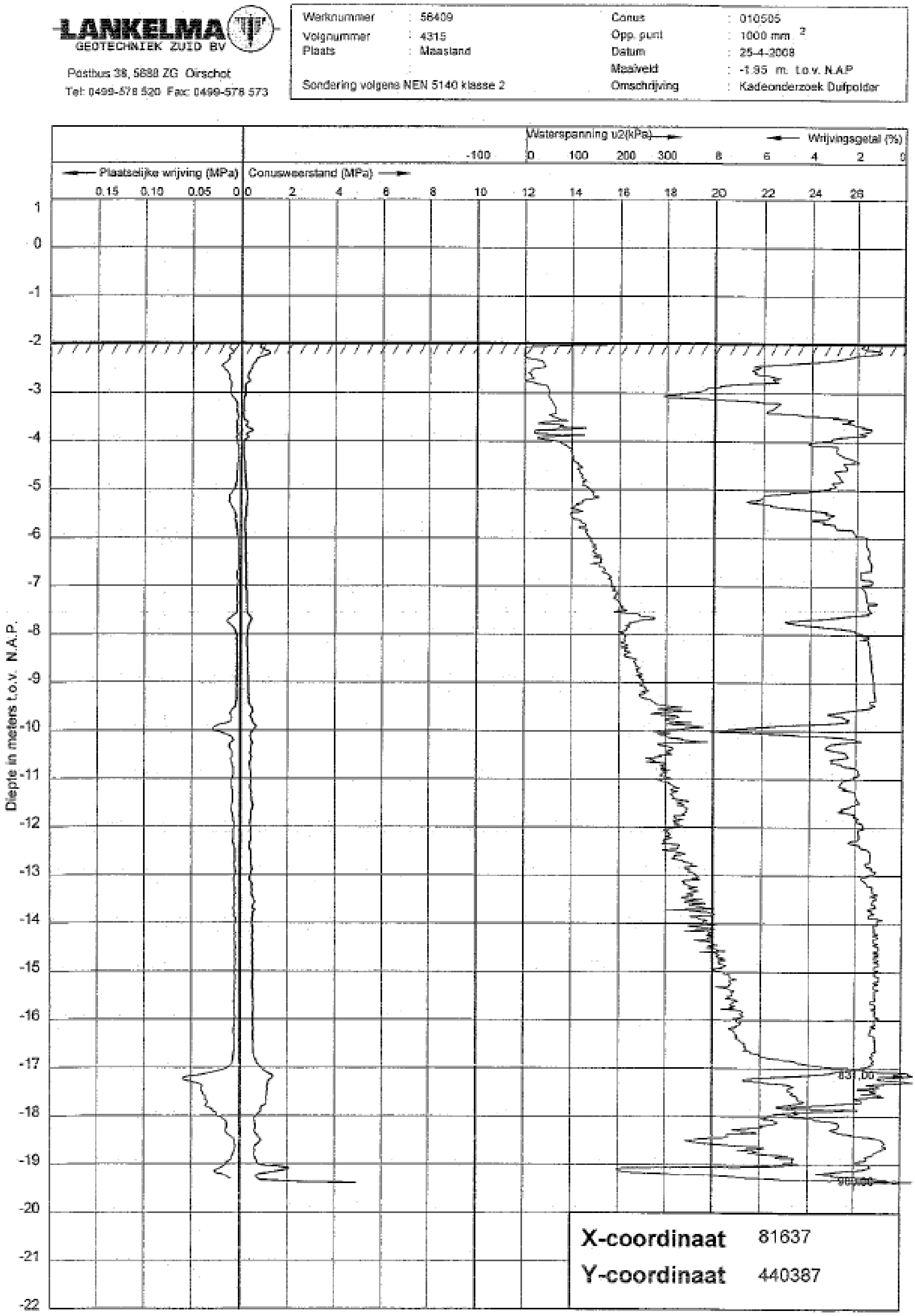


Figure 81: Cone penetration test 4315

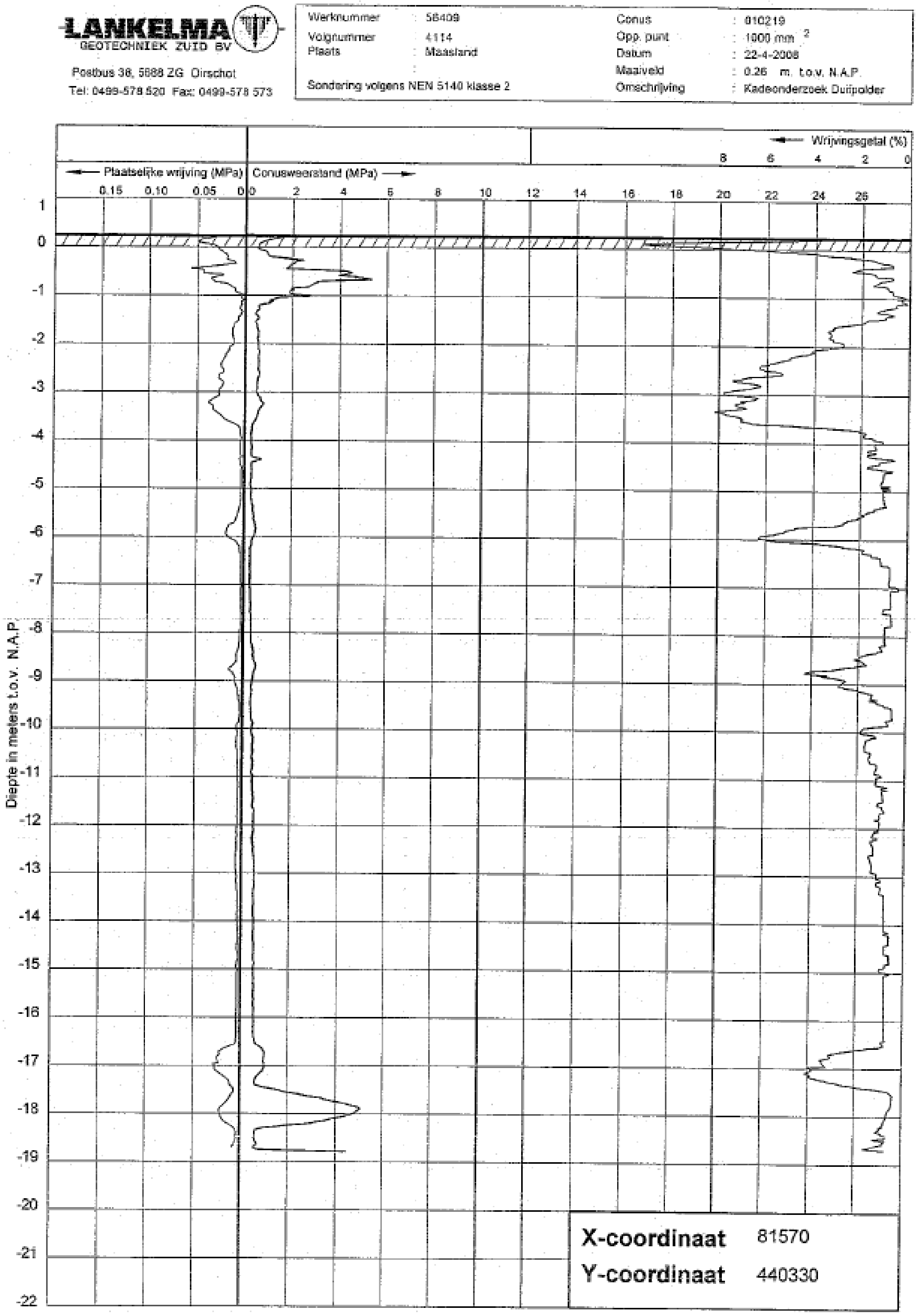


Figure 82: Cone penetration test 4114

In Figure 74 it can be seen that the glass dike is closest located to the soil sample taken in point 4.1.15-HB. Using this data as the basis for the design also gives the most conservative results as this sample contains the largest amount of peat. However, this sample does not contain data for layers lower than N.A.P. -4.86 meter. For soil parameters below this depth, the samples at point 4.3.15-MB_01 and 4.3.15-MB_02 are used in combination with the cone penetration results from point 4315. The soil characteristics as shown in the results of the soil investigations are very detailed. Since the glass dike is not exactly located at the investigation points and the first engineering design is only based on relative simple design checks, a simplification, based on a conservative combination of both the samples and cone penetration tests, is made in which the soil is characterized only by basic soil types, like clay, sand, peat etcetera. The result is shown in Figure 83.

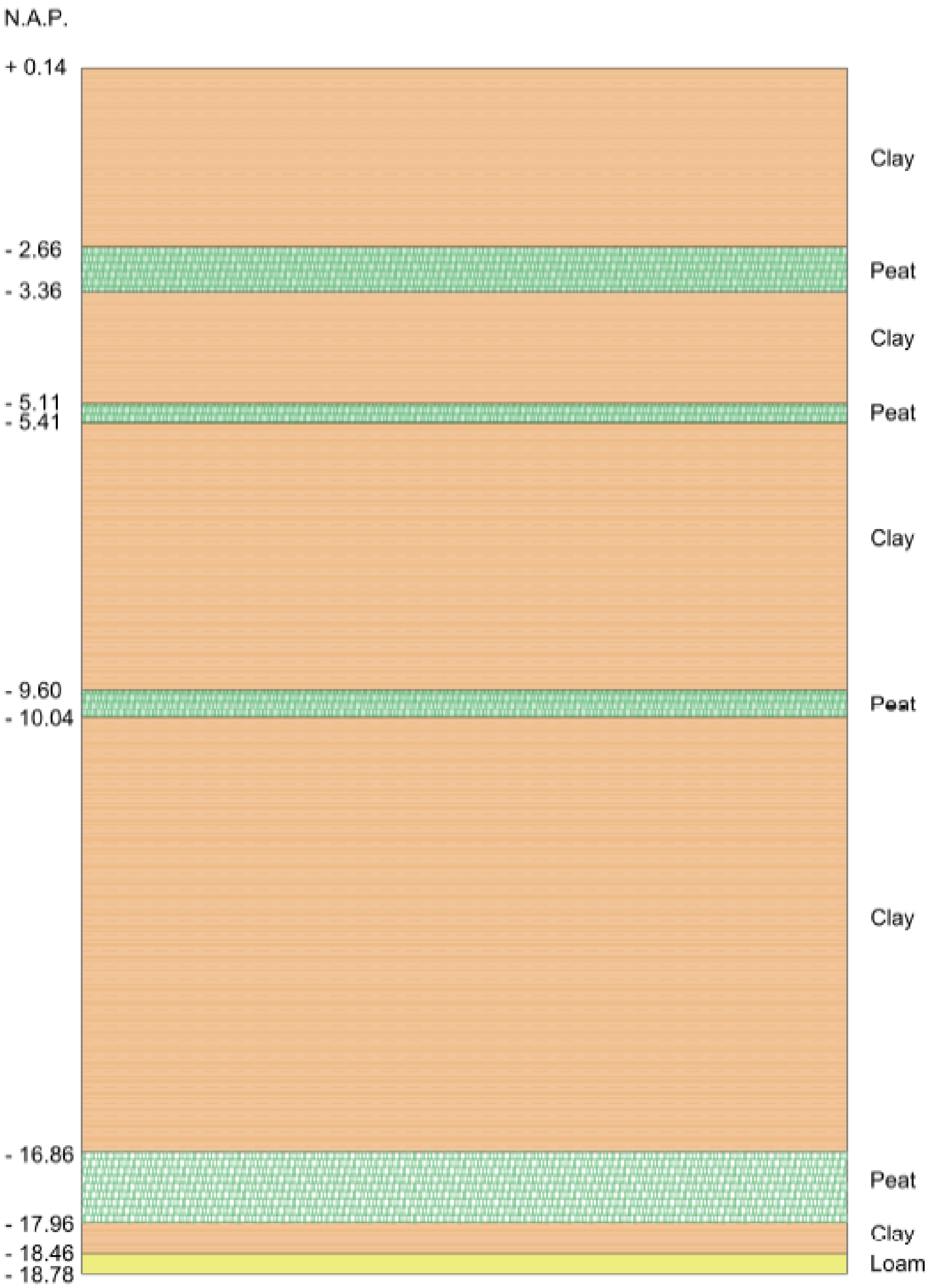


Figure 83: Simplified soil composition at/near the glass dike

Appendix A.3: Non-water retaining objects

Figure *84* shows the trees on the desired location of the glass dike. Besides the trees, there is a telephone cable situated in parts of the dike along the Noordvliet. The exact location is not known.

Figure 84: Trees on the desired location of the glass dike

Appendix B: Design checks for the semi-probabilistic design

Appendix B.1: Shallow foundation - uplift calculation

The hydrostatic pressure under the structure is the driving force for uplift to occur. The situation with the design water level of N.A.P. -0,03 m gives the largest upward pressure. All downward directed forces provide resistance against uplift. The least favourable situation is the case when only the self-weight of the structure is taken into account. Since friction between the soil and walls only have a minor contribution to the resistance, this is neglected. Furthermore, the soil in front of the glass dike might be eroded due to currents or propeller wash and the soil on the polder side of the glass dike might be removed for maintenance works or some other arbitrary reason. The structure is safe for uplift if the design value of the total self-weight is larger than the design value of the upward hydrostatic pressure:

$$G_{stb;d} \geq U_{dst;d}$$

In which:
$G_{stb;d}$ = Design value for the total self-weight [kN]
$U_{dst;d}$ = Design value of the upward hydrostatic pressure force [kN]

Eurocode 7 gives load factors specifically for the failure mechanism uplift. These are shown in Table 33.

Load	Symbol	Value
Permanent		
Unfavourable	$y_{G;dst}$	1,0
Favourable	$y_{G;stb}$	0,9
Variable		
Unfavourable	$y_{Q;dst}$	1,5

Table 33: Load factors for uplift

The governing load combination and corresponding load factors are shown in Table 34.

Load	Element	Direction	(Un)favourable	Load factor
Self-weight	All	Vertical	Favourable	0,9
Hydrostatic pressure	Floor	Vertical	Unfavourable	1,0

Table 34: Governing load combination and applicable load factors

The upward hydrostatic pressure force that acts on the bottom of the structure, is calculated by integrating the upward pressure over the width of the bottom and then multiplying it by the length of the structure. For this, the phreatic line as assumed in section 5.5.1 (Hydraulic boundary conditions) is used as input. The governing water levels and phreatic line are shown in Figure 85. Since the hydrostatic pressure is linearly decreasing over the width of the bottom and constant over the length, the characteristic value of the upward force can be calculated with following formula:

$$U_{dst;ck} = L\left(\frac{1}{2}W(p_2 + p_4)\right) \quad [kN]$$

In which:
L = Length of the structure [m]
W = Width of the structure [m]
p_2 = Hydrostatic pressure at the bottom of the foundation on the water-side [kN/m^2]
p_4 = Hydrostatic pressure at the bottom of the foundation on the polder-side [kN/m^2]

Hydrostatic pressures are calculated below:

$$p_2 = y_w \cdot h = 10 \cdot (-0{,}03 - -3{,}70) = 36{,}70\ kN/m^2$$
$$p_4 = y_w \cdot h = 10 \cdot (-1{,}85 - -3{,}70) = 18{,}50\ kN/m^2$$

In which:

y_w = Specific weight of water [kN/m^3]

h = Depth below the water table or phreatic line [m]

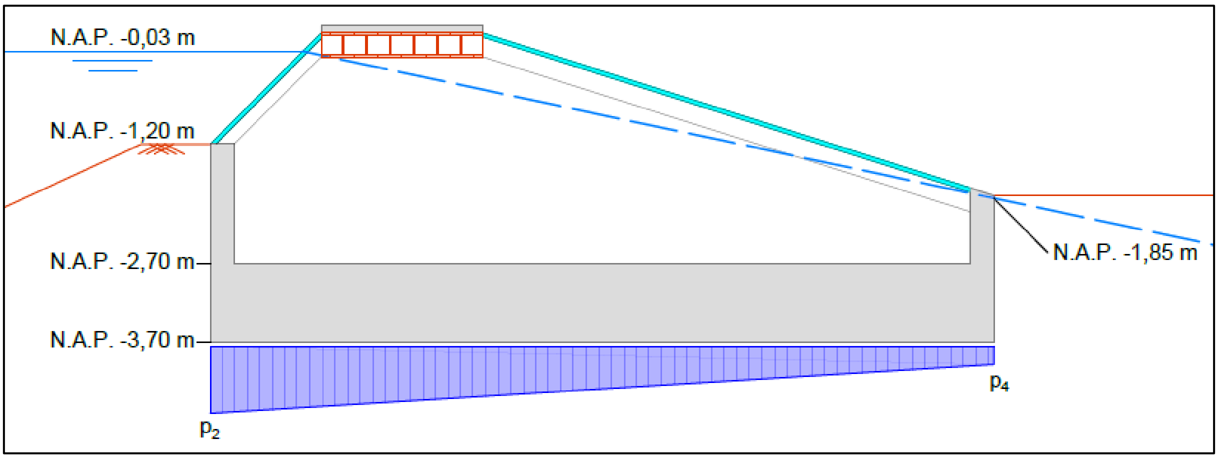


Figure 85: Upward hydraulic pressure at the design water level

Then:

$$U_{dst;ck} = 22 \cdot \left(\frac{1}{2} \cdot 9{,}90 \cdot (36{,}70 + 18{,}50)\right) = 6011{,}28 \text{ kN}$$

The characteristic value of the self-weight of the glass dike is computed with following formula:

$$G_{stb;ck} = \sum_{i=1}^{n} V_i \cdot y_i$$

In which:

V_i = Volume of element i [m^3]

y_i = Specific weight of the material of element i [kN/m^3]

The elements that are taken into account for computation of the total self-weight are the roof, water retaining glass elements, glass elements on the polder-side, wall water-side, wall polder-side, side walls, intermediate wall and the floor. For clarity, some of these elements are shown in Figure 86.

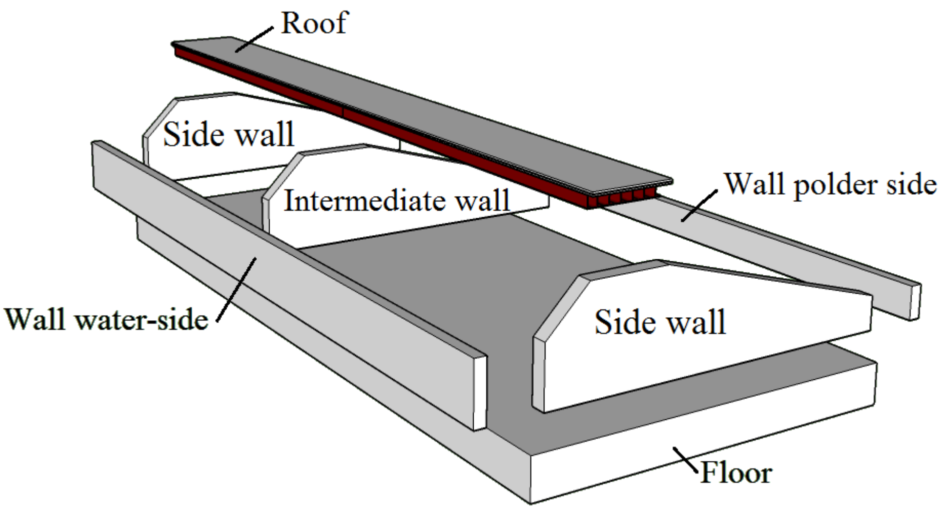


Figure 86: Overview of elements contributing to the self-weight (glass not in figure)

The roof is constructed with steel HE300B beams. Therefore, the self-weight of the roof is not calculated conform the formula shown above, but by multiplying the total length of beams by the weight as specified by steel beam producers. The self-weight of a HE300B beam is 1,19 kN/m for a single beam (staaltabellen.nl, 2015). The self-weight of the glass elements is estimated by using a representative thickness. This representative thickness accounts for the weight of the glass itself, but also for the weight of the connection/supporting systems and frames. The representative thickness is estimated to be 0,10 m for the water retaining glass elements and 0,05 m for the glass elements at the side of the polder. These are conservative estimates. The self-weight of elements and total self-weight of the glass dike is shown in Table 35.

Element	Thickness [m]	Surface area [m^2]	Volume [m^3]	Self-weight [kN]
Floor	1,00	217,80	217,80	5445,00
Wall water-side	0,30	33,00	9,90	247,50
Wall polder-side	0,30	18,70	5,61	140,25
Side walls	0,40	2 x 17,55 = 35,10	14,04	351,00
Intermediate wall	0,30	17,55	5,26	131,63
Glass water-side	0,10	43,56	4,35	108,90
Glass polder-side	0,05	142,56	7,12	178,20
Roof	Total beam length = 6 · 21,60 = 129,60 m; 1,19 kN/m			154,22
Characteristic value of the total self-weight				6756,70

Table 35: Characteristic self-weight of the glass dike

Applying the load factors to the characteristic values of the loads gives the design loads. This is shown in Table 36.

Load	Element	Characteristic value	Load factor	Design value
Vertical	[-]	[kN]	[-]	[kN]
Self-weight	All	6756,70	0,9	6081,03
Hydrostatic pressure	Floor	6011,28	1,0	6011,28

Table 36: Design values of the loads

Now, the design check is performed:

$$G_{stb;d} = 6081{,}03 \text{ kN} > U_{dst;d} = 6011{,}28 \text{ kN}$$

Hence the structure is safe for uplift for the above given dimensions.

Appendix B.2: Shallow foundation - bearing capacity

The bearing capacity is checked with the Brinch Hansen method. This method is also prescribed by Eurocode 7. It is based on Prandtl's theoretical sliding surfaces and makes a distinction between drained and undrained soil. Some time after construction of the structure, the drained situation is applicable. However, if construction is performed in a relative short period, the soil needs time to adjust to the loading as the pore pressures will increase due to the added load. Therefore, the undrained bearing capacity should be checked as well.

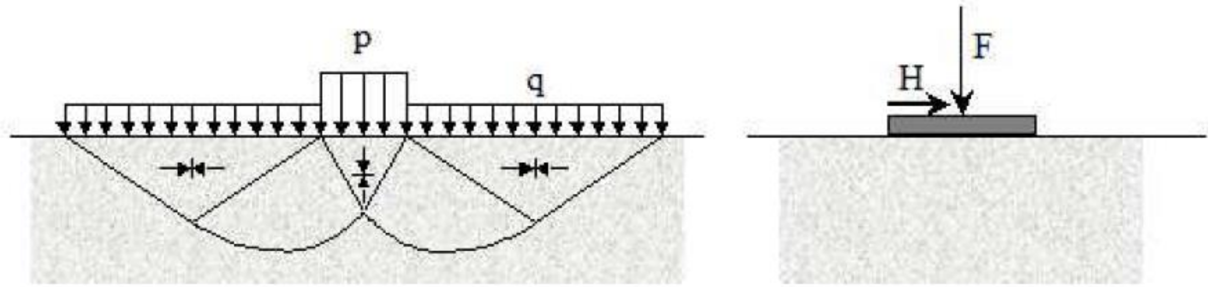


Figure 87: Bearing capacity according to Prandtl and Brinch Hansen (Vrijling, et al., 2015)

In addition to Prandtl's theory, Brinch Hansen applied reduction factors for the influence of a horizontal force and the relation between the length and width of the foundation. Besides the load of the structure itself, also the surcharge q of the surrounding soil is taken into account. This is only the case for embedded foundations. (Vrijling, et al., 2015)

Soil parameters

The soil parameters given in section 5.5.2 are characteristic values given by the Dutch national annex of Eurocode 7. The design values are obtained by applying the material factors shown in Table 37. These material factors are also prescribed by the Dutch national annex of Eurocode 7 specifically for shallow foundations.

Soil parameter	Symbol	Material factor (y_x)
Angle of internal friction	φ'	1,15
Effective cohesion	c'	1,60
Undrained shear strength	c_u	1,35
Specific weight	y	1,10

Table 37: Material factors (Eurocode 7)

Then the following design values are obtained for the soil parameters:

$$\varphi'_{\mathrm{d}} = \mathrm{ARCTAN}\big(\mathrm{TAN}(\varphi')/y_{\varphi'}\big) = \mathrm{ARCTAN}(\mathrm{TAN}(15)/1{,}15) = 13°$$

$$\mathrm{c}'_{\mathrm{d}} = \mathrm{c}'/\gamma_{\mathrm{c}'} = 0/1{,}60 = 0 \text{ kPa}$$

$$y_{\mathrm{clay;sat;d}} = y_{\mathrm{clay;dry;d}} = y_{\mathrm{clay;sat}}/\mathrm{y_y} = y_{\mathrm{clay;dry}}/\mathrm{y_y} = 13/1{,}10 = 11{,}80 \text{ kN/m}^3$$

$$y'_{\mathrm{clay;d}} = (y_{\mathrm{clay;sat}} - y_{\mathrm{wat}})/\mathrm{y_y} = (13-10)/1{,}10 = 2{,}70 \text{ kN/m}^3$$

$$\mathrm{c_{u;d}} = \mathrm{c_u}/\mathrm{y_{cu}} = 10/1{,}35 = 7{,}41 \text{ kPa}$$

Loads

The situation with low water gives the smallest upward hydraulic pressure, hence it is governing for the bearing capacity check. All downward directed loads are unfavourable and all upward directed loads are favourable. Horizontal loads are taken into account in the Brinch-Hansen method as well. These are largest at the side of the water. Therefore, horizontal loads are unfavourable if directed to the side of the polder. The loads are given in Table 38 and illustrated in Figure 88.

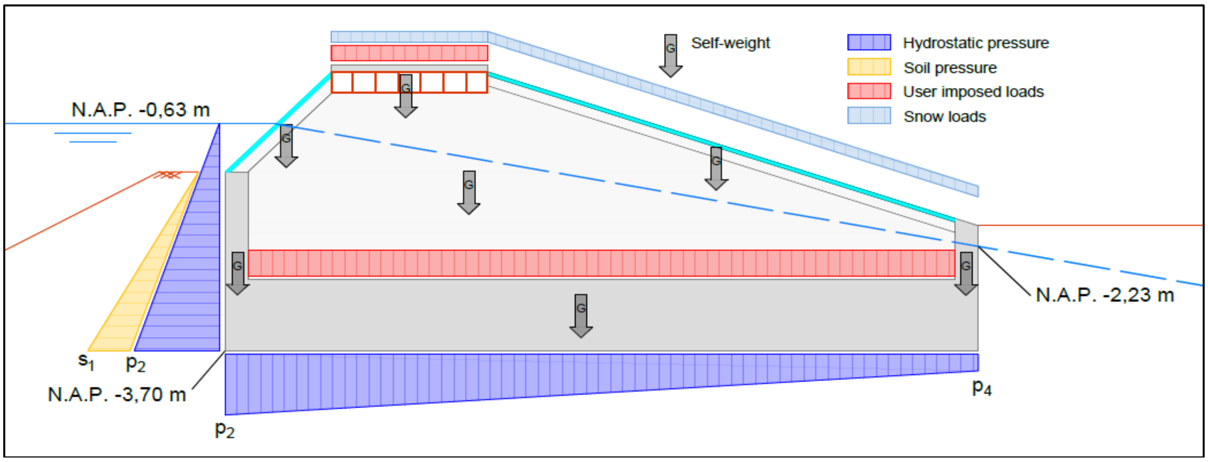


Figure 88: Governing load combination for the bearing capacity check

Load	Element(s)	Direction	(Un)favourable	Type
Self-weight	All	Vertical	Unfavourable	Permanent
User imposed	Roof	Vertical	Unfavourable	Variable
User imposed	Floor	Vertical	Unfavourable	Variable
Snow	Roof	Vertical	Unfavourable	Variable
Snow	Glass polder-side	Vertical	Unfavourable	Variable
Hydrostatic pressure	Floor	Vertical	Favourable	Permanent
Hydrostatic pressure	Glass and wall water-side	Horizontal	Unfavourable	Permanent
Soil pressure	Wall water-side	Horizontal	Unfavourable	Permanent

Table 38: Governing load combination for the bearing capacity check

Load factors are prescribed by the Dutch national annex of Eurocode 7 and are shown in Table 39. As explained in section 6.3.1, variable loads are also multiplied by a load combination factor of ψ = 0,70. Only the dominant variable load is multiplied by a full factor of 1,50.

Load factor		Symbol	Value
Permanent	Favourable	γ_G	1,35
	Unfavourable		0,90
Variable	Unfavourable	γ_Q	1,50

Table 39: Load factors for bearing capacity (Eurocode 7)

Self-weight

The characteristic values of the self-weight of the elements are readily computed for the uplift check. The results are shown in Table 40.

Element	Thickness [m]	Surface area [m^2]	Volume [m^3]	Self-weight [kN]
Floor	1,00	217,80	217,80	5445,00
Wall water-side	0,30	33,00	9,90	247,50
Wall polder-side	0,30	18,70	5,61	140,25
Side walls	0,40	2 x 17,55 = 35,10	14,04	351,00
Intermediate wall	0,30	17,55	5,26	131,63
Glass water-side	0,10	43,56	4,35	108,90
Glass polder-side	0,05	142,56	7,12	178,20
Roof	Total beam length = 6 · 21,60 = 129,60 m; 1,19 kN/m			154,22
Characteristic value of the total self-weight				6756,70

Table 40: Characteristic self-weight of the glass dike

User imposed loads

User imposed loads are prescribed by Eurocodes. The values per square meter are given in section 6.3.5. The total user imposed load for the roof is:

$$I_{roof;ck} = B_{roof} \cdot L_{roof} \cdot q_{f;ck} = 2{,}05 \cdot 22{,}00 \cdot 4{,}30 = 193{,}92 \text{ kN}$$

In which:

$I_{roof;ck}$	= Characteristic value of the user imposed load on the roof	[kN]
B_{roof}	= Width of the roof	[m]
L_{roof}	= Length of the roof	[m]
$q_{f;ck}$	= Characteristic value of the user imposed distributed load on the roof	[kN/m^2]

The total user imposed load for the floor is:

$$I_{floor;ck} = B_{floor} \cdot L_{floor} \cdot q_{ck} = 9{,}30 \cdot 20{,}90 \cdot 4{,}00 = 777{,}48 \text{ kN}$$

In which:

$I_{floor;ck}$	= Characteristic value of the user imposed load on the floor	[kN]
B_{floor}	= Width of the floor between the walls	[m]
L_{floor}	= Length of the floor between the walls	[m]
q_{ck}	= Characteristic value of the user imposed distributed load on the floor	[kN/m^2]

Snow loads

Similar to user imposed loads, snow loads are also prescribed by the Eurocodes. The value of the load per square meter for the desired location of the glass dike is shown in section 6.3.4. The characteristic value of the total snow load on the roof is computed below:

$$S_{roof;ck} = B_{roof} \cdot L_{roof} \cdot S_{ck} = 2{,}05 \cdot 22{,}00 \cdot 0{,}40 = 18{,}04 \text{ kN}$$

In which:

$S_{roof;ck}$	= Characteristic value of the total snow load on the roof	[kN]
S_{ck}	= Characteristic value of the distributed snow load	[kN/m^2]

The snow load on the glass on the polder-side of the glass dike is computed similar to the snow load on the roof. However, snow that falls between the roof and the edge of the structure at the polder-side is distributed over the sloped glass elements. Therefore, the characteristic load is less than 0,40 kN/m^2. This can be taken into account by not taking into account the full length of the glass elements, but only the horizontal distance between the beginning and the end of the glass elements. Then, the characteristic value of the total snow load on the glass elements on the polder-side of the glass dike is:

$$S_{glass;ck} = B_{b-e} \cdot L \cdot S_{ck} = 6{,}45 \cdot 22{,}00 \cdot 0{,}40 = 56{,}76 \text{ kN}$$

In which:

$S_{glass;ck}$	= Characteristic value of the total snow load on the glass (polder-side)	[kN]
B_{b-e}	= Horizontal distance between the beginning and the end of the glass elements	[m]
S_{ck}	= Characteristic value of the distributed snow load	[kN/m^2]

Hydraulic pressure

The upward hydraulic pressure that acts on the floor is calculated in the same fashion as done for the uplift check. The formula for the total upward pressure under the structure is shown below:

$$U_{ck} = L\left(\frac{1}{2}B(p_2 + p_4)\right)$$

In which:

U_{ck}	= Characteristic value of the total upward hydraulic pressure	[kN]
L	= Length of the structure	[m]
B	= Width of the structure	[m]
p_2	= Water pressure at the bottom of the foundation on the water-side	[kN/m^2]
p_4	= Water pressure at the bottom of the foundation on the polder-side	[kN/m^2]

The water pressures are calculated according to the formulas given in section 6.3.2. This is shown below:

$$p_2 = y_w \cdot h = 10 \cdot (-0{,}63 - -3{,}70) = 30{,}70\ \mathrm{kN/m^2}$$
$$p_4 = y_w \cdot h = 10 \cdot (-2{,}23 - -3{,}70) = 14{,}70\ \mathrm{kN/m^2}$$

In which:

y_w	= Specific weight of water	[kN/m^3]
h	= depth below the water table or phreatic line	[m]

Then:

$$U_{ck} = 22 \cdot \left(\frac{1}{2} \cdot 9{,}90 \cdot (30{,}70 + 14{,}70)\right) = 4944{,}06\ \mathrm{kN}$$

The hydrostatic pressure acting on the glass dike at the water-side of the structure is computed with the formula given in section 6.3.2. Multiplying with the length of the structure gives the total characteristic value:

$$U_{h;ck} = \frac{1}{2} \cdot y_w \cdot h^2 \cdot L = 0{,}5\ \cdot 10 \cdot (-0{,}63 - -3{,}70)^2 \cdot 22 = 1036{,}74\ \mathrm{kN}$$

In which:

$U_{h;ck}$	= Characteristic value of the total horizontal hydrostatic pressure	[kN]

Soil pressure

The only remaining load to be calculated is the soil pressure that is acting on the wall at the side of the water. Since the soil in front of the structure is composed of only one layer and fully situated under the water line, the formula of vertical soil pressure given in section 6.3.3 can be reduced to:

$$s'_{1;v} = y'_{clay;d} \cdot d = 2{,}70\ \cdot (-1{,}20 - -3{,}70) = 6{,}75\ \mathrm{kN/m^2}$$

In which:

$S'_{1;v}$	= Value of the effective vertical soil pressure at the bottom of the floor	[kN/m^2]
$y'_{clay;d}$	= Design value of the effective weight of clay	[kN/m^3]
d	= Thickness of the soil layer	[m]

No horizontal displacement of the structure is allowed, hence the coefficient of neutral soil pressure (Jaky) should be applied:

$$K_n = 1 - \sin(\varphi'_d) = 1 - \sin(13) = 0{,}775$$

In which:

K_n	= Coefficient of neutral soil pressure	[-]

Then, the effective horizontal soil pressure at the bottom of the floor is calculated as follows:

$$s'_{1;h;n} = K_n s'_{1;v} = 0{,}775 \cdot 6{,}75 = 5{,}23\ \mathrm{kN/m^2}$$

This horizontal soil pressure is linear increasing from zero at the top of the layer to the value calculated above at the bottom of the floor. Then, the characteristic value of the total horizontal soil pressure acting on the structure is calculated as shown below:

$$N_{ck} = \frac{1}{2} \cdot s'_{1;h;n} \cdot d \cdot L = 0{,}5 \cdot 5{,}23 \cdot (-1{,}2 - -3{,}70) \cdot 22 = 143{,}83\ \mathrm{kN}$$

In which:
N_{ck} = Characteristic value of the total horizontal soil pressure [kN]

Design values of the loads
Applying the load factors to the characteristic values of the loads gives the design loads. This is shown in Table 41. The hydraulic pressure that is acting on the floor is split up into two parts, a constant pressure acting on the entire floor (rectangular pressure) and a pressure that is decreasing from the water-side to the polder-side (triangular pressure). This is done to simplify the computation of the resultant design moment.

Load	Elements/parts	Characteristic value	Load factor	Design value
Vertical	[-]	[kN]	[-]	[kN]
Self-weight	Floor	5445,00	1,35	7350,75
Self-weight	Wall water-side	247,50	1,35	334,13
Self-weight	Wall polder-side	140,25	1,35	189,34
Self-weight	Side wall (North)	175,50	1,35	236,93
Self-weight	Side wall (South)	175,50	1,35	236,93
Self-weight	Intermediate wall	131,63	1,35	177,70
Self-weight	Glass water-side	108,90	1,35	147,02
Self-weight	Glass polder-side	178,20	1,35	240,57
Self-weight	Roof	154,22	1,35	208,20
Snow	Roof	18,04	1,05 (0,7x1,5)	27,06
Snow	Glass	56,76	1,05 (0,7x1,5)	85,14
User imposed	Roof	193,92	1,05 (0,7x1,5)	203,62
User imposed	Floor	777,48	1,5	1166,22
Hydraulic pressure	Floor (rectangle)	-3201,66	0,9	-2881,49
Hydraulic pressure	Floor (triangle)	-1742,40	0,9	-1568,16
Horizontal				
Hydrostatic pressure	Water-side	1036,74	1,35	1399,60
Soil pressure	Wall water-side	143,83	1,35	194,17

Table 41: Design values of the loads

Bearing capacity of the drained soil

The bearing capacity of the soil for the drained situation is computed with the formula of Brinch Hansen:

$$\frac{R}{A'} = c'_d N_c b_c s_c i_c + q' N_q b_q s_q i_q + \frac{1}{2} y'_d B' N_y b_y s_y i_y$$

In which:
R/A' = Bearing capacity of the soil [kN/m^2]
q' = Effective pressure of the surcharge [kN/m^2]
c'_d = Effective cohesion [kN/m^2]
y'_d = Specific weight of the soil [kN/m^3]
B' = Effective width of the foundation [m]

And dimensionless factors for respectively the cohesion, surcharge and specific weight parts of the formula:

$N_{c/q/y}$	= Bearing capacity factors	[-]
$b_{c/q/y}$	= Factors for the foundation angle	[-]
$s_{c/q/y}$	= Factors for the shape of the foundation	[-]
$i_{c/q/y}$	= Force inclination factors	[-]

The effective length and width of the foundation depends on the acting loads. Eccentric resultant loads result in smaller effective dimensions. For the length, the loading is symmetric, so L' = L. For the width B' = B - 2·e_b, in which B is the width of the foundation and e_b is computed as follows:

$$e_b = \frac{\Sigma M}{\Sigma V}$$

In which ΣM is the resultant moment, induced by the design loads and ΣV the resultant of all vertical design loads. Table 42 shows the computation of ΣM and ΣV. The leverarms are specified as the distance between the load and the centre of the foundation and are measured in Autocad.

$$e_b = \frac{1604{,}51}{6153{,}93} = 0{,}26 \text{ m} \qquad \text{and} \qquad B' = 9{,}90 - 2 \cdot 0{,}26 = 9{,}38 \text{ m}$$

Design load	Elements/parts	Value [kN]	Leverarm [m]	Induced moment [kNm]
Vertical				
Self-weight	Floor	7350,75	0,00	0,00
Self-weight	Wall water-side	334,13	-4,80	-1603,80
Self-weight	Wall polder-side	189,34	4,80	908,82
Self-weight	Side wall North	236,93	-1,05	-248,77
Self-weight	Side wall South	236,93	-1,05	-248,77
Self-weight	Intermediate wall	177,70	-1,05	-186,59
Self-weight	Glass water-side	147,02	-4,10	-602,76
Self-weight	Glass polder-side	240,57	1,58	378,90
Self-weight	Roof	208,20	-2,53	-525,70
Snow	Roof	27,06	-2,53	-68,33
Snow	Glass	85,14	1,58	134,10
User imposed	Roof	203,62	-2,53	-514,14
User imposed	Floor	1166,22	0,00	0,00
Hydrostatic pressure	Floor (rectangle)	-2881,49	0,00	0,00
Hydrostatic pressure	Floor (triangle)	-1568,16	-1,65	2587,46
Total vertical (ΣV)		**6153,93**		
Horizontal				
Hydrostatic pressure	Water-side	1399,60	1,02	1432,26
Soil pressure	Wall water-side	194,17	0,83	161,83
Total horizontal (ΣH)		**1593,80**		
Total moment (ΣM)				**1604,51**

Table 42: Computation of resultant design loads

The Brinch-Hansen formula takes into account cohesion, surcharge and capacity of the soil below the foundation. Surcharge of the soil is not taken into account, since the soil on the polder-side might be excavated for maintenance purposes. The dimensionless factors are calculated below:

Dimensionless factors for the bearing capacity below the foundation

$$N_q = e^{\pi \tan(\varphi'_d)} \tan^2\left(45 + \frac{\varphi'_d}{2}\right) = e^{\pi \tan(13)} \tan^2\left(45 + \frac{13}{2}\right) = 3{,}26$$

$$N_C = (N_q - 1)\cot(\varphi'_d) = (3{,}26 - 1)\cot(13) = 9{,}79$$

$$N_y = 2(N_q - 1)\tan(\varphi'_d) = 2(3{,}26 - 1)\tan(13) = 1{,}04$$

Dimensionless factors for the angle of the foundation with the horizontal axis

$$b_q = b_y = \left(1 - \alpha \cdot \tan^2(\varphi'_d)\right) = \left(1 - 0 \cdot \tan^2(13)\right) = 1$$

In which α is the angle of the foundation with the horizontal axis (=0º)

$$b_c = b_q - (1 - b_q)/N_c \tan(\varphi'_d) = 1 - (1 - 1)/9{,}79 \tan(13) = 1$$

Dimensionless factors for the shape of the foundation (for rectangular shapes)

$$s_q = 1 + (B'/L')\sin(\varphi'_d) = 1 + (9{,}38/22)\sin(13) = 1{,}10$$

$$s_y = 1 - 0{,}3(B'/L') = 1 - 0{,}3(9{,}38/22) = 0{,}87$$

$$s_c = \frac{(s_q N_q - 1)}{(N_q - 1)} = \frac{(1{,}10 \cdot 3{,}26 - 1)}{(3{,}26 - 1)} = 1{,}14$$

Dimensionless factors for the inclination of the resultant force

$$m = (2 + B'/L')/(1 + B'/L') = (2 + 9{,}38/22)/(1 + 9{,}38/22) = 1{,}70$$

$$i_q = \left[1 - H/\left(V + A'c'_d \cot(\varphi'_d)\right)\right]^m = \left[1 - 1593{,}80/\left(6153{,}93 + 9{,}38 \cdot 22 \cdot 0 \cdot \cot(13)\right)\right]^{1{,}70} = 0{,}60$$

$$i_c = i_q - (1 - i_q)/\left(N_c \tan(\varphi'_d)\right) = 0{,}60 - (1 - 0{,}60)/\left(9{,}79 \tan(13)\right) = 0{,}42$$

$$i_y = \left[1 - H/\left(V + A'c'_d \cot(\varphi'_d)\right)\right]^{m+1} = \left[1 - 1593{,}80/\left(6153{,}93 + 9{,}38 \cdot 22 \cdot 0 \cdot \cot(13)\right)\right]^{1{,}70+1}$$

$$i_y = 0{,}44$$

All parameters are now known and substituted in the Brinch Hansen formula. This gives the bearing capacity for the drained soil as shown below:

$$\frac{R}{A'} = 0 \cdot 9{,}79 \cdot 1 \cdot 1{,}14 \cdot 0{,}42 + 0 + 0{,}5 \cdot 2{,}70 \cdot 9{,}38 \cdot 1{,}04 \cdot 1 \cdot 0{,}87 \cdot 0{,}44 = 5{,}04 \text{ kN/m}^2$$

Bearing capacity for undrained soil

The bearing capacity of the soil for the undrained situation is computed with the formula shown below:

$$\frac{R}{A'} = (\pi + 2)c_{u;d} b_c s_c i_c + q$$

The effective cohesion is replaced by the undrained shear strength. Furthermore, the weight of the soil in the sliding planes is not taken into account. The computation of the dimensionless factors is also different. This is shown below. The definition of these factors remain unchanged.

$b_c = 1 - 2\alpha/(\pi + 2) = 1 - 2 \cdot 0(\pi + 2) = 1$

$s_c = 1 + 0{,}2(B'/L') = 1 + 0{,}2(9{,}38/22) = 1{,}08$

$$i_c = \frac{1}{2}\left(1 + \sqrt{1 - \frac{H}{A'c_{u;d}}}\right) \quad \text{if } H \leq A'c_{u;d}$$

$$i_c = \frac{1}{2} \quad \text{if } H \geq A'c_{u;d}$$

$H = 1593{,}80 > A'c_{u;d} = 9{,}38 \cdot 22 \cdot 7{,}41 = 1529{,}013\ \text{kN} \rightarrow i_c = \frac{1}{2}$

Similar to the drained situation, surcharge is not taken into account: $q = 0\ \text{kN/m}^2$. Then:

$$\frac{R}{A'} = (\pi + 2) \cdot 7{,}41 \cdot 1 \cdot 1{,}08 \cdot 0{,}5 + 0 = 20{,}55\ \text{kN/m}^2$$

Maximum pressure exerted on the soil below the structure:
The maximum pressure on the soil exerted by the structure is obtained with help of the following formula:

$$\sigma'_{max} = \frac{\sum V}{BL} + \frac{\sum M}{\frac{1}{6}LB^2} = \frac{6153{,}93}{9{,}9 \cdot 22} + \frac{1604{,}51}{\frac{1}{6} \cdot 22 \cdot 9{,}9^2} = 32{,}72\ \text{kN/m}^2$$

In which:

σ'_{max}	= Maximum effective soil pressure directly below the structure	[kN/m²]
ΣV	= Design value of the vertical resultant load	[kN]
ΣM	= Design value of the resultant moment, induced by design loads	[kNm]
B	= Width of the structure	[m]
L	= Length of the structure	[m]

Since the upward hydraulic pressure is already taken into account in the computation of the resultant vertical load, the above obtained value of the maximum exerted soil pressure is in fact the effective soil pressure.

Design check
The maximum pressure exerted on the soil below the structure may not exceed the bearing capacity of the soil, so the following criteria must be met:

$\sigma'_{max} \leq R/A'$

For the drained and undrained soil respectively:
$\sigma'_{max} = 32{,}72\ \text{kN/m}^2 > 5{,}04\ \text{kN/m}^2 = R/A'$
$\sigma'_{max} = 32{,}72\ \text{kN/m}^2 > 20{,}55\ \text{kN/m}^2 = R/A'$

Hence the design criteria for the bearing capacity is not met. From this, it is concluded that a pile foundation is required.

Appendix B.3: Pile foundation – tension

The pile foundation has sufficient resistance against uplift if the pile which is subjected to the largest upward directed (tension) force does not fail. Three types of tension failure can be distinguished. One possible failure mechanism is that piles are pulled out of the soil because of insufficient friction between the shaft of the piles and the soil. Another possibility is that the shaft friction is large, but the pile is pulled out together with a clump of soil. In order to prevent this, the total weight of the clump should be larger than the upward directed force on the pile. Lastly, the pile itself can fail if the strength is too low. For correctly designed piles, this is usually not the case. Therefore, only the following ultimate limit states are checked:

$F_{s;d} \leq F_{r;shaft;d}$
$F_{s;d} \leq F_{r;clump;d}$

In which:

$F_{s;d}$	= Design value of the largest tension force that is acting on a single pile	[kN]
$F_{r;shaft;d}$	= Design value of the maximum shaft friction (for tension)	[kN]
$F_{r;clump;d}$	= Design value of the tension capacity according to the clump criterion	[kN]

Estimate of the required dimensions

A first estimate of the required pile dimensions, together with a pile scheme is given in Figure 89. The pile dimensions are input for the computation of the tension resistance of single piles. The scheme is input for analysis of the structural behaviour, but also for the calculation of design loads (self-weight of piles).

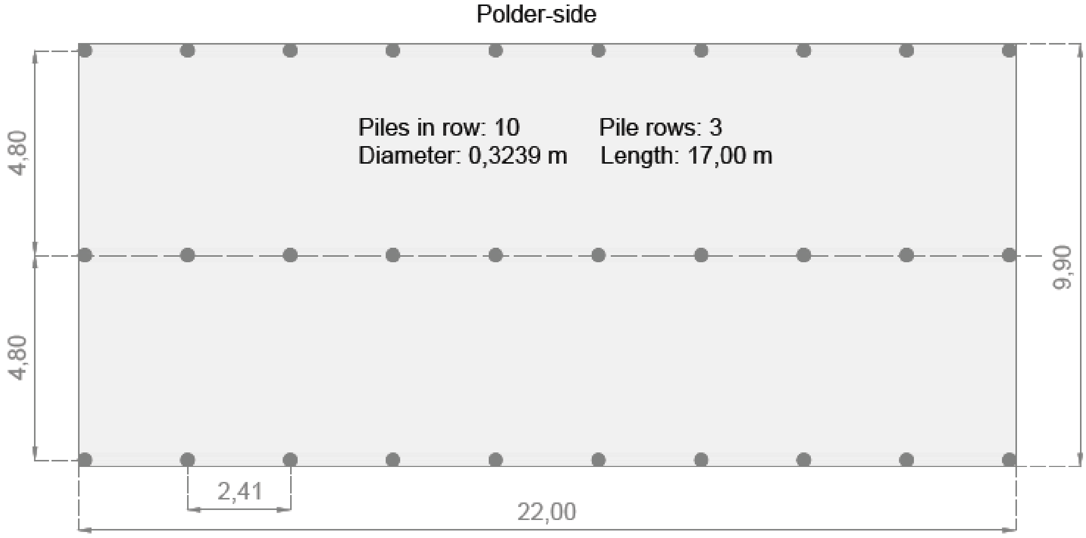


Figure 89: First estimate of pile foundation dimensions

Tension capacity (resistance)

In this part of the appendix, the maximum admissible tension force on a single pile is computed. First, the soil parameters are given.

Soil parameters

Characteristic values of the soil parameters are given in section 5.5.2. For the design of pile foundations, Eurocode 7 prescribes the following material factors:

Soil parameter	Symbol	Material factor (y_x)
Angle of internal friction	φ’	1,15
Effective cohesion	c’	1,60
Undrained shear strength	c_u	1,35
Specific weight	y	1,10

Table 43: Soil parameters for the design of pile foundations (Eurocode 7)

Then, the following design values are obtained for the parameters of clay:

$q_c = 200 \text{ kPa (CPT value)}$

$\varphi'_d = \text{ARCTAN}(\text{TAN}(\varphi')/y_{\varphi'}) = \text{ARCTAN}(\text{TAN}(15)/1{,}15) = 13°$

$c'_d = c'/y_{c'} = 0/1{,}6 = 0 \text{ kPa}$

$y_{clay;sat;d} = y_{clay;dry;d} = y_{clay;sat}/y_y = y_{clay;dry}/y_y = 13/1{,}1 = 11{,}80 \text{ kN/m}^3$

$y'_{clay;d} = (y_{clay;sat} - y_{wat})/y_y = (13{,}00 - 10{,}00)/1{,}1 = 2{,}70 \text{ kN/m}^3$

$c_{u;d} = c_u/y_{cu} = 10/1{,}35 = 7{,}41 \text{ kPa}$

As will be explained later, peat layers are not taken into account for the shaft friction calculation. For the clump criterion, only the specific weight is relevant:

$y_{peat;sat;d} = y_{peat;dry;d} = y_{peat;sat}/y_y = y_{peat;dry}/y_y = 12/1{,}1 = 10{,}90 \text{ kN/m}^3$

Maximum shaft friction

The maximum friction between the shaft of a pile and the surrounding soil is calculated with the following formula:

$$F_{r;shaft;d} = \int_{z=0}^{L} q_{c;z;d} \cdot f_1 \cdot f_2 \cdot O_{p;mean} \cdot \alpha_t \, dz$$

In which:

$F_{r;shaft;d}$	= Design value for maximum shaft friction	[kN]
$q_{c;z;d}$	= Design value of the cone resistance at depth z	[kN/m^2]
f_1	= Pile installation factor ($f_1 \geq 1$)	[-]
f_2	= Cone resistance reduction factor ($f_2 \leq 1$)	[-]
$O_{p;mean}$	= Average circumference of the pile shaft	[m]
α_t	= Pile class factor	[-]
L	= Length of the pile	[m]
z	= Depth (0 at the top of the pile)	[m]

For clay, the factors f_1 and f_2 are both equal to 1. The value of the cone resistance is approximately constant over the entire length of the pile. For this, the lowest value in the cone penetration test results as shown in Appendix A.2 is used:

$q_{c;z;max} = q_c = 200$ kPa (constant over the length of the pile)

The design value of the cone resistance is obtained by applying the following formula:

$$q_{c;z;d} = \frac{q_{c;z;rep}}{y_{m;b4} \cdot y_{m;var;qc}}$$

In which:

$y_{m;b4}$	= Material factor for tension piles (=1,4)	[-]
$y_{m;var;qc}$	= Material factor for variable loads (=1,0)	[-]

And:

$$q_{c;z;rep} = \xi \cdot q_{c;z;max}$$

With:

ξ = 0,86 (Factor for the number of piles (>10) and CPT`s (=2))

Then:

$$q_{c;z;d} = \frac{\xi \cdot q_{c;z;max}}{y_{m;b4} \cdot y_{m;var;qc}} = \frac{0{,}86 \cdot 200}{(1{,}4 \cdot 1{,}0)} = 122{,}86 \text{ kPa}$$

The soil consists of clay and peat layers. However, the shear stress in peat layers might reduce to zero due to creep effects. Therefore, the shaft friction in these layers is assumed to be zero. The average circumference of a pile is computed by multiplying the real circumference by the ratio of the total thickness of soil layers that contribute to the shaft friction over the total thickness of all soil layers:

$$O_{p;mean} = \frac{\sum_{i=1}^{n} T_{clay;i}}{\sum_{j=1}^{m} T_{peat;j} + \sum_{i=1}^{n} T_{clay;i}} \pi D = \frac{14{,}90}{17{,}00} \cdot \pi \cdot 0{,}3239 = 0{,}89 \text{ m}$$

$T_{clay;i}$	= Thickness of clay layer i	[m]
$T_{peat;j}$	= Thickness of peat layer j	[m]

For the computation above, the soil composition as shown in section 5.5.2 is used as input. The pile class factor depends on the qc value and the ratio of pile length over pile diameter. For the pile dimensions given in Figure 89, the pile class factor is:

α_t = 0,025 ($q_c \leq$ 1000 kPa and z/D > 20)

Since the value of the cone resistance is constant over the entire pile depth, the following simplification is made:

$$F_{r;shaft;d} = \int_{z=0}^{L} q_{c;z;d} \cdot f_1 \cdot f_2 \cdot O_{p;mean} \cdot \alpha_t dz = L \cdot q_{c;z;d} \cdot O_{p;mean} \cdot \alpha_t$$

Substituting the above determined values of the parameters gives the following shaft friction resistance:

$$F_{r;shaft;d} = 17 \cdot 122{,}86 \cdot 0{,}89 \cdot 0{,}025 = 46{,}47 \text{ kN}$$

Clump criterion

Even if the shaft friction of piles is very large, they can still get pulled out. In that situation, a clump of soil is pulled out of the ground together with the pile. Then, the maximum admissible tension force on a pile is determined by the weight of this clump. The weight of the pile itself is not taken into account for computation of the resistance as this will be taken into account as favourable load in the design check. The weight of the clump is calculated by multiplying the volume of the clump by the mean specific soil weight:

$$F_{clump} = (V_{cone} + V_{cylinder})\gamma_{d;mean}$$

The shape of the clump is approximated by a cylinder and a cone. This is shown in Figure 90.

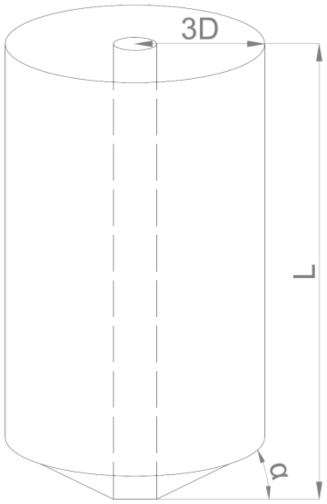


Figure 90: Simplified shape of the soil clump

The angle of the cone is approximately 2/3 times the design value of the angle of internal friction:

$$\alpha = \frac{2}{3}\varphi'_d = \frac{2}{3} \cdot 13 = 8{,}67°$$

The radius of the influence area of one pile is equal to three times the diameter of the pile.

$$R_{infl} = 3 \cdot D = 3 \cdot 0{,}3239 \approx 0{,}97 \text{ m}$$

The minimum distance between piles is 2,44 m. Then, the influence areas of the piles do not intersect, so they are not influenced by each other. The radius of the cone at the bottom is equal to the pile diameter divided by two. The radius of the cone at the top is equal to the influence are of the pile.

$$R_{cone;bottom} = \frac{D}{2} = \frac{0{,}3239}{2} \approx 0{,}16 \text{ m}$$

$$R_{cone;top} = R_{infl} = 0{,}97 \text{ m}$$

The height of the cone is computed using trigonometry:

$$H_{cone} = (R_{cone;top} - R_{cone;bottom}) \cdot \tan(\alpha) = (0{,}97 - 0{,}16) \cdot \tan(8{,}67) = 0{,}12 \text{ m}$$

The volume of the cone is computed with following formula:

$$V_{cone} = \frac{1}{3}\pi H_{cone}[R^2_{cone;top} + R_{cone;top} \cdot R_{cone;bottom} + R^2_{cone;bottom}] - \frac{1}{4}H_{cone}\pi D^2$$

This gives:

$$V_{cone} = \frac{1}{3}\pi \cdot 0{,}12 \cdot [0{,}97^2 + 0{,}16 \cdot 0{,}97 + 0{,}16^2] - \frac{1}{4} \cdot 0{,}12 \cdot \pi \cdot 0{,}3239^2 = 0{,}11 \text{ m}^3$$

The volume of the cylinder is computed in a similar fashion:

$$V_{cylinder} = \frac{1}{4}\pi(L - H_{cone})((6D)^2 - D^2) = \frac{1}{4}\pi(17 - 0{,}12)((6 \cdot 0{,}3239)^2 - 0{,}3239^2) = 48{,}68 \text{ m}^3$$

The mean specific weight of the soil is calculated with use of the following expression:

$$y_{d;mean} = \left(y_{d;peat} \sum_{j=1}^{m} T_{peat;j} + y_{d;clay} \sum_{i=1}^{n} T_{clay;i} \right) \Big/ L$$

In which:

$y_{d;mean}$	= Mean specific weight of soil	[kN/m^3]
$y_{d;peat}$	= Specific weight of peat	[kN/m^3]
$y_{d;clay}$	= Specific weight of clay	[kN/m^3]
$T_{peat;j}$	= Thickness of peat layer j	[m]
$T_{clay;i}$	= Thickness of clay layer I	[m]
L	= Length of the pile	[m]

For the computation above, the soil composition as shown in section 5.5.2 is used as input. Then:

$$y_{d;mean} = (10{,}90 \cdot 2{,}10 + 11{,}80 \cdot 14{,}90)/17 = 11{,}70 \text{ kN/m}^3$$

Now, the weight of the clump can be calculated:

$$F_{clump} = (V_{cone} + V_{cylinder})y_{d;mean} = (0{,}11 + 48{,}68) \cdot 11{,}70 = 570{,}84 \text{ kN/m}^3$$

Loads

The situation with high water gives the largest upward hydraulic pressure, hence it is governing for the tension capacity check of piles. All downward directed loads are favourable and all upward directed loads are unfavourable. Besides vertical loads, moments induced by design loads will also induce pile forces in axial direction. The hydrostatic pressure acting on the water-side of the structure is dominant for the resultant moment. Therefore, this load together with the soil pressure on the water-side are unfavourable loads. The soil pressure on the polder-side of the structure is not taken into account. This results in a more conservative design. The loads are given in Table 44 and illustrated in Figure 91.

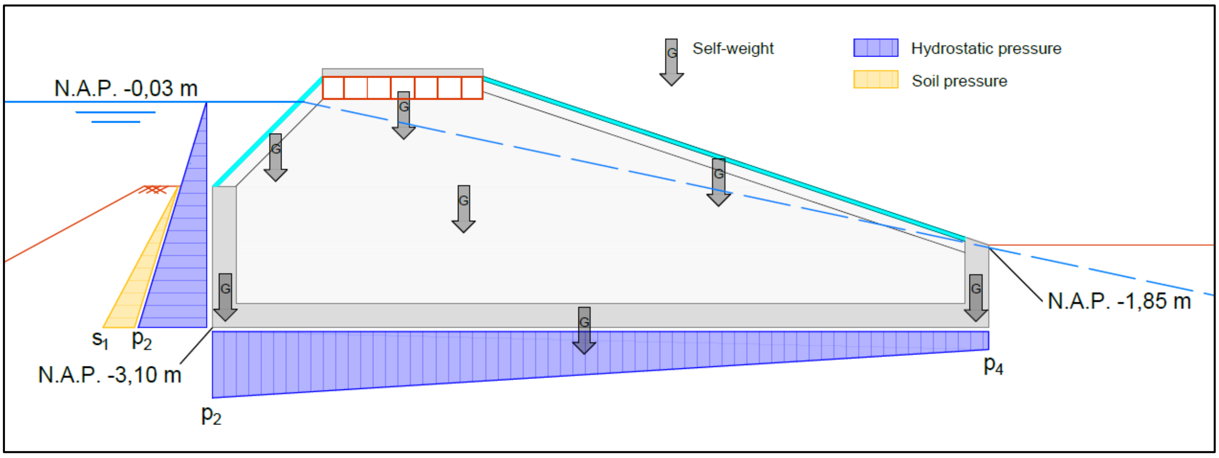


Figure 91: Governing load combination for the calculation of tension forces on piles

Load	Element(s)	Direction	(Un)favourable	Type
Self-weight	All	Vertical	Favourable	Permanent
Hydraulic pressure	Floor	Vertical	Unfavourable	Permanent
Hydraulic pressure	Water-side	Horizontal	Unfavourable	Permanent
Soil pressure	Wall water-side	Horizontal	Unfavourable	Permanent

Table 44: Governing load combination for the calculation of tension forces on piles

For design calculations of pile foundations, load factors are prescribed by Eurocode 7. These factors are shown in Table 45.

Load factor		Symbol	Value
Permanent	Favourable	y_G	1,35
	Unfavourable		0,90
Variable	Unfavourable	y_Q	1,50

Table 45: Load factors for pile foundations (Eurocode 7)

Self-weight

The self-weight of elements is already computed in the uplift check for a shallow foundation. This can be seen in Appendix B.1. However, the floor is thinner in case a pile foundation is applied, hence it has a lower self-weight. Furthermore, the self-weight of the piles is added. The computation of characteristic values of the self-weight of elements is shown in Table 46.

Element	Thickness [m]	Surface area [m^2]	Volume [m^3]	Self-weight [kN]
Floor	0,40	217,80	87,12	2178,00
Wall water-side	0,30	33,00	9,90	247,50
Wall polder-side	0,30	18,70	5,61	140,25
Side walls	0,40	2 x 17,55 = 35,10	14,04	351,00
Intermediate wall	0,30	17,55	5,26	131,63
Glass water-side	0,10	43,56	4,35	108,90
Glass polder-side	0,05	142,56	7,12	178,20
Piles	Total pile length = 30 piles · 17,00 m = 510,00 m; 2,56 kN/m			1305,60
Roof	Total beam length = 6 beams · 21,60 m = 129,60 m; 1,19 kN/m			154,22

Table 46: Characteristic values of the self-weight of elements

The self-weight of piles is composed of the weight of a steel tube with a concrete fill. Steel tubes with an external diameter of 0,3239 meter and steel thickness of 0,01 meter are applied. The weight of such a tube is 0,75 kN/m (staaltabellen.nl, 2015). The weight of the concrete fill is calculated below:

$$G_{pile-cf;ck} = \frac{1}{4}\pi(D-2t)^2 \cdot y_c = \frac{1}{4}\pi(0{,}3239-0{,}02)^2 \cdot 25 = 1{,}81\ \mathrm{kN/m}$$

In which:

$G_{pile-cf;ck}$	= Characteristic value of the weight of the concrete fill in a pile	[kN/m]
D	= External diameter of the steel tube	[m]
t	= Steel thickness	[m]
y_c	= Specific weight of concrete	[kN/m^3]

Then, the characteristic value of the self-weight of one meter pile is:

$$G_{pile;ck} = G_{pile-tube;ck} + G_{pile-cf;ck} = 0{,}75 + 1{,}81 = 2{,}56\ \mathrm{kN/m}$$

Hydraulic pressure
The upward hydraulic pressure that acts on the floor is calculated in the same fashion as done for the uplift check. The formula of the total upward hydraulic pressure under the structure is shown below:

$$U_{ck} = L\left(\frac{1}{2}B(p_2 + p_4)\right)$$

In which:

U_{ck}	= Characteristic value of the total upward hydraulic pressure (force)	[kN]
L	= Length of the structure	[m]
B	= Width of the structure	[m]
p_2	= Water pressure at the bottom of the foundation on the water-side	[kN/m^2]
p_4	= Water pressure at the bottom of the foundation on the polder-side	[kN/m^2]

The water pressures are calculated according to the formulas given in section 6.3.2. This is shown below:

$$p_2 = y_w \cdot h = 10 \cdot (-0{,}03 - -3{,}10) = 30{,}70 \text{ kN/m}^2$$
$$p_4 = y_w \cdot h = 10 \cdot (-1{,}85 - -3{,}10) = 12{,}50 \text{ kN/m}^2$$

In which:

y_w	= Specific weight of water	[kN/m^3]
h	= Depth below the water table or phreatic line	[m]

Then:

$$U_{ck} = 22 \cdot \left(\frac{1}{2} \cdot 9{,}90 \cdot (30{,}70 + 12{,}50)\right) = 4704{,}48 \text{ kN}$$

The hydrostatic pressure acting on the glass dike at the waterside of the structure is calculated with the formula given in section 6.3.2. Multiplying with the length of the structure gives the total characteristic value:

$$U_{h;ck} = \frac{1}{2} \cdot y_w \cdot h^2 \cdot L = 0{,}5 \cdot 10 \cdot (-0{,}03 - -3{,}10)^2 \cdot 22 = 1036{,}74 \text{ kN}$$

In which:

$U_{h;ck}$	= Characteristic value of the total horizontal hydrostatic pressure (force)	[kN]

Soil pressure
Soil pressures are calculated according to the formula`s given in section 6.3.3. The effective vertical soil pressure at the bottom of the floor is:

$$s'_{1;v} = y'_{clay;d} \cdot d = 2{,}70 \cdot (-1{,}20 - -3{,}10) = 5{,}13 \text{ kN/m}^2$$

In which:

$s'_{1;v}$	= Value of the effective vertical soil pressure at the bottom of the floor	[kN/m^2]
$y'_{clay;d}$	= Design value of the effective weight of clay	[kN/m^3]
d	= Thickness of the soil layer	[m]

No horizontal displacement of the structure is allowed, hence the coefficient of neutral soil pressure (Jaky) should be applied:

$$K_n = 1 - \sin(\varphi'_d) = 1 - \sin(13) = 0{,}775$$

In which:

K_n	= Coefficient for neutral soil pressure	[-]

Then, the effective horizontal soil pressure at the bottom of the floor is calculated as follows:

$$s'_{1;h;n} = K_n s'_{1;v} = 0{,}775 \cdot 5{,}13 = 3{,}98\ \mathrm{kN/m^2}$$

This horizontal soil pressure is linear increasing from zero at the top of the layer to the value calculated above at the bottom of the floor. Then, the characteristic value of the total horizontal soil pressure acting on the structure is calculated as shown below:

$$N_{ck} = \frac{1}{2} \cdot s'_{1;h;n} \cdot d \cdot L = 0{,}5 \cdot 3{,}98 \cdot (-1{,}20 - -3{,}10) \cdot 22 = 83{,}18\ \mathrm{kN}$$

In which:
N_{ck} = Characteristic value of the total horizontal soil pressure [kN]

Design values of the loads
Applying the load factors to the characteristic values of the loads gives the design loads. This is shown in Table 47.

Load	Elements/parts	Characteristic value	Load factor	Design value
Vertical	[-]	[kN]	[-]	[kN]
Self-weight	Floor	2178,00	0,9	1960,20
Self-weight	Wall water-side	247,50	0,9	222,75
Self-weight	Wall polder-side	140,25	0,9	126,23
Self-weight	Side wall (North)	175,50	0,9	157,95
Self-weight	Side wall (South)	175,50	0,9	157,95
Self-weight	Intermediate wall	131,63	0,9	118,47
Self-weight	Glass water-side	108,90	0,9	98,01
Self-weight	Glass polder-side	178,20	0,9	160,38
Self-weight	Piles	1305,60	0,9	1175,04
Self-weight	Roof	154,22	0,9	138,80
Hydraulic pressure	Floor (rectangle)	-2722,50	1,35	-3675,38
Hydraulic pressure	Floor (triangle)	-1981,98	1,35	-2675,67
Horizontal				
Hydrostatic pressure	Water-side	1036,74	1,35	1399,60
Soil pressure	Wall water-side	83,18	1,35	112,30

Table 47: Design values of the loads

Pile forces in axial direction

In order to perform design checks on single piles, the design loads need to be translated into pile forces in axial direction. Therefore, the distributions of forces over the piles is examined. Axial pile forces are the result of vertical design loads and moments induced by all design loads. For these moments, also the horizontal design loads are relevant. For the analysis of axial pile forces, the following assumptions are made:

- Pile forces in axial direction are positive for compression
- The floor of the structure is stiff and does not bend
- Vertical loads are equally distributed over the piles
- Moments cause a differentiation of axial forces in the piles over the width of the foundation
- Axial forces in piles are linear proportional to the vertical displacement of the piles
- Moments are computed around the center of the bottom of the floor

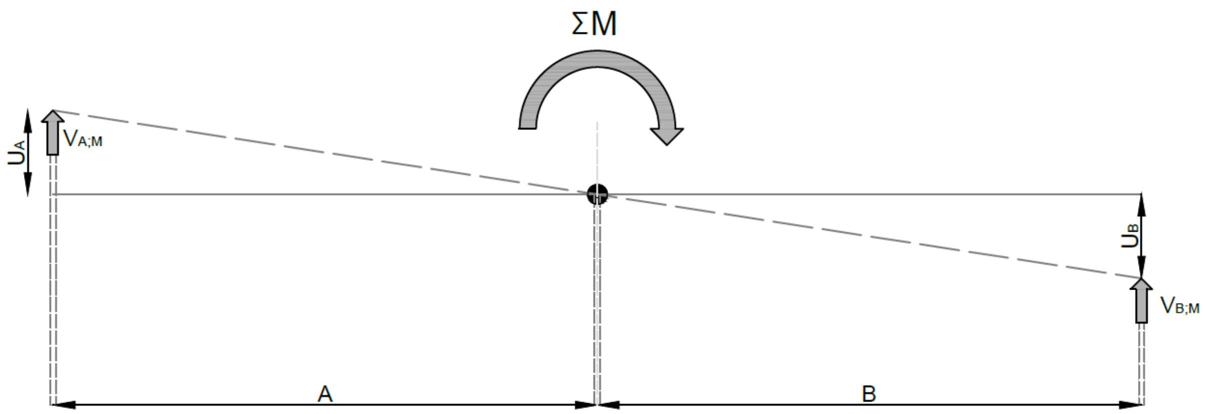


Figure 92: Schematization of axial pile forces caused by moments that are induced by design loads

With these assumptions, the pile displacement (and axial force) is linear increasing with the distance from the centre of the foundation. For the symmetric pile scheme as shown in Figure 89, axial pile forces induced by the resultant moment, are relative easy to obtain with help of following formula:

$$V_{A;M;1p} = -V_{B;M;1p} = -\frac{\sum M}{2A\#_{row}}$$

In which:

$V_{A;M;1p}$	= Axial force in a single pile on the water-side induced by the resultant moment	[kN]
$V_{B;M;1p}$	= Axial force in a single pile on the polder-side induced by the resultant moment	[kN]
ΣM	= Resultant moment induced by all design loads	[kNm]
A	= Distance between the centre line of the foundation and the centre of a pile at the edge	[m]
$\#_{row}$	= Number of piles in a row parallel to the length of the structure	[-]

Since the piles on the centre line of the foundation will not displace due to rotation, these piles are not subjected to axial forces due to the resultant moment. The piles at the water-side of the foundation are subjected to tension and the piles at the polder-side to compression. Therefore, the piles on the water-side are governing for the tension design check. The axial pile force, induced by vertical design loads, is easily calculated with following formula:

$$V_V = \frac{\sum V}{\#_{total}}$$

In which:

V_V	= Axial force in a single pile induced by vertical design loads	[kN]
ΣV	= Resultant vertical design load	[kN]
$\#_{total}$	= Total number of piles	[-]

Then, the total tension force (= -1 times the axial force) in a pile at the water-side is:

$$F_{s;d} = -(V_V + V_{A;M;1p}) = -\left(\frac{\sum V}{\#_{total}} - \frac{\sum M}{2A\#_{row}}\right)$$

The values of the resultant loads are computed in Table 48. Leverarms are measured with Autocad.

Design load	Value [kN]	Leverarm [m]	Induced moment [kNm]
Vertical			
Floor	1960,20	0,00	0,00
Wall water side	222,75	-4,80	-1069,20
Wall polder-side	126,23	4,80	605,88
Side wall North	157,95	-1,05	-165,85
Side wall South	157,95	-1,05	-165,85
Intermediate wall	118,47	-1,05	-124,39
Glass water-side	98,01	-4,10	-401,84
Glass polder-side	160,38	1,58	252,60
Piles	1175,04	0,00	0,00
Roof	138,80	-2,53	-350,46
Hydrostatic pressure (rectangle)	-3675,38	0,00	0,00
Hydrostatic pressure (triangle)	-2675,67	-1,65	4414,86
Resultant vertical load	**-2035,28**		
Horizontal			
Hydrostatic pressure load	1399,60	1,02	1432,26
Soil pressure water-side	112,30	0,63	71,12
Resultant horizontal load	**1511,89**		
Resultant moment			**4499,12**

Table 48: Computation of resultant design loads

Then, the design value of the maximum tension force on a single pile is:

$$F_{s;d} = -\left(\frac{\sum V}{\#_{total}} - \frac{\sum M}{2A\#_{row}}\right) = -\left(\frac{-2035{,}28}{30} - \frac{4499{,}12}{2 \cdot 4{,}8 \cdot 10}\right) = 114{,}79 \text{ kN}$$

Design check

Now that both the tension resistance of a pile and the maximum occurring tension force on a single pile are known, the design check is performed. In the calculation of the tension resistance it appeared that the maximum shaft friction of a pile is much smaller than the weight of a clump of soil that could be pulled out of the ground. Therefore, the shaft friction is governing for the design check.

$F_{s;d}$ = 114,79 kN
$F_{r;shaft;d}$ = 46,47 kN

Hence:
$F_{s;d} > F_{r;shaft;d}$

As can be seen, the tension resistance of the piles is not sufficient. Therefore, the dimensions of the foundation must be changed. After this, the tension capacity of the piles is checked again.

Altered dimensions of the pile foundation

The maximum tension force on piles can be reduced by increasing the number of piles. Another option is to increase the tension resistance by increasing the pile dimensions. Off course, a combination of both is also possible. Increasing the size of piles is not desired as heavier equipment is needed for construction. Increasing the number of piles will decrease the length of floor spans, thereby reducing internal stresses in the floor. Therefore, the number of piles is increased, without changing the dimensions of single piles. A new pile scheme is set up and shown in Figure 93.

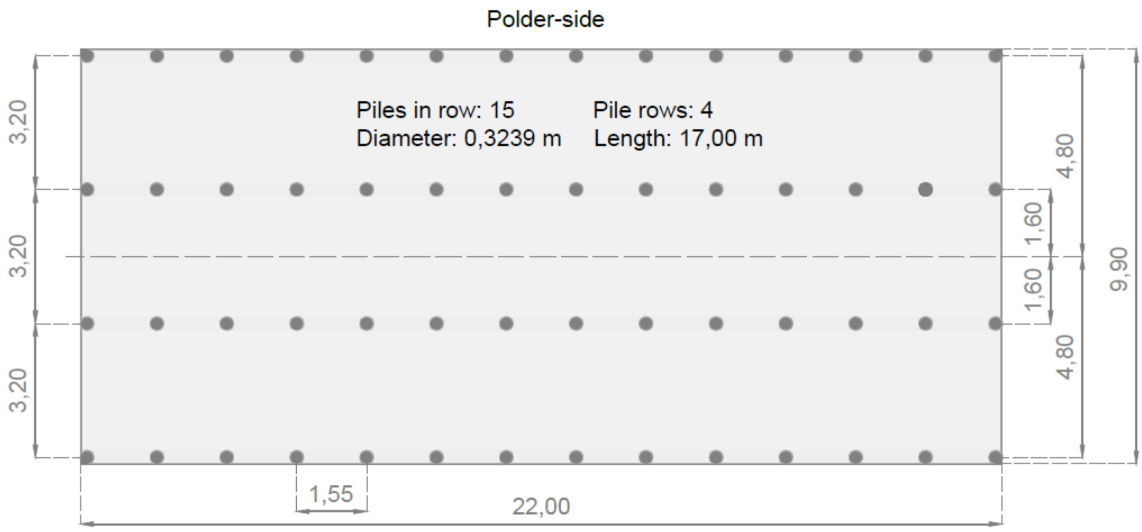


Figure 93: Second estimate of pile foundation dimensions

Loads for the altered dimensions of the pile foundation

For the altered dimensions of the foundation, only the total self-weight of the piles has been changed. The consequence is that the resultant vertical load is increased. Since the pile scheme is symmetric in a cross section of the structure, just as for the first pile scheme, the resultant moment remains unchanged. Since the number of piles is doubled compared to the previous pile scheme, the total self-weight of the piles is also doubled:

$$G_{piles;tot;ck} = 2 \cdot 1305{,}60 = 2611{,}20 \text{ kN}$$
$$G_{piles;tot;d} = 2 \cdot 1175{,}04 = 2350{,}80 \text{ kN}$$

In which:

$G_{piles;tot;ck}$	= Characteristic value of the total self-weight of the piles	[kN]
$G_{piles;tot;d}$	= Design value of the total self-weight of the piles	[kN]

The resultant vertical design load is also increased by 1175,04 kN, thus:

$$\Sigma V = -2035{,}28 + 1175{,}04 = -860{,}24 \text{ kN}$$

And the resultant design moment remains unchanged:

$$\Sigma M = 4499{,}12 \text{ kNm}$$

Pile forces in axial direction for the altered dimensions of the pile foundation

For the altered pile scheme, the analysis of axial pile forces is slightly more complicated. Again, the piles at the water-side of the foundation are governing for the tension design check. For analysis of axial pile forces, induced by the resultant moment, the following assumptions apply:

- Pile forces in axial direction are positive for compression
- The floor of the structure is stiff and does not bend
- Vertical loads are equally distributed over the piles
- Moments cause a differentiation of axial forces in the piles over the width of the foundation
- Axial forces in piles are linear proportional to the vertical displacement of the piles
- Moments are computed around the center of the bottom of the floor

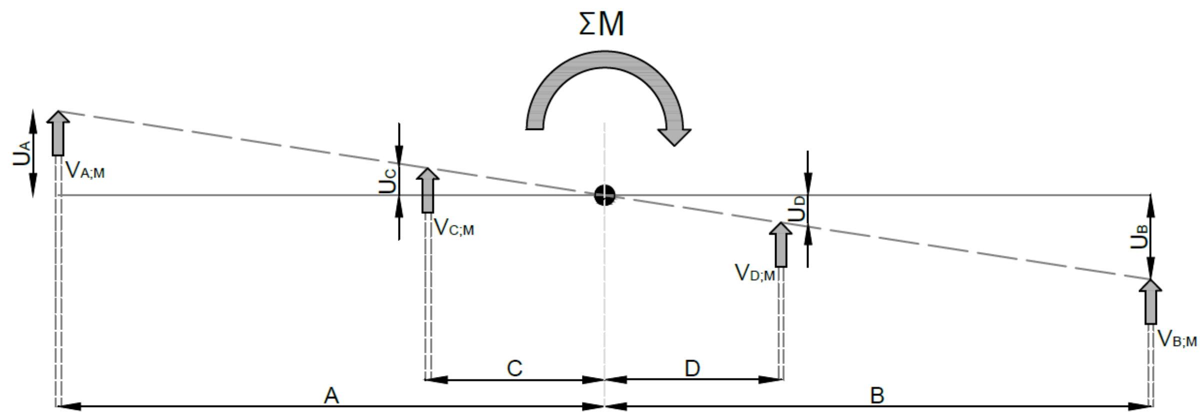


Figure 94: Schematization of axial pile forces induced by the resultant moment

For the axial pile forces induced by the resultant moment, the following relations are identified:

$$\sum M = -A \cdot V_{A;M} + B \cdot V_{B;M} - C \cdot V_{C;M} + D \cdot V_{D;M}$$

$$V_{A;M} = kU_A$$
$$V_{B;M} = kU_B$$
$$V_{C;M} = kU_C$$
$$V_{D;M} = kU_D$$

$$U_A = -U_B$$
$$U_C = -U_D$$
$$U_D = (D/B)U_B$$

In which $V_{a/b/c/d;M}$ are axial forces induced by the resultant moment and $U_{a/b/c/d}$ are pile displacements. The parameter k represents the spring constant that relates the displacement of the piles to the force that is transferred from the piles to the soil. With help of these relations, all axial pile forces can be expressed in the axial force in the piles at the water-side of the structure:

$$V_{A;M} = V_{A;M}$$
$$V_{B;M} = kU_B = -kU_A = -V_{A;M}$$
$$V_{C;M} = kU_c = k(D/B)U_A = (D/B)V_{A;M}$$
$$V_{D;M} = kU_D = -k(D/B)U_A = -(D/B)V_{A;M}$$

Then:

$$\sum M = -[A + B + C(D/B) + D^2/B]V_{A;M}$$

The equation above is applicable for pile rows parallel to the length of the foundation. The axial force in a single pile at the water-side is obtained by dividing the force by the number of piles in a row:

$$V_{A;M;1p} = -\frac{\sum M}{[A + B + C(D/B) + D^2/B]\#_{row}}$$

Note that the expression above only gives the axial pile force that is induced by the resultant moment. For the total axial pile force, also the resultant vertical load needs to be taken into account. The total tension force (= -1 times the axial force) in a pile at the water-side is:

$$F_{s;d} = -(V_V + V_{A;M;1p}) = -\left(\frac{\sum V}{\#_{total}} - \frac{\sum M}{[A + B + C(D/B) + D^2/B]\#_{row}}\right)$$

Thus:

$$F_{s;d} = -\left(\frac{-860{,}24}{60} - \frac{4499{,}12}{[4{,}80 + 4{,}80 + 1{,}60(1{,}60/4{,}80) + 1{,}60^2/4{,}80] \cdot 15}\right) = 42{,}46 \text{ kN}$$

Design check for the altered dimensions of the pile foundation

Since the dimensions of single piles are unchanged, there is no need to recalculate the tension resistance. Again, the design check for tension resistance is performed:

$F_{s;d}$ = 42,46 kN
$F_{r;shaft;d}$ = 46,47 kN

Hence:

$$F_{s;d} < F_{r;shaft;d}$$

For the altered dimensions of the pile foundation as shown in Figure 93, the tension forces on piles are strongly reduced. Furthermore, the governing tension resistance criterion (shaft friction) is met.

Appendix B.4: Pile foundation – compression

The pile foundation has sufficient resistance against uplift if the pile that is subjected to the largest downward directed axial (compression) force does not fail. The pile fails if the soil that is surrounding the pile is not able to take over the load. The compression capacity of piles is determined by the sum of shaft friction and tip resistance. However, the soil at the desired location of the glass dike consists entirely of weak and cohesive layers. Therefore, there is no tip resistance, but only shaft friction. Since the soil is preloaded by the (removed) dike, settlements are only expected to occur as a consequence of the positive shaft friction. Therefore, negative shaft friction is not taken into account. In Appendix B.3, a pile foundation was dimensioned to prevent uplift. This foundation is used as input for the compression design check. The pile dimensions and scheme are shown in Figure 95.

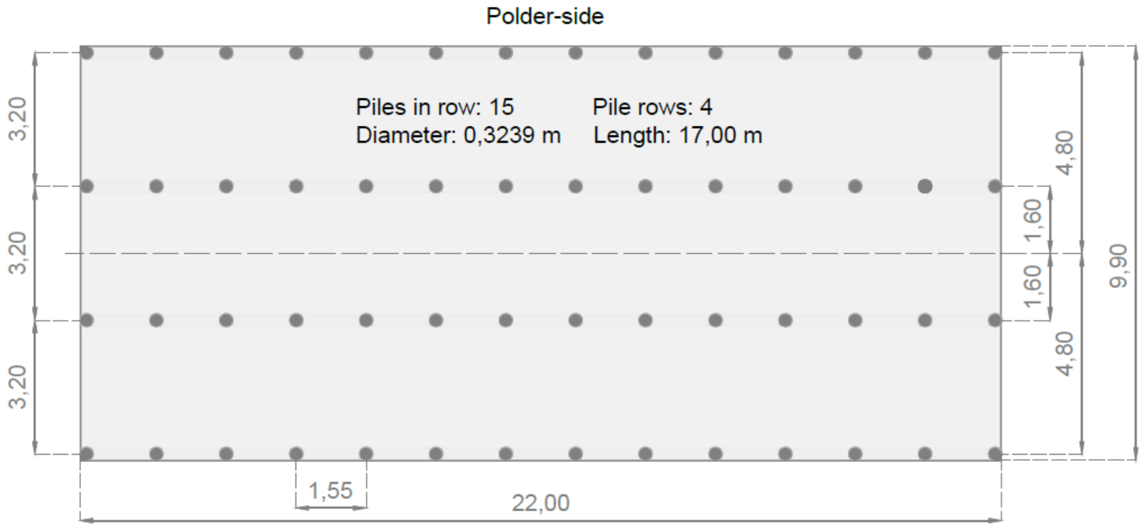


Figure 95: Pile foundation dimensions

It should be noted that the resistance that can be provided by the shaft friction is much larger for compression forces than for tension forces. The compression capacity of a pile is sufficient if the following criterion is satisfied:

$$F_{s;d} \leq F_{r;shaft;d}$$

In which:

$F_{s;d}$ = Design value of the largest compression force that is acting on a single pile [kN]

$F_{r;shaft;d}$ = Design value of the maximum shaft friction (for compression) [kN]

Compression capacity (resistance)

In this part of the appendix, the maximum admissible compression force on a single pile is computed. The soil consists of clay and peat layers. However, the shear stress in peat layers might reduce to zero due to creep effects. Therefore, the shaft friction in these layers is assumed to be zero. Design values of clay parameters were determined in Appendix B.3. These are shown in Table 49.

Soil parameter	Symbol	Design value
Angle of internal friction	φ'_d	13°
Effective cohesion	c'_d	0 kPa
Undrained shear strength	$c_{u;d}$	7,41 kPa
Specific weight	γ_d	11,80 kN/m^3
Effective (submerged) weight	γ'_d	2,70 kN/m^3

Table 49: Design values of clay parameters

The maximum friction between the shaft of a pile and the surrounding soil is calculated with the following formula:

$$F_{r;shaft;d} = O_{p;mean} \int_0^L \alpha_s \cdot q_{c;z;a}\, dz$$

In which:

$F_{r;shaft;d}$	= Maximum shaft friction force	[kN]
$O_{p;mean}$	= Mean circumference of the pile shaft	[m]
α_s	= Reduction factor	[-]
$q_{c;z;a}$	= Cone resistance in the cone penetration test at depth z	[kN/m^2]
L	= Length of the pile	[m]
z	= Depth below foundation (equal to zero at the top of the pile)	[m]

The value of the cone resistance is approximately constant over the entire length of the pile. For this, the lowest value in the cone penetration test results as shown in Appendix A.2 is used:

$q_{c;z;a} = q_c = 200$ kPa (constant over the length of the pile)

For clay, the reduction factor α_s depends on the CPT values and on the relative pile depth, which is the ratio of the embedded pile depth over the pile diameter. For $q_c \leq 1000$ kPa and $z/D > 20$, the following reduction factor is applicable:

$\alpha_s = 0{,}055$

The soil consists of clay and peat layers. However, the shear stress in peat layers might reduce to zero due to creep effects. Therefore, the shaft friction in these layers is assumed to be zero. This is taken into account in the calculation of the average circumference of the pile shaft. The average circumference of a pile is computed by multiplying the real circumference by the ratio of the total thickness of soil layers that contribute to the shaft friction over the total thickness of all soil layers:

$$O_{p;mean} = \frac{\sum_{i=1}^{n} T_{clay;i}}{\sum_{j=1}^{m} T_{peat;j} + \sum_{i=1}^{n} T_{clay;i}} \pi D = \frac{14{,}90}{17{,}00} \cdot \pi \cdot 0{,}3239 = 0{,}89\ \text{m}$$

For the computation above, the soil composition as shown in section 5.5.2 is used as input. Since the value of the cone resistance is constant over the depth, the following simplification is made:

$$F_{r;shaft;d} = O_{p;mean} \int_0^L \alpha_s \cdot q_{c;z;a}\, dz = O_{p;mean} \cdot \alpha_s \cdot q_{c;z;a} \cdot L$$

Substituting the above determined values of the parameters gives:

$$F_{r;shaft;d} = O_{p;mean} \cdot \alpha_s \cdot q_{c;z;a} \cdot L = 0{,}89 \cdot 0{,}055 \cdot 200 \cdot 17 = 166{,}43\ \text{kN}$$

Loads

The situation with low water gives the smallest upward hydraulic pressure. As a consequence, more downward directed loads are transferred via the piles to the soil. All downward directed loads are unfavourable and all upward directed loads are favourable. For calculation of compression forces in the piles, a similar approach is used as for the calculation of tension forces (Appendix B.3). The basis of this approach is that pile forces in axial direction are induced by vertical loads and moments that are induced by horizontal and vertical loads. Calculation of the resultant moment with characteristic values of the loads shows that this moment is clockwise (in a cross-section of the structure with the polder on the right side). This moment is calculated around the centre of the bottom of the structure. As a consequence, the piles on the polder side of the glass dike are subjected to the largest compression force. Therefore, the soil pressure and hydrostatic pressure on the water-side are unfavourable loads. The soil- and water pressure on the polder side of the structure are favourable as they induce a moment in opposite direction of the resultant moment. However, since the soil on this side might be excavated for maintenance or some other arbitrary reason, these loads are not taken into account. The loads are given in Table 50 and illustrated in Figure 96.

Load	Element(s)	Direction	(Un)favourable	Type
Self-weight	All	Vertical	Unfavourable	Permanent
Hydraulic pressure	Floor	Vertical	Favourable	Permanent
Hydraulic pressure	Water-side	Horizontal	Unfavourable	Permanent
Soil pressure	Wall water-side	Horizontal	Unfavourable	Permanent
User imposed	Roof, Floor	Vertical	Unfavourable	Variable
Snow	Roof, Glass	Vertical	Unfavourable	Variable

Table 50: Governing load combination for calculation of compression forces on piles

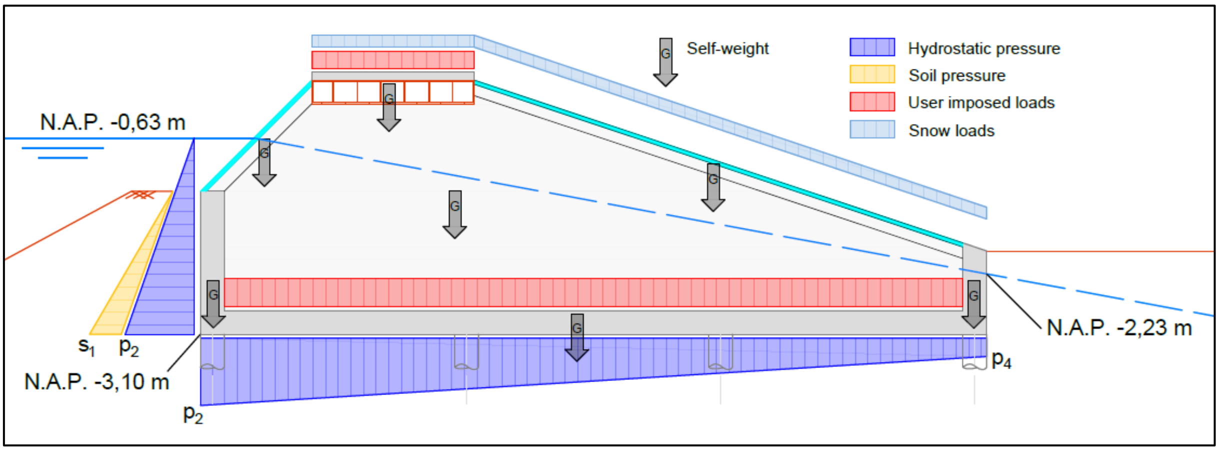


Figure 96: Governing load combination for calculation of compression forces on piles

For design calculations of pile foundations, load factors are prescribed by Eurocode 7. These factors are shown in Table 51.

Load factor		Symbol	Value
Permanent	Favourable	γ_G	1,35
	Unfavourable		0,90
Variable	Unfavourable	γ_Q	1,50

Table 51: Load factors for pile foundations (Eurocode 7)

Most of the loads are already computed for other design checks as the same loads apply. Calculation of the self-weight and soil pressure can be seen in Appendix B.3 (tension piles). Snow loads and user imposed loads are calculated in Appendix B.2 (bearing capacity for a shallow a foundation). The characteristic values of these loads are shown in Table 52. In the same table, load factors are applied to obtain the design values. Only the values of hydraulic loads are different for this design check. These are calculated below:

<u>*Hydraulic pressure*</u>

The upward hydraulic pressure that acts on the floor is calculated in the same fashion as done for the uplift check. The formula of the total upward hydraulic pressure under the structure is shown below:

$$U_{ck} = L\left(\frac{1}{2}B(p_2 + p_4)\right)$$

U_{ck}	= Characteristic value of the total upward hydraulic pressure (force)	[kN]
L	= Length of the structure	[m]
B	= Width of the structure	[m]
p_2	= Water pressure at the bottom of the foundation on the water-side	[kN/m^2]
p_4	= Water pressure at the bottom of the foundation on the polder-side	[kN/m^2]

The water pressures are calculated according to the formulas given in section 6.3.2. This is shown below:

$$p_2 = \gamma_w \cdot h = 10 \cdot (-0{,}63 - -3{,}10) = 24{,}70 \text{ kN/m}^2$$
$$p_4 = \gamma_w \cdot h = 10 \cdot (-2{,}23 - -3{,}10) = 8{,}70 \text{ kN/m}^2$$

In which:

γ_w	= Specific weight of water	[kN/m^3]
h	= Depth below the water table or phreatic line	[m]

Then:

$$U_{ck} = 22 \cdot \left(\frac{1}{2} \cdot 9{,}90 \cdot (24{,}70 + 8{,}70)\right) = 3637{,}26 \text{ kN}$$

The hydrostatic pressure acting on the glass dike at the water-side of the structure is calculated with the formula given in section 6.3.2. Multiplying with the length of the structure gives the total characteristic value:

$$U_{h;ck} = \frac{1}{2} \cdot \gamma_w \cdot h^2 \cdot L = 0{,}5 \cdot 10 \cdot (-0{,}63 - -3{,}10)^2 \cdot 22 = 671{,}10 \text{ kN}$$

In which:

$U_{h;ck}$	= Characteristic value of the total horizontal hydrostatic pressure (force)	[kN]

Design values of loads

Table 52 gives an overview of the characteristic- and design values of the loads.

Load	Elements/parts	Characteristic value	Load factor	Design value
Vertical	[-]	[kN]	[-]	[kN]
Self-weight	Floor	2178,00	1,35	2940,30
Self-weight	Wall water-side	247,50	1,35	334,13
Self-weight	Wall polder-side	140,25	1,35	189,34
Self-weight	Side wall (North)	175,50	1,35	236,93
Self-weight	Side wall (South)	175,50	1,35	236,93
Self-weight	Intermediate wall	131,63	1,35	177,70
Self-weight	Glass water-side	108,90	1,35	147,02
Self-weight	Glass polder-side	178,20	1,35	240,57
Self-weight	Piles	2611,20	1,35	3525,12
Self-weight	Roof	154,22	1,35	208,20
Snow	Roof	18,04	1,05 (0,7x1,5)	27,06
Snow	Glass	56,76	1,05 (0,7x1,5)	85,14
User imposed	Roof	193,92	1,05 (0,7x1,5)	203,62
User imposed	Floor	777,48	1,5	1166,22
Hydraulic pressure	Floor (rectangle)	-1894,86	0,9	-1705,37
Hydraulic pressure	Floor (triangle)	-1742,40	0,9	-1568,16
Horizontal				
Hydrostatic pressure	Water-side	671,10	1,35	905,99
Soil pressure	Wall water-side	83,18	1,35	112,30

Table 52: Characteristic- and design values of the loads

Pile forces in axial direction

In order to perform design checks on single piles, the design loads need to be translated into pile forces in axial direction. Axial pile forces are the result of vertical design loads and moments induced by all design loads. The maximum compression force in a single pile is:

$$F_{s;d} = V_V + V_{A;M;1p}$$

In which:

$V_{A;M;1p}$	= Axial force in a single pile on the water-side induced by the resultant moment	[kN]
V_V	= Axial force in a single pile induced by vertical design loads	[kN]

The derivation of these parameters is performed in Appendix B.3. This derivation was based on following assumptions:

- Pile forces in axial direction are positive for compression
- The floor of the structure is stiff and does not bend
- Vertical loads are equally distributed over the piles
- Moments cause a differentiation of axial forces in the piles over the width of the foundation
- Axial forces in piles are linear proportional to the vertical displacement of the piles
- Moments are computed around the center of the bottom of the floor

The formulas that resulted from this analysis are shown here:

$$V_V = \frac{\sum V}{\#_{total}}$$

And:

$$V_{A;M;1p} = \frac{\sum M}{[A + B + C(D/B) + D^2/B]\#_{row}}$$

In which:

ΣM	= Resultant moment induced by all design loads	[kNm]
ΣV	= Resultant vertical design load	[kN]
$\#_{total}$	= Total number of piles	[-]
$\#_{row}$	= Number of piles in a row parallel to the length of the structure	[-]
A,B,…	= Dimensions (see Figure 97)	[m]

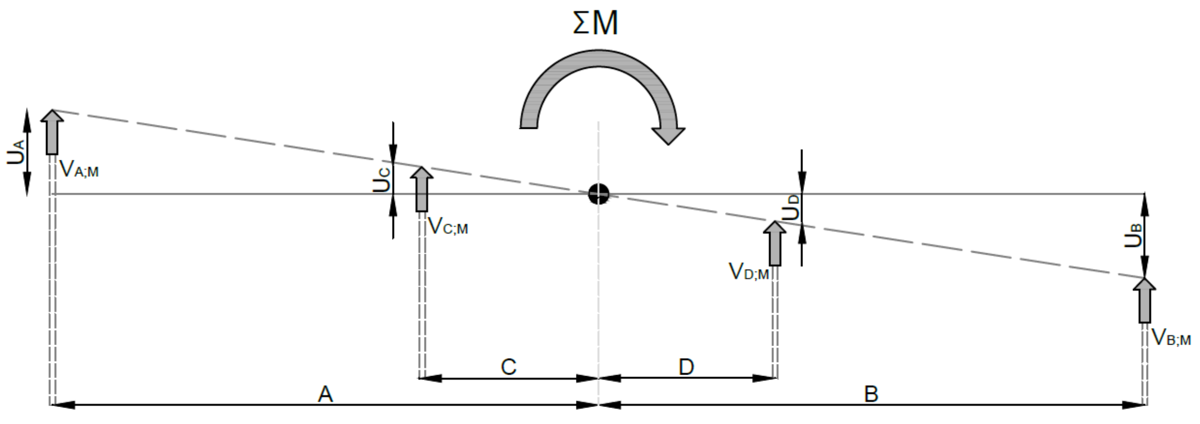


Figure 97: Schematization of axial pile forces induced by the resultant moment

Then, the formula of the maximum occurring compression force on singe piles becomes:

$$F_{s;d} = V_V + V_{A;M;1p} = \frac{\sum V}{\#_{total}} + \frac{\sum M}{[A + B + C(D/B) + D^2/B]\#_{row}}$$

The values of the resultant loads are computed in Table 53. Lever arms are measured in Autocad.

Design load	Value [kN]	Leverarm [m]	Induced moment [kNm]
Vertical			
Floor	2940,30	0,00	0,00
Wall water side	334,13	-4,80	-1603,80
Wall polder-side	189,34	4,80	908,82
Side wall North	236,93	-1,05	-248,77
Side wall South	236,93	-1,05	-248,77
Intermediate wall	177,70	-1,05	-186,59
Glass water-side	147,02	-4,10	-602,76
Glass polder-side	240,57	1,58	378,90
Piles	3525,12	0,00	0,00
Roof	208,20	-2,53	-525,70
Snow (roof)	27,06	-2,53	-68,33
Snow glass (polder-side)	85,14	1,58	134,10
User imposed (roof)	203,62	-2,53	-514,13
User imposed (floor)	1166,22	0,00	0,00
Hydrostatic pressure (rectangle)	-1705,37	0,00	0,00
Hydrostatic pressure (triangle)	-1568,16	-1,65	2587,46
Resultant vertical load	**6444,72**		
Horizontal			
Hydrostatic pressure load	905,99	0,82	745,93
Soil pressure water-side	112,30	0,63	71,12
Resultant horizontal load	**1018,28**		
Resultant moment			**827,48**

Table 53: Computation of resultant design loads

Then, the design value of the maximum compression force on a single pile is:

$$F_{s;d} = \frac{6444{,}72}{60} + \frac{827{,}48}{[4{,}80 + 4{,}80 + 1{,}60(1{,}60/4{,}80) + 1{,}60^2/4{,}80] \cdot 15} = 112{,}58 \text{ kN}$$

Design check

Now that both the compression resistance of a pile and the maximum occurring tension force on a single pile are known, the design check is performed:

$F_{s;d}$ = 112,58 kN
$F_{r;shaft;d}$ = 166,43 kN

Hence:

$$F_{s;d} < F_{r;shaft;d}$$

From this, it is concluded that the compression capacity of the piles is sufficient.

Appendix B.5: Bending strength of concrete elements

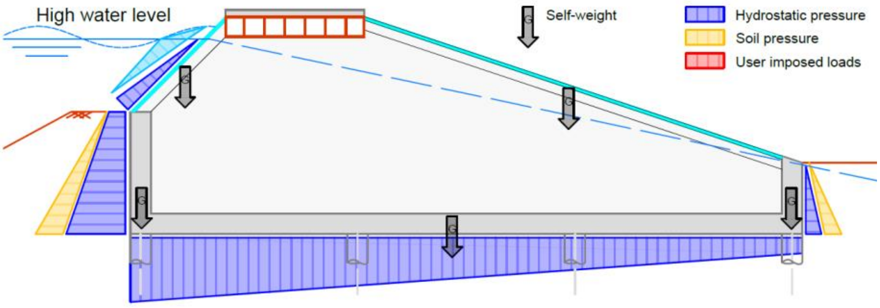
Qualitative analysis of internal forces

The structure of the glass dike consists of concrete elements (floor and walls), which have a thickness of either 0,30 meter or 0,40 meter. All these elements are assessed by performing only one strength check for each thickness. In order to determine which elements are governing, the distribution of internal forces is investigated. The purpose of this analysis is to obtain a qualitative description of the internal forces in the structure as a consequence of the loads. For this, no load factors are applied. In the cross-section of the glass-dike, two load combinations are investigated. These are shown in Table 54.

Load	Load combination 1	Load combination 2
Self-weight	Yes	Yes
Water pressure	High water	Low water
Soil pressure	Yes	Yes
User imposed	No	Yes
Snow	No	No
Wave	Yes	No

Table 54: Load combinations for qualitative analysis of internal forces

For the walls, it is clear that the largest horizontal forces gives the largest bending moments. Gaining qualitative insight into the internal force distribution of the floor is more difficult. Inserting the load combinations above in Matrixframe will give this insight. The first load combination gives the largest upward pressure on the floor. For the second load combination, this upward pressure is reduced to a minimum. To illustrate the effect, bending moment distributions for the load combinations are given respectively in Figure 98 and Figure 99.



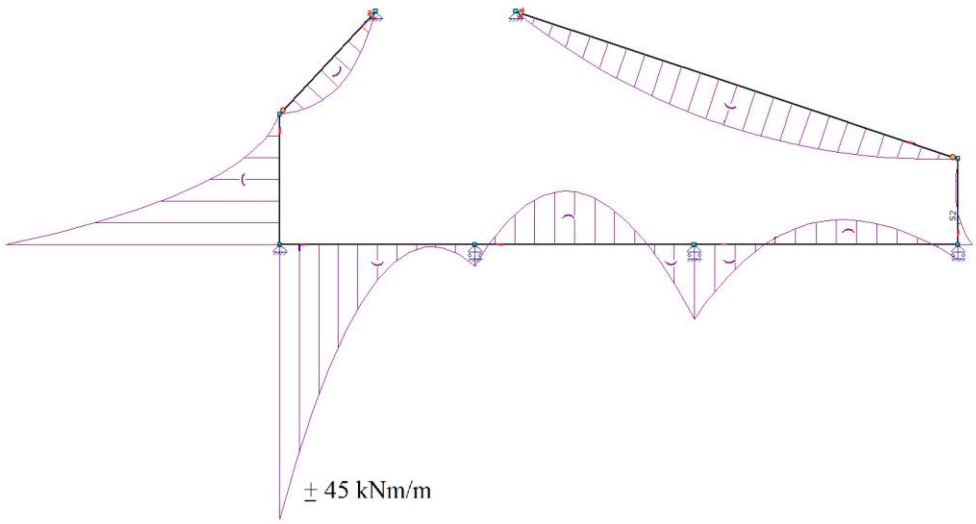


Figure 98: Bending moment distribution for load combination 1

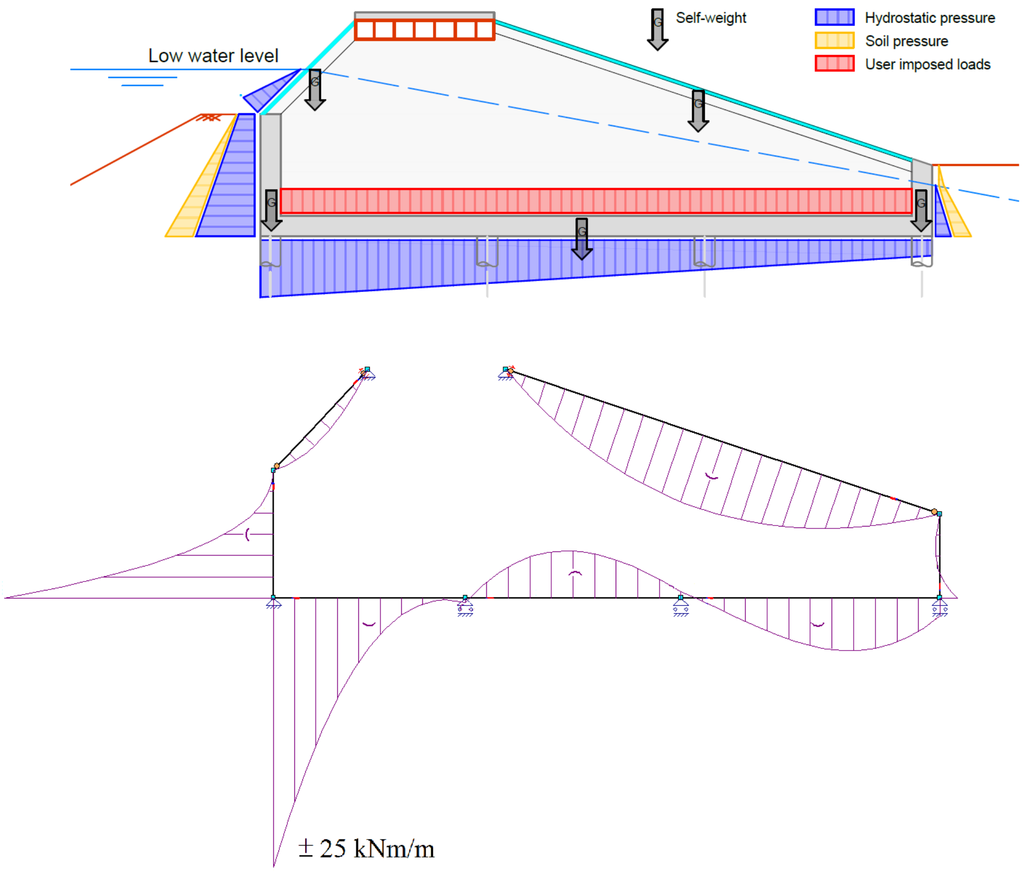


Figure 99: Bending moment distribution for load combination 2

As can be seen, the bending moment in the floor is dominated by the hydrostatic pressure against the wall at the water-side. The bending moment in the wall at the water-side is fully passed on to the floor. Since the wall is thinner than the floor, the strength check is only performed for the wall. Then, it is implicitly verified that the strength of the floor is sufficient when looking at a cross-section of the structure.

For a length-section of the structure, the distribution of internal forces is qualitatively similar. The bending moment in the floor will be dominated by the soil- and water pressure that is acting on the side walls. Since the loads on the side walls are larger than the loads on the wall and glass at the water-side of the structure and since the floor spans (in between the piles) are smaller in the length section than the floor spans in the cross-section, this effect (bending moment in floor dominated by side walls) is even larger. The thickness of the side walls is equal to the thickness of the floor. Therefore, the strength check is performed for the side walls, thereby implicitly verifying the strength of the floor as well.

From the considerations above, it is concluded that the following strength checks should be performed for the concrete elements:

- Bending moment resistance of the side walls (0,40 m thick)
- Bending moment resistance of the wall at the water-side (0,30 m thick)

Bending strength check: wall water-side

The wall is sufficient strong if the ultimate bending strength is larger than the maximum bending moment. This is the case if the design condition below is satisfied. For the wall at the water-side, a cross-section of the structure with a depth of one meter is regarded.

$M_{E;d} \leq M_{R;d}$

$M_{E;d}$	= Design value of the maximum bending moment	[kNm/m]
$M_{R;d}$	= Design value of the ultimate bending strength	[kNm/m]

Loads

The bending moment in the wall is maximum at high water. Part of the wave pressure and hydrostatic pressure that is acting on the glass elements is transferred to the wall. The wall itself is loaded by soil pressure and groundwater pressure. These are all unfavourable loads. The soil level is assumed to be equal to top of the wall. Since there are no permanent loads acting on the wall from inside the structure, there are no favourable loads. The governing load combination and load factors are shown in Table 55 and illustrated in Figure 100.

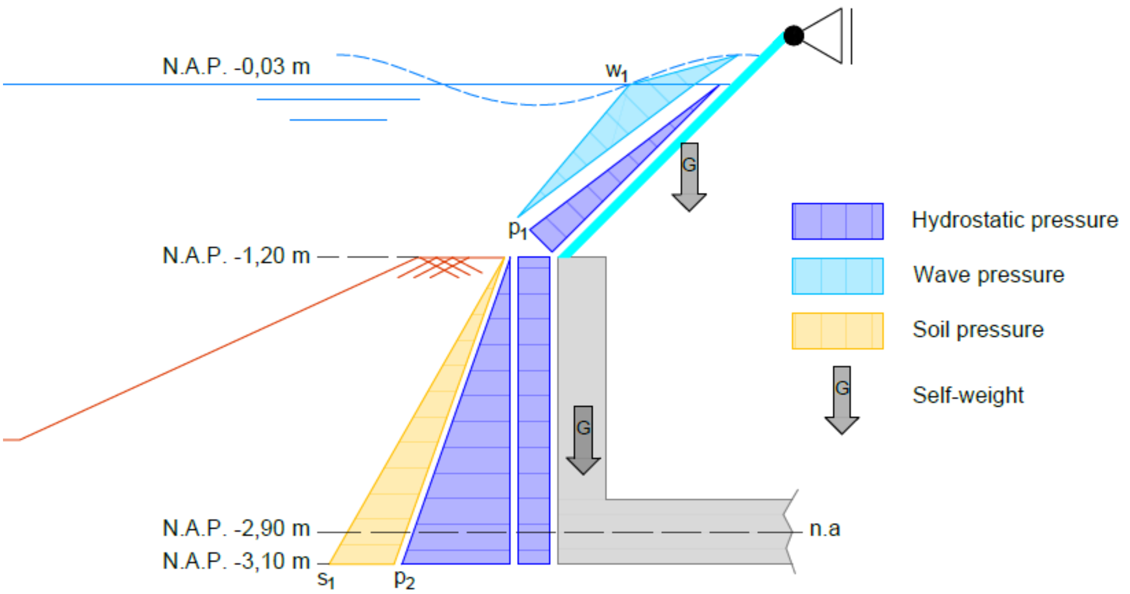


Figure 100: Schematization of loads on the concrete wall at the water-side

Load	Element(s)	Direction	(Un)favourable	Load type
Self-weight	Glass	Vertical	Unfavourable	Permanent
Hydrostatic pressure	Glass	Inclined	Unfavourable	Permanent
Wave pressure	Glass	Inclined	Unfavourable	Variable
Self-weight	Wall water-side	Vertical	Unfavourable	Permanent
Hydrostatic pressure	Wall water-side	Horizontal	Unfavourable	Permanent
Soil pressure	Wall water-side	Horizontal	Unfavourable	Permanent

Table 55: Governing load combination and load factors

For the design of concrete elements, load factors are prescribed by Eurocode 2. These factors are shown in Table 56.

Load factor		Symbol	Value
Permanent	Favourable	γ_G	1,35
	Unfavourable		0,90
Variable	Unfavourable	γ_Q	1,50

Table 56: Load factors (Eurocode 2)

Instead of calculating the loads, only the design values of the pressures are calculated. These values are inserted in Matrixframe to compute the maximum design bending moment. For high water, these values were calculated in the tension capacity check for piles. Only the wave pressure is not yet calculated. In section 6.3.2, it was assumed that the wave pressure is maximum at the water level and zero at the soil level. The wave pressure at the water level is calculated below:

$$w_1 = \gamma_w H_i = 10 \cdot 0{,}1 = 1 \text{ kN/m}^2$$

In which:

w_1	= Wave pressure at the water level	[kN/m^2]
γ_w	= Specific weight of water	[kN/m^3]
H_i	= Height of the incoming wave	[m]

The characteristic- and design values of the pressures are shown in Table 57.

Pressure	Elements/parts	Index	Characteristic value	Load factor	Design value
[-]	[-]	[-]	[kN/m^2]	[-]	[kN/m^2]
Hydrostatic	Glass	p_2	11,70	1,35	15,80
Wave	Glass	w_1	1,00	1,50	1,50
Hydrostatic	Wall water-side	p_2	11,70	1,35	15,80
Hydrostatic	Wall water-side	p_4	30,70	1,35	41,45
Soil	Wall water-side	s_1	3,98	1,35	5,37

Table 57: Characteristic- and design values of the loads

<u>Computation of the bending moment</u>

The pressures are inserted in Matrixframe. The resulting bending moment diagram is shown in Figure 101.

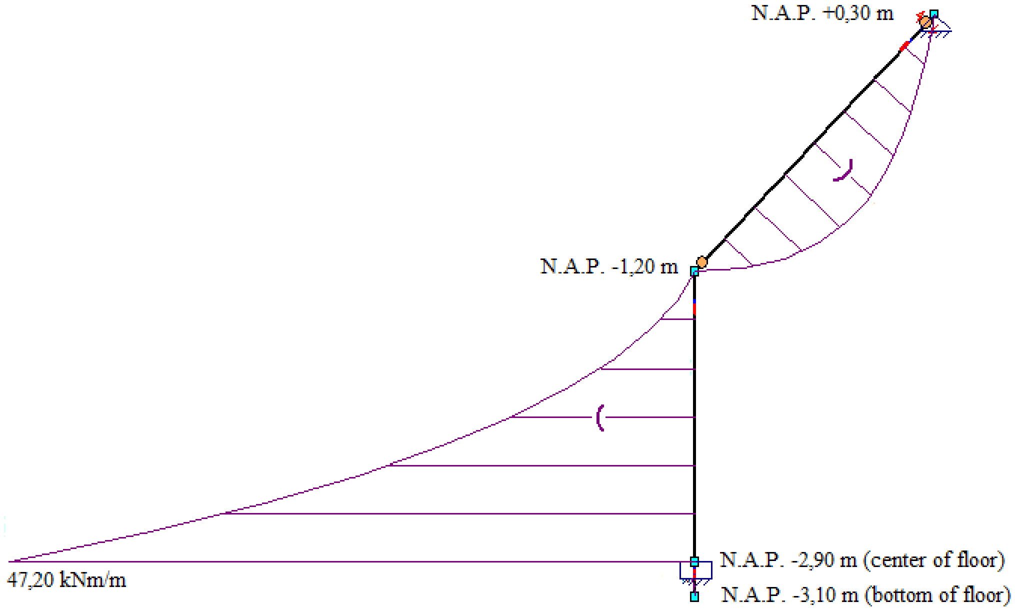


Figure 101: Bending moment diagram for the wall at the water side

As can be seen in the figure, the wall at the water-side is schematized as if it is fully fixed at the height of the centre of the floor. The soil- and water pressure below this height will not contribute to the bending moment in the wall, but will be directly transferred to the floor. The design value of the maximum bending moment in the wall at the water-side is:

$M_{E;d} = 47{,}20$ kNm/m

Bending moment resistance

In order to compute the bending moment resistance, the amount of reinforcement must be determined. This is done by calculating the bending moment resistance a couple of times for different amounts of reinforcement. For this, the formulas that were used were programmed in spreadsheets. The required reinforcement that resulted from these calculations is used as input for the hand calculation of the bending moment resistance as shown below. First, the material properties and dimensions are given in Table 58.

Steel properties	Symbol	Value	Unit
Steel quality	-	B500B	-
Characteristic yielding strength	f_{yk}	500	N/mm^2
Material factor	y_s	1,15	-
Design yielding strength	f_{yd}	435	N/mm^2
Concrete properties			
Concrete quality	-	C20/25	-
Characteristic compressive cube strength	f_{ck}	25	N/mm^2
Material factor	y_c	1,5	-
Design compressive strength	f_{cd}	16,67	N/mm^2
Dimensions			
Thickness	t	300	mm
Width	w	1000	mm
Reinforcement			
Diameter of reinforcement bar	Ø	10	mm
Number of bars	#	10	-
Concrete cover	c	50	mm

Table 58: Properties and required amount of reinforcement bars

The distance between the centre of the reinforcement bars and the outer fiber of the concrete is:

$$d = t - \left(c + \frac{Ø}{2}\right) = 300 - \left(50 + \frac{10}{2}\right) = 245 \text{ mm}$$

The cross-sectional area of the reinforcement bars is:

$$A_s = \frac{Ø^2}{4}\pi\# = \frac{10^2}{4} \cdot 10\pi = 785 \text{ mm}^2$$

This gives a reinforcement percentage of:

$$\rho = (A_s/(t \cdot w)) \cdot 100 = 785/(300 \cdot 1000) \cdot 100 = 0{,}26\%$$

The percentage of reinforcement should be large enough to prevent brittle failure when cracking of the concrete occurs, but not too large as the steel must yield before the concrete reaches its maximum compression strength. This percentage is in between the minimum and maximum reinforcement percentages for concrete quality C20/25 (Vrijling, et al., 2015):

$\rho_{min} = 0{,}15 < 0{,}26 < 1{,}38 = \rho_{max}$

In Ultimate Limit State, the reinforcement is yielding. The tension force in the reinforcement steel is:

$N_s = A_s \cdot f_{yd} = 785 \cdot 435 = 341.304\,N$

Internal equilibrium of normal forces gives the compression force in the concrete:

$N_c = N_s = 341.304\ N$

Eurocode 2 prescribes a factor α = 0,75 for the shape of the compressive zone in the concrete. From this, the height of the concrete compression zone X_u is calculated:

$N_c = \alpha \cdot X_u. \cdot f_{cd} \cdot w = 0{,}75 \cdot X_u \cdot 16{,}67 \cdot 1000 = 12502{,}5 \cdot X_u$

Then:

$X_u = N_c/12502{,}5 = 341.304/12502.5 = 27{,}30\ mm$

The centre of mass of the concrete compression zone is located at a distance of $\beta \cdot X_u$ from the outer fiber of the concrete element. For the factor β, Eurocode 2 prescribes a value of 0,39. Then, the internal leverarm is:

$z = d - \beta \cdot X_u = 245 - 0{,}39 \cdot 27{,}30 = 234{,}35\ mm \approx 234\ mm$

Now, the design value of the bending moment resistance is calculated:

$M_{R;d} = N_s z = 341.304 \cdot 234 = 79.865.136\ Nmm = 79{,}86\ kNm/m$

<u>Design check</u>
Now that both the bending moment and the bending moment resistance are known, the bending strength check is performed:

$M_{E;d}$ = 47,20 kNm/m
$M_{R;d}$ = 79,86 kNm/m

Hence:

$M_{E;d} < M_{R;d}$, thus the bending strength criterion for the wall at the water-side is satisfied.

Since the maximum bending moment in the wall at the polder-side is smaller, also this wall has sufficient bending moment resistance. However, the same amount of reinforcement should be applied.

Bending strength check: Side walls at the transitions with the dike

The side walls at the transitions with the dike are sufficient strong if the ultimate bending strength is larger than the maximum bending moment. For both side walls, the same loads and dimensions apply, so only one strength check is performed. For this check, a length section of the structure with a depth of one meter is regarded. The most unfavourable location on the wall is chosen. This is the location where the loads and the height of the walls are maximum as this results in the largest bending moment. This is clarified in Figure 102.

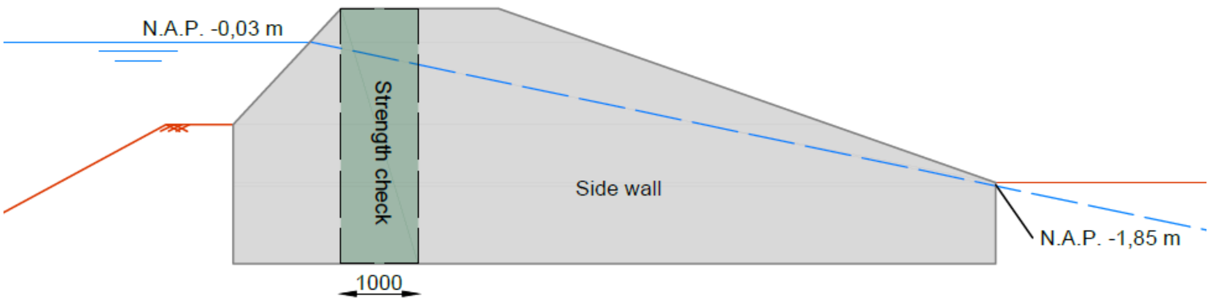


Figure 102: Location on the wall for which the bending strength check is performed

It will be checked if the following design condition is met:

$M_{E;d} \leq M_{R;d}$

$M_{E;d}$	= Design value of the maximum bending moment	[kNm/m]
$M_{R;d}$	= Design value of the ultimate bending strength	[kNm/m]

Loads

The only loads that act on the side walls are the soil pressure and hydrostatic pressure. Since the roof is able to move horizontally with respect to the side walls, the latter are not horizontally supported by the roof. The consequence is that the bending moment in the side walls will be larger and maximum at the connection with the floor. The level of the soil is equal to the height of the roof. For calculation of the hydrostatic pressure, the water level at the point where the strength check is performed is assumed to be equal to the design water level of the Noordvliet. This results in a slightly more conservative design. The loads are illustrated in Figure 103.

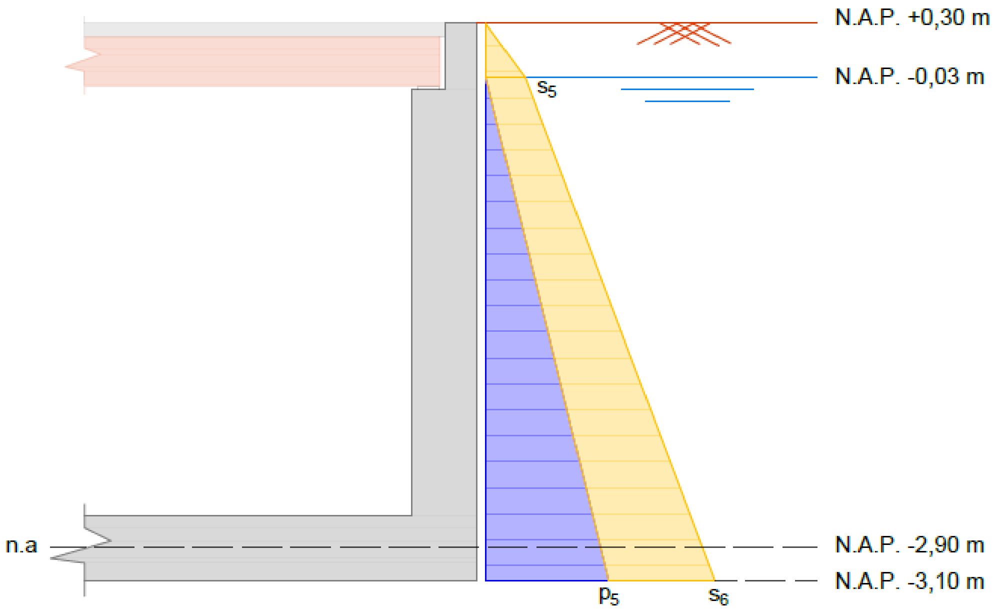


Figure 103: Schematization of loads on a side wall

Just as for the strength check of the wall at the water-side, only the pressures are calculated and not the forces that result from this pressures. These pressures are inserted in Matrixframe, which calculates the bending moment.

Hydrostatic pressure
The hydrostatic pressure at the bottom of the floor is calculated below:

$$p_5 = \gamma_w \cdot h = 10 \cdot (0{,}03 - -3{,}10) = 30{,}70\ \mathrm{kN/m^2}$$

In which:

P_5	= Hydrostatic pressure at the bottom of the floor	[kN/m^2]
γ_w	= Specific weight of water	[kN/m^3]
h	= Depth below the water table	[m]

Soil pressure
Soil pressures are calculated according to the formula`s given in section 6.3.3. The formula of the vertical effective soil pressure is presented below:

$$s'_v = \sum_{i=1}^{n} \gamma_{dry;d,i} d_i + \sum_{j=1}^{m} \gamma_{sat;d,j} d_j - p$$

In which:

s'_v	= Effective vertical soil pressure	[kN/m^2]
$\gamma_{dry;d,i}$	= Design value of the dry specific weight of layer i	[kN/m^3]
d_i	= Thickness of layer i	[m]
$\gamma_{sat;d;j}$	= Design value of the wet specific weight of layer j	[kN/m^3]
d_j	= Thickness of layer j	[m]
p_w	= (ground)water pressure	[kN/m^2]

During construction, the soil is excavated to a level below the bottom of the structure. The soil that is replaced in the last construction phase is entirely composed of clay. Then, there are two layers that should be taken into account when calculating the soil pressure. The first layer, which is the upper layer, consists of dry clay and the second layer, below the water table, consists of saturated clay. Parameters of clay are given in section 5.5.2. Eurocode 7 prescribes material factors for computation of geotechnical loads. Material factors that are relevant for this calculation are given in Table 59, which also gives the design values of the soil parameters.

Soil parameter	Symbol	Characteristic value	Material factor (γ_x)	Design value
Angle of internal friction	φ’	15º	1,15	13º
Specific weight (dry)	γ_{dry}	13 kN/m^3	1,10	14,30 kN/m^3
Specific weight (saturated)	γ_{sat}	13 kN/m^3	1,10	14,30 kN/m^3

Table 59: Material factors and design values of soil parameters

Then:

$$s'_{5;v} = \gamma_{dry;d;1} \cdot d_1 = 14{,}30 \cdot (0{,}30 - -0{,}03) = 4{,}72\ \mathrm{kN/m^2}$$

And:

$$s'_{6;v} = s'_{5;v} + \left(\gamma_{sat;d;2} \cdot d_2 - p_5\right) = 4{,}72 + (14{,}30 \cdot (-0{,}03 - -3{,}10) - 30{,}70) = 17{,}92\ \mathrm{kN/m^2}$$

No horizontal displacement of the structure is allowed, hence the coefficient of neutral soil pressure (Jaky) should be applied:

$$K_n = 1 - \sin\left(\varphi'_d\right) = 1 - \sin(13) = 0{,}775$$

In which:
K_n = Coefficient of neutral soil pressure [-]

Then, the effective horizontal soil pressures are:

$$s'_{5;h;n} = K_n s'_{5;v} = 0{,}775 \cdot 4{,}72 = 3{,}08\ \mathrm{kN/m^2}$$

$$s'_{6;h;n} = K_n s'_{6;v} = 0{,}775 \cdot 17{,}92 = 13{,}88\ \mathrm{kN/m^2}$$

The characteristic and design values of the pressures are shown in Table 60.

Pressure	Elements/parts	Index	Characteristic value	Load factor	Design value
[-]	[-]	[-]	[kN/m^2]	[-]	[kN/m^2]
Hydrostatic	Side wall	p_5	30,70	1,35	41,45
Soil	Side wall	s_5	3,08	1,35	4,16
Soil	Side wall	s_6	13,88	1,35	18,74

Table 60: Characteristic- and design values of the pressures on the side walls

Computation of the bending moment

The pressures are inserted in Matrixframe. The resulting bending moment diagram is shown in Figure 104.

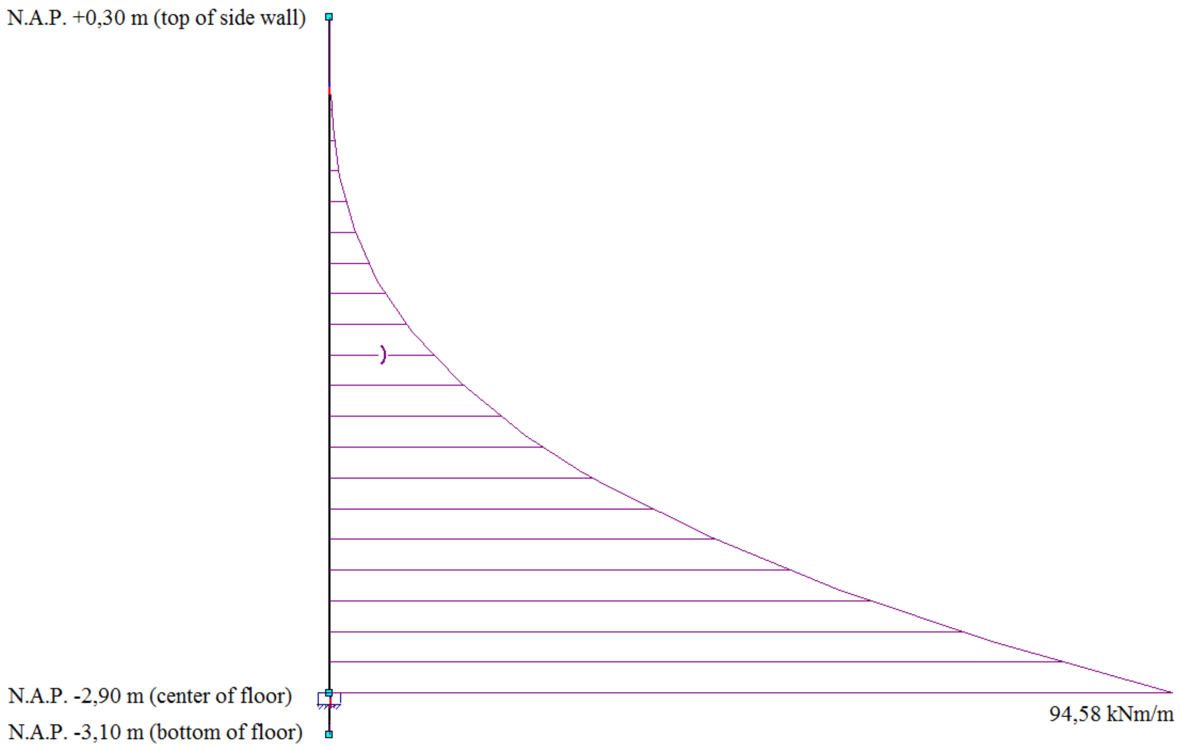


Figure 104: Bending moment diagram of a side wall

As can be seen in the figure, the side wall is schematized as if it is fully fixed at the height of the centre of the floor. The soil- and water pressure below this height will not contribute to the bending moment in the wall, but will be directly transferred to the floor. The design value of the maximum bending moment in the side walls is:

$M_{E;d}$ = 94,58 kNm/m

Bending moment resistance

In order to compute the bending moment resistance, the amount of reinforcement must be determined. This is done by calculating the bending moment resistance a couple of times for different amounts of reinforcement. For this, the formulas that were used were programmed in spreadsheets. The required reinforcement that resulted from these calculations is used as input for the hand calculation of the bending moment resistance as shown below. First, the material properties and dimensions are given in Table 61.

Steel properties	Symbol	Value	Unit
Steel quality	-	B500B	-
Characteristic yielding strength	f_{yk}	500	N/mm^2
Material factor	y_s	1,15	-
Design yielding strength	f_{yd}	435	N/mm^2
Concrete properties			
Concrete quality	-	C20/25	-
Characteristic compressive cube strength	f_{ck}	25	N/mm^2
Material factor	y_c	1,5	-
Design compressive strength	f_{cd}	16,67	N/mm^2
Dimensions			
Thickness	t	400	mm
Width	w	1000	mm
Reinforcement			
Diameter of reinforcement bar	Ø	12	mm
Number of bars	#	10	-
Concrete cover	c	50	mm

Table 61: Properties and required amount of reinforcement bars

The distance between the centre of the reinforcement bars and the outer fiber of the concrete is:

$$d = t - \left(c + \frac{Ø}{2}\right) = 400 - \left(50 + \frac{12}{2}\right) = 344 \text{ mm}$$

The cross-sectional area of the reinforcement bars is:

$$A_s = \frac{Ø^2}{4}\pi\# = \frac{12^2}{4} \cdot 10\pi = 1131 \text{ mm}^2$$

This gives a reinforcement percentage of:

$$\rho = (A_s/(t \cdot w)) \cdot 100 = 1131/(400 \cdot 1000) \cdot 100 = 0{,}28\%$$

The percentage of reinforcement should be large enough to prevent brittle failure when cracking of the concrete occurs, but not too large as the steel must yield before the concrete reaches its maximum compression strength. This percentage is in between the minimum and maximum reinforcement percentages for concrete quality C20/25 (Vrijling, et al., 2015):

$$\rho_{min} = 0{,}15 < 0{,}28 < 1{,}38 = \rho_{max}$$

In Ultimate Limit State, the reinforcement is yielding. The tension force in the reinforcement steel is:

$$N_s = A_s \cdot f_{yd} = 1131 \cdot 435 = 491.985 \text{ N}$$

Internal equilibrium of normal forces gives the compression force in the concrete:

$$N_c = N_s = 491.985 \text{ N}$$

Eurocode 2 prescribes a factor α = 0,75 for the shape of the compressive zone in the concrete. From this, the height of the concrete compression zone X_u is calculated:

$$N_c = \alpha \cdot X_u \cdot f_{cd} \cdot w = 0{,}75 \cdot X_u \cdot 16{,}67 \cdot 1000 = 12502{,}5 \cdot X_u$$

Then:

$$X_u = N_c / 12502{,}5 = 491.985/12502.5 = 39{,}35 \text{ mm}$$

The centre of mass of the concrete compression zone is located at a distance of $\beta \cdot X_u$ from the outer fiber of the concrete element. For the factor β, Eurocode 2 prescribes a value of 0,39. Then, the internal leverage arm is:

$$z = d - \beta \cdot X_u = 344 - 0{,}39 \cdot 39{,}35 = 328{,}65 \text{ mm} \approx 328 \text{ mm}$$

Now, the design value of the bending moment resistance is calculated:

$$M_{R;d} = N_s z = 491.985 \cdot 328 = 161.371.080 \text{ Nmm} = 161{,}37 \text{ kNm/m}$$

Design check

Now that both the bending moment and the bending moment resistance are known, the bending strength check is performed:

$M_{E;d}$ = 94,58 kNm/m
$M_{R;d}$ = 161,37 kNm/m

Hence:

$M_{E;d} < M_{R;d}$, thus the bending strength criterion for the wall at the water-side is satisfied.

Since the maximum bending moment in the floor is smaller than the bending moment in the wall, while it has the same thickness, the bending moment resistance of the floor is sufficient as well, given that the right amount of reinforcement is applied.

Appendix B.6: Bending of the roof

The only loads that act on the roof are the self-weight and the user-imposed load. The characteristic value of the user-imposed load is 4,30 kN/m^2 as readily determined in section 6.3.5. The self-weight is composed of the steel HE300B beams and the weight of additional materials for insulation, cover and other purposes. Since technical specifications for steel beams are given by manufacturers, it is easiest to perform the design check for a single beam. One HE300B beam has a width of 0,30 meter. Therefore, the user imposed load should be multiplied by 0,30. The characteristic- and design values of the loads are given in Table 62. For the weight of the additional materials a conservative estimate is obtained by assuming that the roof is covered by a 0,05 meter thick concrete layer. Off course this is not true in reality.

Load	Elements/parts	Type	Characteristic value	Load factor	Design value
[-]	[-]	[-]	[kN/m]	[-]	[kN/m]
Self-weight	HE300B	Permanent	1,19	1,35	1,61
Self-weight	Additional weight	Permanent	0,39	1,35	0,53
User-imposed	Roof	Variable	1,29	1,50	1,94

Table 62: Loads for the bending strength check of the roof

The roof is schematized as a continues beam that is supported by simple supports on the locations of the side walls and the intermediate wall. Computing the maximum bending moment in the beam for the indeterminate system is possible by hand, but for simplicity Matrixframe software is used instead. The schematization of the beam and the bending moment diagram are both shown in Figure 105.

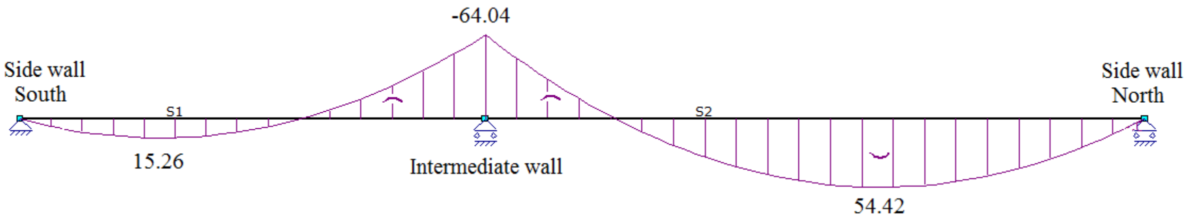


Figure 105: Schematization and bending moment diagram of a roof beam

The largest bending moment is obtained at the intermediate support when variable loads are applied on both spans. The absolute value of the maximum bending moment then is:

$$M_{E;d} = 64.04 \text{ kNm}$$

For the bending moment resistance it is assumed that the beam fails when the steel yields. Yielding starts at the outer fibre at a strain rate of:

$$\varepsilon_y = f_{yd}/E_s = \kappa_y/2h$$

In which:

ε_y	= Strain rate at which yielding starts	[-]
f_{yd}	= Design stress in the steel at which it starts to yield	[kN/m^2]
E_s	= Young's modulus of steel	[kN/m^2]
κ_y	= Curvature in the beam at which it starts to yield at the outer fibre	[1/m]
h	= Height of the beam	[m]

The bending moment resistance is then obtained with:

$$M_{R;d} = E_s I \varkappa_y = \frac{2 I f_{yd}}{h}$$

In which:

$M_{R;d}$	= Design value of the bending moment resistance of the beam	[kNm]
I	= Second moment of inertia in the cross-section of the beam	[m^4]

Substituting the properties of a steel HE300B beam with steel quality S355 (staaltabellen.nl, 2015), results in the following bending moment resistance:

$$M_{R;d} = \frac{2 \cdot 8{,}563 \cdot 10^{-5} \cdot 355 \cdot 10^{3}}{0{,}30} = 202{,}65 \text{ kNm}$$

Hence:

$$M_{E;d} = 64{,}04 < 202{,}65 = M_{R;d}$$

From this, it is concluded that the steel beams have sufficient bending strength.

Maximum displacement (Serviceability Limit State)

Additional to the bending check in ‘Ultimate Limit State’, it is checked if the displacement of the roof is small compared to the length of the largest span. This is a ‘Serviceability Limit State’ that is imposed by the Eurocode. For this, load factors of 1,00 may be used. The maximum displacement of the roof is automatically given when the type of beam is inserted in Matrixframe and the bending moment is computed. The largest displacement is obtained if the variable load is only applied on the largest roof span. This gives:

$$\frac{w}{L} = \frac{0{,}011}{12.1} = \frac{1}{1100}$$

In which:

w	= Maximum displacement in the roof for the applied loads	[m]
L	= Length of the largest roof span	[m]

The relative displacement is much smaller than the requirement stated in the Eurocode (w/L < 1/500), hence also the displacement requirement in SLS is satisfied.

Appendix C: Fault tree for structural failure of the glass dike

Structural failure - part A: Insufficient strength

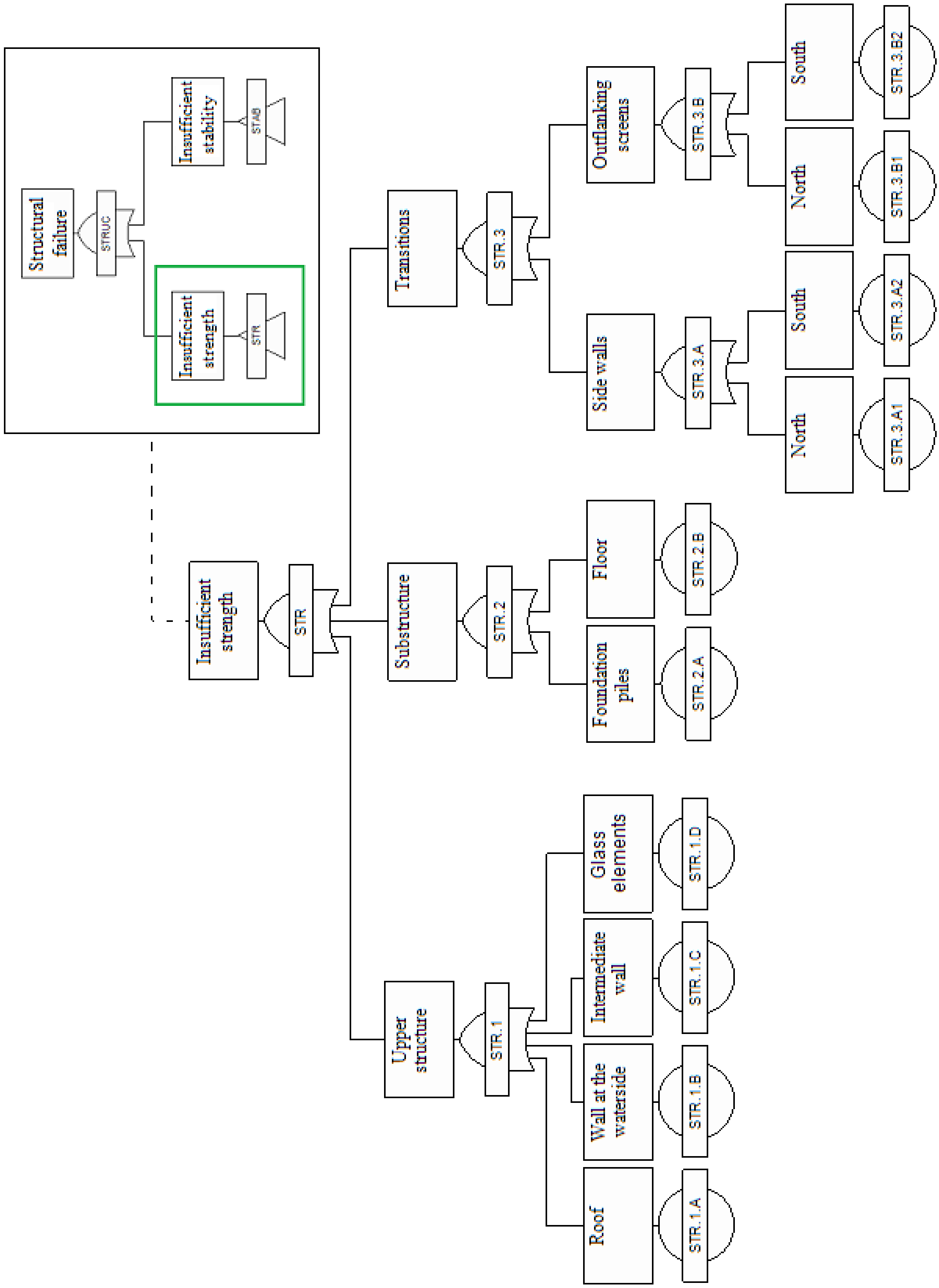


Figure 106: Fault tree for structural failure of the glass dike - part A (strength)

Structural failure - part B: Insufficient stability

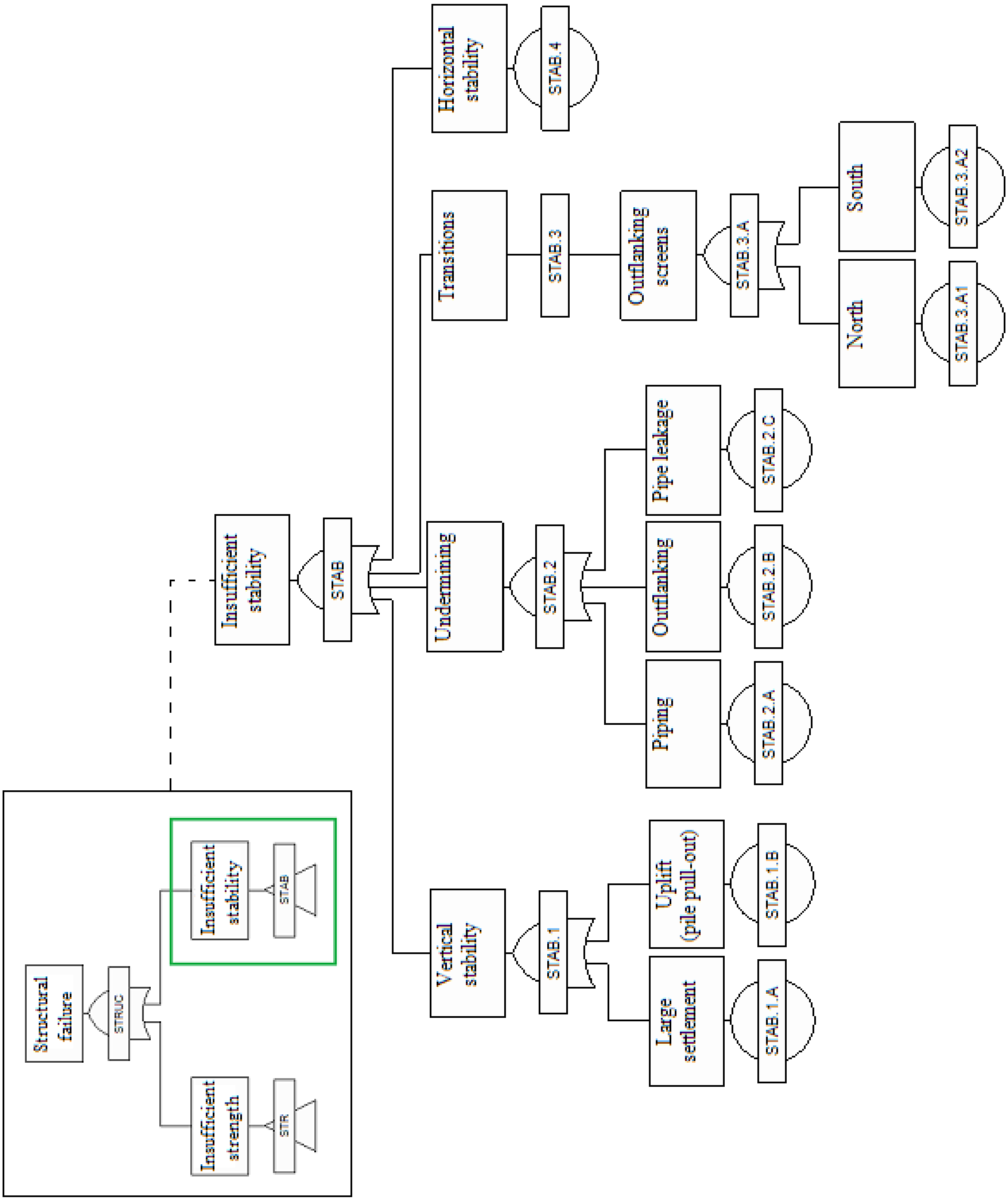


Figure 107: Fault tree for structural failure of the glass dike - part B (stability)

Appendix D: Impact force of a boat collision on a glass element

The contact force between an object and a plate during low velocity impacts can be modelled by a TDOF (two degrees of freedom) spring-mass system. Low velocity impacts are defined as impact with an initial velocity of the object below 50 m/s. In this model, the springs represent the stiffness of the impactor (object) and the stiffness of the target (plate). The masses in the model are the mass of the impactor and the mass of the target (Xu, Li, Lu, & Zhou, 2009). In case of the glass dike, the mass of the boat that hits the glass element is much larger than the mass of the displaced glass during the impact. Therefore, the mass of the glass plate is neglected, thereby reducing the problem to a SDOF system (single degree of freedom). This is illustrated in Figure 108.

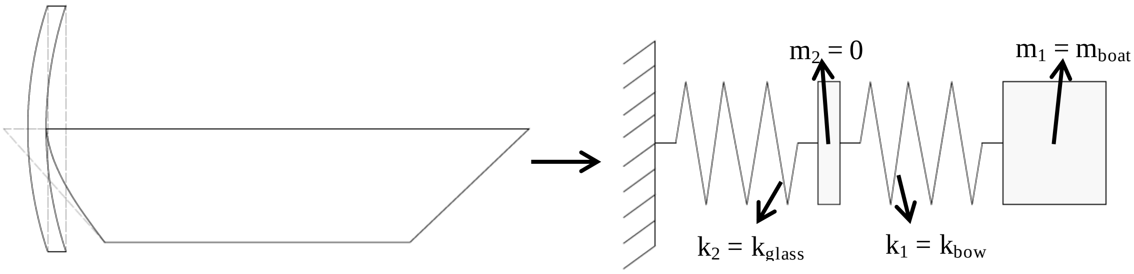


Figure 108: Mass-spring model schematization of a boat collision

In this model, it is assumed that all kinetic energy of the boat is absorbed by elastic deformations in the glass element and the bow of the boat. The boat is assumed not to yaw or to slide off the glass. Furthermore, energy absorption by glass damage and plastic deformations in the boat are neglected. Therefore, this model gives a rather conservative value of the impact force. For the SDOF system the equivalent spring stiffness, which is a combination of the stiffness of the glass plate and the stiffness of the bow, is given by following expression:

$$k_{eq} = \frac{1}{\frac{1}{k_{glass}} + \frac{1}{k_{bow}}}$$

The equation of motion for the SDOF system with the initial conditions is given below:

$$mu''(t) + ku = 0 \qquad u(0) = 0 \qquad u'(0) = V_p$$

In which:

m	= Mass of the boat (since only one mass is left, the index is removed)	[kg]
k	= Equivalent stiffness of the combination of the bow and glass (index removed)	[N/m]
$u(t)$	= Displacement of the mass as a function of time	[m]
V_p	= Velocity of the boat perpendicular to the glass	[m/s]

The maximum displacement of the mass can be obtained by solving the equation of motion for the given initial conditions. From this, the maximum contact force can be deduced. An easier way to obtain the contact force is to apply conservation of energy. It is assumed that all kinetic energy of the boat is converged to potential energy. The latter is provided by elastic deformation of the bow and the glass element, which are represented by the springs. For linear elastic springs the following equation for the conservation of energy applies, in which u_{max} is the maximum displacement of the mass relative to the point of first contact:

$$\frac{1}{2} m {V_p}^2 = \frac{1}{2} k_{eq} {u_{max}}^2$$

Solving the equation above for the maximum displacement and maximum impact force gives:

$u_{max} = V_p\sqrt{m/k}$ and $F_{max} = ku_{max} = V_p\sqrt{km}$

Velocity of the boat perpendicular to the glass element

Only the kinetic energy of the boat that is directed perpendicular to the glass element is absorbed by bending of the glass plate. Therefore, the velocity of the boat needs to be translated to the velocity perpendicular to the glass dike. This velocity depends on the angle of approach, but also on the angle of the glass elements with respect to the horizontal axis as depicted in Figure 109.

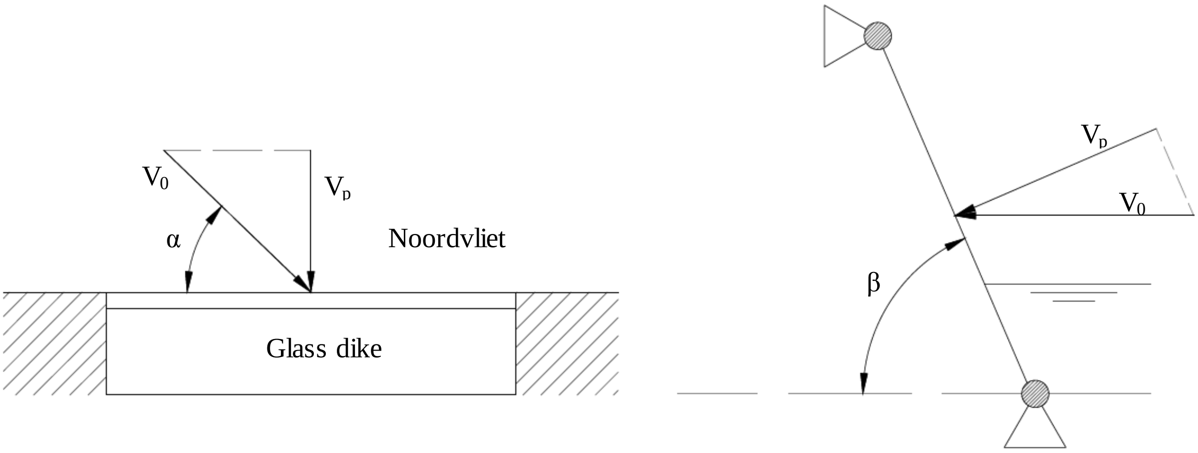


Figure 109: Relation between total velocity of the boat and velocity perpendicular to the glass element

In the semi-probabilistic design, the angle of the glass elements were assumed to be 45 degrees. It is likely that this angle is increased in later design stages, which leads to a larger impact force. To anticipate on this, an angle of 80º is assumed in this calculation. The velocity of the boat perpendicular to the glass elements is then given by the expression below:

$$V_p = V_0 \sin(\alpha)\sin(\beta)$$

In which:

V_p	= Velocity of the boat perpendicular to the glass element	[m/s]
V_0	= Velocity of the boat	[m/s]
α	= Angle between the glass dike and the sailing direction of the boat	[º]
β	= Angle of the glass element with respect to the horizontal axis	[º]

Stiffness of the glass element and the bow of the boat

The stiffness of the glass elements is computed with aid of plate theory. The stiffness of a plate is composed of the bending and shear stiffness. For a completely clamped plate, these are given by the following expressions (Don & Karunarathna, 2013):

$$K_b = \frac{4\pi E h^3}{3(1-\upsilon^2)a^2} \qquad K_s = \frac{4\pi G h}{3}\left(\frac{E}{E-4\upsilon G}\right)\left(\frac{1}{(4/3)+\log(a/a_c)}\right)$$

In which:

K_b	= Bending stiffness of a plate	[kN/m]
K_s	= Shear stiffness of a plate	[kN/m]
E	= Young's modulus of the material	[kN/m^2]
G	= Shear modulus of the material	[kN/m^2]
υ	= Poisson ratio of the material	[-]
h	= Thickness of the plate	[m]
a	= Radius of the plate (is half of the shortest side for rectangular plates)	[m]
a_c	= Half of the plate thickness	[m]

The total stiffness of the plate is computed with the expression below (Don & Karunarathna, 2013):

$$K_{bs} = \frac{K_b K_s}{K_b + K_s}$$

It should be noted that this stiffness only holds for a plate loaded in the centre. The stiffness would be higher if the plate is loaded in some other point, especially at the sides. Higher stiffness results in a larger impact force. For now, it is sufficient to apply the expressions given above.

The glass elements are composed of glass plies and plastic interlayers. The Young's modulus, shear modulus and Poisson ratio for both of these materials need to be taken into account in the stiffness calculation. This can be done by taking the mean values over the thickness of the layers. For this, following expressions should be applied (Xu, Li, Lu, & Zhou, 2009):

$$E_m = \frac{(\Sigma t_g)E_g + (\Sigma t_p)E_p}{\Sigma t_g + \Sigma t_p} \qquad \upsilon_m = \frac{(\Sigma t_g)\upsilon_g + (\Sigma t_p)\upsilon_p}{\Sigma t_g + \Sigma t_p} \qquad G_m = \frac{E_m}{2(1+\upsilon_m)}$$

In which:

E_m	= Mean Youngs modulus	[kN/m^2]
E_g	= Youngs modulus of glass	[kN/m^2]
E_p	= Youngs modulus of the plastic interlayer	[kN/m^2]
υ_m	= Mean Poisson ratio	[-]
υ_g	= Poisson ratio of glass	[-]
υ_p	= Poisson ratio of the plastic interlayer	[-]
G_m	= Mean shear modulus	[kN/m^2]
Σt_g	= Total thickness of the glass plies	[m]
Σt_p	= Total thickness of the plastic interlayers	[m]

The bow stiffness of the boat is expressed as a percentage of the stiffness of the glass element. This is explained in section 8.2.4. The equivalent spring stiffness in the SDOF model that results is shown below:

$$k = \frac{1}{\frac{1}{K_{bs}} + \frac{1}{xK_{bs}}}$$

In this expression, x is defined with a percentage.

__Parameters__

Parameter	Symbol	Value	Unit
Angle of the glass element with respect to the horizontal axis	β	80	°
Plate thickness	h	0,085	m
Plate radius (=half of the shortest side)	a	1,00	m
Total thickness of glass plies	Σt_g	0,08	m
Total thickness of plastic interlayers	Σt_p	0,005	m
Young's modulus of glass	E_g	70.000.000	kN/m^2
Young's modulus of SentryGlas (interlayers) during impact	E_p	692.000	kN/m^2
Poisson ratio of glass	υ_g	0,23	-
Poisson ratio of SentryGlas (interlayers)	υ_p	0,49	-

Table 63: Values of parameters that are not part of the sensitivity analysis

In order to get an impression of the sensitivity of the impact force to varying parameters, the calculation explained above is repeated many times for varying velocity, mass, stiffness and angle of approach of the boat. Therefore, the above explained calculation procedure is not performed here, but a spreadsheet has been used instead. The results are shown in section 8.2.4. For some parameters however, the values are kept constant for all computations. These values are shown in Table 63.

Appendix E: Limit state functions for overloading of glass elements

Appendix E.1: Derivation of the limit state functions

The total number of limit states in the Monte Carlo simulation is 1320. Half of these limit states concern rectangular glass sections in glass plies with an exposed surface, while the other half concern rectangular glass sections in glass plies that do not have an exposed surface (not an outer ply). The latter become relevant after fracture of an outer ply. A large part of these limit states is computed with identical limit state functions. In fact, only three limit state functions are required, concerning:

- Fracture in a rectangular glass section at the submerged support
- Fracture in a rectangular glass section in the field
- Fracture in a rectangular glass section at the dry support

The general principle of a limit state function is given by following expression:

$$Z = R - S$$

In which:
Z = Limit state function
R = Strength part of the limit state (Resistance)
S = Load part of the limit state (Solicitation)

Failure has occurred if Z < 0. For fracture in a glass ply, R is replaced by the practical tensile strength of glass and S is replaced by the tensile stress at the surface of the glass element. This gives the following limit state function:

$$Z = f_g - \sigma_s$$

In which:

f_g	= Practical tensile strength of glass	[MPa]
σ_s	= Tensile stress at the glass surface	[MPa]

The practical tensile strength of the glass is a stochastic variable and does not need to be computed with input of other stochastic variables. This is not the case for the tensile stress at the glass surface as this depends on the hydrostatic pressure, dimensions of the glass element and the bonding that is provided by the interlayers. In section 8.4.1, it is explained that the glass elements are schematized as beams that are clamped at two sides. This is illustrated in Figure 110.

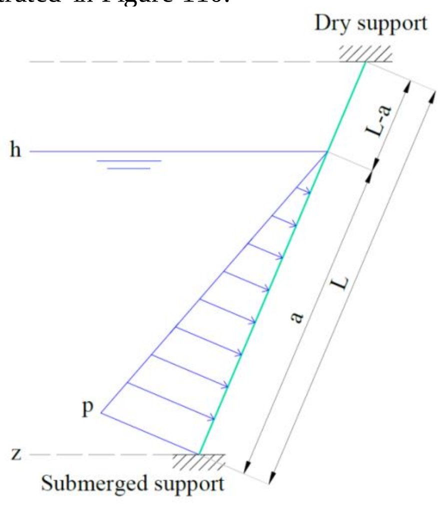


Figure 110: Schematization of the glass elements loaded by hydrostatic pressure

Symbols in Figure 110:

h	= Height of the water level relative to N.A.P.	[m]
z	= Height of the submerged support relative to N.A.P.	[m]
p	= Maximum value of the hydrostatic pressure	[MN/m]
a	= Submerged length of the glass element	[m]
L	= Total length of the glass element	[m]

Stress at the surface of a glass element

Stresses at the surface of a glass panel are proportional to the bending moment and inversely proportional to the thickness of the element. Since the panels are not loaded in axial direction (no normal force), it is allowed to use the following expression to obtain stresses at the glass surface:

$$\sigma_s(x) = \left|\frac{6M(x)}{b \cdot {t_{eq}}^2}\right| = \sqrt{\left(\frac{6M(x)}{b \cdot {t_{eq}}^2}\right)^2}$$

In which:

x	= Location in the glass element measured from the submerged support	[m]
$\sigma_s(x)$	= Tensile stress at the surface of the glass at x	[MPa]
M(x)	= Bending moment in the glass at x	[MNm]
b	= Width of the glass element	[m]
t_{eq}	= Equivalent glass thickness	[m]

For simplicity, the width of a glass element is set to one meter. Although the elements will be wider in reality, this has no consequences for the stresses that are computed. However, note that the hydrostatic pressure should also be computed for one meter width. Also note that absolute stress values are computed. The reason for this is that tensile stresses are indicated by a positive sign and for the above given schematization of the glass, compression stress on one side of the element implies tensile stress on the other side of the element. By computing absolute values, tensile stresses in both outer glass plies in the elements are taken into account instead of just one. Since compression on one side of the panel implies tension on the other side of the panel, the total in tension loaded surface area of the two outer glass plies are exactly equal to the surface area of one glass ply. Therefore, this approach will not lead to an over- or underestimate of the failure probability.

Bending moment diagram

In order to reproduce the bending moment diagram for a loaded glass panel, use is made of the following relations (for beams):

$$M(x) = EI\kappa(x); \qquad V(x) = \frac{dM(x)}{dx}; \qquad q(x) = -\frac{dV(x)}{dx}; \qquad \kappa(x) = \frac{d\varphi(x)}{dx}; \qquad \varphi(x) = -\frac{dw(x)}{dx};$$

In which:

x	= Location in the glass element measured from the submerged support	[m]
M(x)	= Bending moment in the glass element at x	[MNm]
V(x)	= Shear force in the glass element at x	[MN]
q(x)	= Distributed load acting perpendicular on the glass element at x	[MN/m]
φ(x)	= Rotation of the glass element (relative to unloaded situation) at point x	[-]
Κ(x)	= Curvature of the glass element at x	[1/m]
w(x)	= Displacement of the glass element at x	[m]
EI	= Bending stiffness of the glass element	[MNm^2]

From these relations, the following differential equation is deduced:

$$EI\frac{d^4w(x)}{dx^4} = q(x)$$

By solving this equation, the displacement field of the glass element is obtained, which in turn can be used to compute the bending moment diagram. However, since every glass element consists of a part that is loaded by hydrostatic pressure and a part that is not loaded, solving the differential equation for the entire length of the element will give a discontinuity at the water line. Therefore, the glass element is split up in two parts, each described by a separate differential equation. Above the water line, the distributed load is equal to zero. For the submerged part of the glass, the distributed load is equal to the hydrostatic pressure. This leads to the following two differential equations.

For the submerged part of the glass element ($x < a$): $$EI\frac{d^4w_1(x)}{dx^4} = p\left(1 - \frac{x}{a}\right)$$

For the part of the glass element above the water line ($x > a$): $$EI\frac{d^4w_2(x)}{dx^4} = 0$$

Expressions that apply to the submerged part of the glass are indicated with index 1. Expressions that apply to the part of the glass element situated above the water line is indicated with index 2. Integration of the differential equations gives the following expressions for the displacement field:

For $x < a$: $$w_1(x) = -\frac{px^5}{120EIa} + \frac{px^4}{24EI} + \frac{C_1x^3}{6} + \frac{C_2x^2}{2} + C_3x + C_4$$

For $x > a$: $$w_2(x) = \frac{C_5x^3}{6} + \frac{C_6x^2}{2} + C_7x + C_8$$

Expressions for the integration constants ($C_{1t/m8}$) are obtained by solving the system of linear equations that is obtained by substituting the boundary conditions below in the expressions for the displacement field:

At the submerged support ($x = 0$):
$w_1 = 0$ and $\varphi_1 = 0$

At the dry support ($x = L$):
$w_2 = 0$ and $\varphi_2 = 0$

At the water line ($x = a$):
$w_1 = w_2$; $\varphi_1 = \varphi_2$; $M_1 = M_2$; and $V_1 = V_2$

This results in 8 equations with 8 unknowns. Solving these equations by hand is a rather extensive procedure. Therefore, Maple (software) has been used to solve the equations for the integration constants. The result is shown in Table 64.

The functions for the displacement field in a glass element, loaded by hydrostatic pressure, are now defined. The functions for the bending moment diagram are obtained by substituting the displacement functions into the following expression:

$$M(x) = EI\frac{d^2w(x)}{dx^2}$$

This results in:

For $x < a$: $$M_1(x) = \frac{px^3}{6a} - \frac{px^2}{2} - EI(C_1x + C_2)$$

For $x > a$: $$M_2(x) = -EI(C_5x + C_6)$$

Integration constant (submerged part)	Expression	Integration constant (dry part)	Expression
C_1	$\frac{ap(10L^3-5La^2+2a^3)}{20L^3EI}$	C_5	$\frac{pa^3(5L-2a)}{20L^3EI}$
C_2	$\frac{pa^2(10L^2-10La+3a^2)}{60L^2EI}$	C_6	$-\frac{pa^3(10L-3a)}{60L^2EI}$
C_3	0	C_7	$\frac{pa^3}{24EI}$
C_4	0	C_8	$-\frac{pa^4}{120EI}$

Table 64: Expressions for the integration constants

Locations of maximum stresses

Stresses at the glass surface, as qualitatively depicted in the right part of Figure 111, are now described by expressions. Note that stresses vary over the length (height) of the glass surface, but are constant in direction of the width. As mentioned at the start of this appendix, three events are distinguished for failure of a glass ply:

- Fracture in a rectangular glass section at the submerged support
- Fracture in a rectangular glass section in the field
- Fracture in a rectangular glass section at the dry support

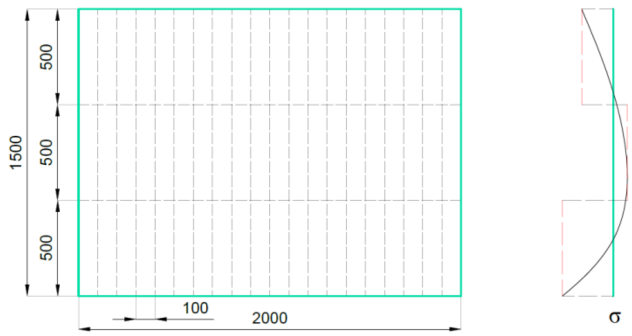


Figure 111: Rectangular sections in a glass ply for which the strength is checked separately

For every rectangular glass section, the maximum tensile stress in the section is compared to strength of that particular section. In Figure 111, it can directly be seen that for fracture in rectangular sections next to the supports, the stresses should be computed directly at the (fixed) supports. So $M_1(0)$ and $M_2(L)$ should be computed and substituted in the function for the stress ($\sigma(x)$).

For fracture of glass sections in the field, the maximum bending moment in the field must be computed. This bending moment is located in the submerged part of the glass, but the exact location is not known yet. For this, the following expression is solved for x:

$$\frac{dM_1(x)}{dx}=0$$

Again, this is done with help of Maple. The maximum field moment is located at:

$$x=-\frac{-ap+\sqrt{2C_1EIap+a^2p^2}}{p}$$

To obtain the maximum stress in the field, this expression for x is substituted in the expression for the bending moment in the submerged part of the glass element, which in turn is substituted in the function for the stress. In summary, the following bending moments are computed and substituted in the function for the stress:

Event	Bending moment
Fracture of the glass ply at the submerged support	$M_1(0)$
Fracture of the glass ply in the field (centre area)	$M_1\left(-\frac{-ap+\sqrt{2C_1EIap+a^2p^2}}{p}\right)$
Fracture of the glass ply at the support near the roof	$M_2(L)$

Table 65: Governing bending moments for the rectangular glass sections

Glass thickness

The thickness of the glass is computed with following expression:

$$t=\sum_{i=1}^{n} t_{p;i}+\sum_{j=1}^{m} t_{f;j}$$

In which:

t	= Total thickness of the glass element	[m]
$t_{p;i}$	= Thickness of glass ply i	[m]
$t_{f;i}$	= Thickness of interlayer j	[m]

For a glass element that consists of four glass plies, this becomes:

$$t=t_{p;1}+t_{p;2}+t_{p;3}+t_{p;4}+t_{f;1}+t_{f;2}+t_{f;3}$$

However, the equivalent thickness of the glass is used to obtain maximum stresses at the glass surface. This is computed with the following expression:

$$t_{eq}=r\cdot t$$

In which:

t_{eq}	= Equivalent thickness of the glass panel	[m]
r	= Conversion factor	[-]

For a glass panel that consists of four glass plies with equal thickness, the conversion factor is in between 0.5 and 1.0. For r = 0.5, there is no bonding between the plies. The conversion factor depends on temperature and duration of loading. However, it should not be mistaken with the bonding factor as described in NEN2608 as this is computed differently. In the limit state functions, the conversion factor will be treated as a stochastic variable. So instead of applying functions for the conversion factor, which include temperature and load duration, it is attempted to obtain a safe distribution function that describes the conversion factor statistically. This is done later in this appendix.

Bending stiffness

For computation of stresses at the glass surface, the equivalent thickness method is applied. Then, the laminated glass element is treated as a monolithic glass plate with a thickness equal to the equivalent thickness of the laminated panel. For the bending stiffness, this means that a glass element is treated as a homogeneous glass plate. Since the glass panels are schematized with supports on two sides only, they can be treated as beams with rectangular cross-sections. Then, the bending stiffness is computed with following expression:

$$EI=E\left(\frac{1}{12}\cdot b\cdot {t_{eq}}^3\right)$$

In which:

EI	= Bending stiffness of the glass element	[MNm^2]
E	= Young's modulus of glass	[MPa]
b	= Width of the glass element	[m]
t_{eq}	= Equivalent glass thickness	[m]

As mentioned before, stresses are computed for one meter width of the glass, so one meter is taken for b, even if the real width is larger.

Hydrostatic pressure

The maximum hydrostatic pressure is located at the lower support. This pressure is computed with:

$$p = y_w(h - z)b$$

In which:

p	= Maximum hydrostatic pressure	[MN/m]
y	= Volumetric weight of water	[MN/m^3]
h	= Height of the water level relative to N.A.P.	[m]
z	= Height of the submerged support relative to N.A.P.	[m]
b	= Width of the glass element	[m]

The length of the glass element that is loaded by hydrostatic pressure is computed with:

$$a = (h - z)/\sin(\beta)$$

In which:

a	= Length of the submerged part of the glass element	[m]
β	= Angle of the glass element with respect to the horizontal axis	[rad]

Overview of expressions and variables in the limit state functions

All variables and expressions that are needed to compose the limit state functions are defined. However, substituting all expressions in order to obtain limit state functions with only stochastic base variables, results in very long and complex expressions. This is not done. Instead, an overview of the expressions and variables in the limit state functions is given Table 66.

Expression	Submerged support	Field	Dry support
General limit state function	$Z = f_g - \sigma_s$		
Stress at the glass surface	$\sigma_s(x) = \left\lvert\frac{6M(x)}{b \cdot t_{eq}{}^2}\right\rvert = \sqrt{\left(\frac{6M(x)}{b \cdot t_{eq}{}^2}\right)^2}$		
Bending moment	$M_1(x) = \frac{px^3}{6a} - \frac{px^2}{2} - EI(C_1x + C_2)$		$M_2(x) = -EI(C_5x + C_6)$
Location of maximum stress	$x = 0$	$x = -\frac{-ap + \sqrt{2C_1EIap + a^2p^2}}{p}$	$x = L$
Integration constant 1	$C_1 = -\frac{ap(10L^3 - 5La^2 + 2a^3)}{20L^3EI}$		-
Integration constant 2	$C_2 = \frac{pa^2(10L^2 - 10La + 3a^2)}{60L^2EI}$		-
Integration constant 5	-		$C_5 = \frac{pa^3(5L - 2a)}{20L^3EI}$

Integration constant 6	-	$C_6 = -\dfrac{pa^3(10L - 3a)}{60L^2EI}$
Bending stiffness	$EI = E\left(\dfrac{1}{12} \cdot b \cdot {t_{eq}}^3\right)$	
Equivalent glass thickness	$t_{eq} = r \cdot t$	
Real glass thickness	$t = \sum_{i=1}^{n} t_{p;i} + \sum_{j=1}^{m} t_{f;j}$	
Max. hydrostatic pressure	$p = y_w(h - z)b$	
Submerged length of the glass	$a = (h - z)/\sin(\beta)$	

Table 66: Overview of expressions in the limit state functions

Limit state functions are obtained if expressions in Table 66 are substituted step by step in upward direction, starting from the lowest row in the table. This will result in expressions with the following stochastic variables only:

f_g	= Tensile strength of the glass	[MPa]
E_g	= Young's modulus of glass	[MPa]
r	= Conversion factor to obtain the equivalent thickness	[-]
h	= Height of the water level relative to N.A.P.	[m]
z	= Height of the bottom of the glass plate relative to N.A.P.	[m]
y_w	= Specific weight of water	[MN/m^3]
L	= Length (/height) of a glass element	[m]
b	= Width of the glass element (only in computation)	[m]
$t_{p;i}$	= Thickness of a glass ply	[m]
$t_{f;j}$	= Thickness of a foil layer	[m]
β	= Angle of the glass with respect to the horizontal axis	[Rad]

Appendix E.2: Statistical descriptors of stochastic variables

For probabilistic calculations, a probability distribution function and corresponding statistical parameters needs to be assigned to every stochastic variable. The derivation (or argumentation) of these functions and parameters is shown below.

Tensile strength of the glass
The tensile strength of glass is described by Weibull functions. Since this is extensively elaborated in section 4.3, this is not further explained here. Weibull parameters for the strength of heat-strengthened glass are obtained with help of Matlab and are based on four point bending strength tests (Veer, Louter, & Bos, 2009).

Probability distribution function	Weibull
Scale parameter (u)	114,76
Shape parameter (k)	3,93

Young's modulus of glass
The Young's modulus is a material property that is fairly constant. For the glass, a normal distribution is assumed. The mean value is taken to be 70.000 MPa (see section 4.1), while for the standard deviation a value of 100 MPa is chosen. This is only 0,14 percent of the mean value.

Probability distribution function	Normal
Mean (μ)	70.000
Standard deviation (σ)	100

Conversion factor to obtain the equivalent thickness
For a glass panel that consists of four glass plies with equal thickness, the conversion factor is in between 0.5 and 1.0. For r = 0.5, there is no bonding between the plies. The conversion factor depends on temperature and duration of loading. If one ply in a panel fractures, three glass plies remain intact, which are bonded by two interlayers. Then, the relative number of interlayers becomes smaller and hence, the conversion factor will increase (in the extreme case of one glass ply there are no interlayers, so r = 1.0). For impact loads at normal temperature, the conversion factor is almost equal to one, while for long term loads at very high temperatures (above 50 degrees Celsius) it approaches 0.5. The glass elements are permanently loaded by hydrostatic pressure, but the temperature will always be below 30 degrees Celsius (glass temperature is controlled by the temperature of water in the Noordvliet and air inside the glass dike). Therefore, a mean value of 0.6 is assumed. Furthermore, it is assumed to be normally distributed with a standard deviation of 5 %, which is equal to 0.03.

Probability distribution function	Normal
Scale parameter (μ)	0,6
Shape parameter (σ)	0,03

It is expected that these assumptions are safe. However, since there is no evidence, this should be investigated more thoroughly. Another solution would be to assume no bonding between the glass plies at all. Since insight in the effect of bonding is desired, this is not done.

Height of the water level relative to N.A.P.
The height of the water level varies over time. Since the result of the probabilistic calculation is a yearly failure probability, yearly maximum water levels are regarded. These should be described by an extreme value distribution. For this, a shifted exponential distribution is chosen. Data of the water level in the Noordvliet is only present for the years 2010 up to 2015. The yearly maxima for these years are shown in Table 67.

Year	Maximum water level
2010	-0,37 m N.A.P.
2011	-0,35 m N.A.P.
2012	-0,37 m N.A.P.
2013	-0,40 m N.A.P.
2014	-0,38 m N.A.P.
2015	-0,40 m N.A.P.

Table 67: Yearly maxima of the water level in the Noordvliet

The average yearly maxima is -0,38 m N.A.P. This value will be used as minimum in the shifted exponential distribution. Exponential distribution functions are described by a rate parameter. The value of this parameter is obtained by calibrating the distribution function such, that a probability of exceedance of 1/100 per year is obtained for a water level of -0,03 m N.A.P. This way, the probability distribution of the water level corresponds to the water level exceedance probability as prescribed by the Flood Safety standard. In Figure 112, a graph is shown of this probability distribution function.

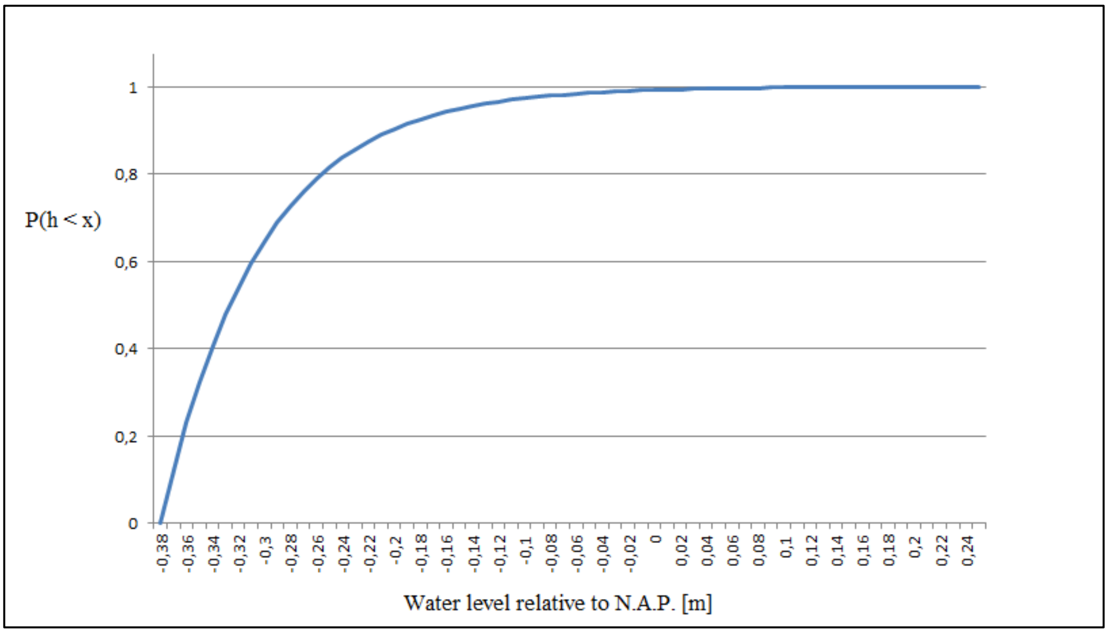


Figure 112: Shifted exponential probability distribution function for the water level

For the yearly maximum water level, the following statistical descriptors are obtained:

Probability distribution function	Shifted exponential
Rate parameter (λ)	13
Minimum (ε)	-0,38

Height of the bottom of the glass elements relative to N.A.P.

The height of the bottom of the glass panels is defined in the design. Incorrect placement of the panels may cause deviation from this height. Large construction errors are usually solved, but nothing is done if deviations from the design do not exceed pre-defined tolerances. Therefore, it is assumed that the glass elements are placed at the correct height (bottom at -1,2 m N.A.P.) with maximum 0,01 meter deviation. This is taken into account by assuming a normal distribution. The standard deviation is approximately half of the maximum deviation. This result in the following statistical properties:

Probability distribution function	Normal
Mean (μ)	-1,2
Standard deviation (σ)	0,005

Specific weight of water

At 4 degrees Celcius, the specific weight of pure fresh water is 0,010 MN/m^3. Most of the year, the temperature of water in the Noordvliet will be higher, resulting in a lower specific weight. However, suspended sediments cause the specific weight to increase. The effect of temperature and suspended sediments are small. Therefore, a normal distribution with a standard deviation of 0.5 % is assumed.

Probability distribution function	Normal
Mean (μ)	0,01
Standard deviation (σ)	0,00005

Length of a glass element

Deviations from the length of glass panels are the result of production processes. In practice, the length is always in between some tolerances. For larger deviations, the glass panel is not accepted/approved for use in the glass dike. Again, this is taken into account by applying a normal distribution:

Probability distribution function	Normal
Mean (μ)	1,5
Standard deviation (σ)	0,001

Width of a the glass element

For the physical model of the glass elements (see section 8.4.1), stresses at the glass surface are constant in horizontal direction. Therefore, the width of a glass element has no influence on the stresses that are computed. For simplicity, a width of one meter is used. There is no need to take into account uncertainty. Note that the failure probability will increases if the total width of glass in the glass dike is increased. This is taken into account by performing probabilistic calculations for more rectangular glass section. The width parameter described here, is only used to compute stresses in the glass.

Probability distribution function	Deterministic
Value	1,00

Thickness of glass plies and interlayers

For the thickness of glass plies, a normal distribution is assumed. Tolerances for the thickness of glass plies are given in NEN2608. For a ply with a thickness of 0,01 meter, the maximum deviation is 0,0003 meter (0,3 mm tolerance). The standard deviation is approximately half of the maximum deviation.

Probability distribution function	Normal
Mean (μ)	0,01
Standard deviation (σ)	0,00015

Similar to the thickness of glass plies, a normal distribution is assumed for the thickness of interlayer foils. The thickness of SentryGlas foil is 0,00152 meter (1,52 mm). Since information on tolerances is not available, an estimate has been made for the standard deviation.

Probability distribution function	Normal
Mean (μ)	0,00152
Standard deviation (σ)	0,00002

Angle of the glass elements with respect to the horizontal axis

If the angle of a glass elements with respect to the horizontal axis deviates from the angle as defined in the design, this is caused by placement (construction) errors. A similar reasoning holds for the height of the bottom of the glass elements. Large construction errors are usually solved, but nothing is done if deviations from the design do not exceed pre-defined tolerances. Therefore, it is assumed that glass elements are placed at the correct angle (1.107 Rad) with a standard deviation of 1 %.

Probability distribution function	Normal
Mean (μ)	1,107
Standard deviation (σ)	0,01107

Appendix E.3: Validation of the limit state functions

The limit state functions for fracture in a glass ply are long and complex. Expressions for the bending moment diagram under- and above the water line were plotted with Maple. This is shown in Figure 113, in which the blue line is only correct for bending moments in the glass above the water line (x > a), while the red line is only correct for bending moments below the water line (x < a). These lines for the bending moment were obtained by substituting the mean values of stochastic variables as deterministic values into the expressions. For the water level, the design value of -0,03 m N.A.P. (according to the Flood Safety Standard) was used instead of the mean. The same parameters (and values) were used to compute the bending moment diagram with Matrixframe. This is shown in Figure 114.

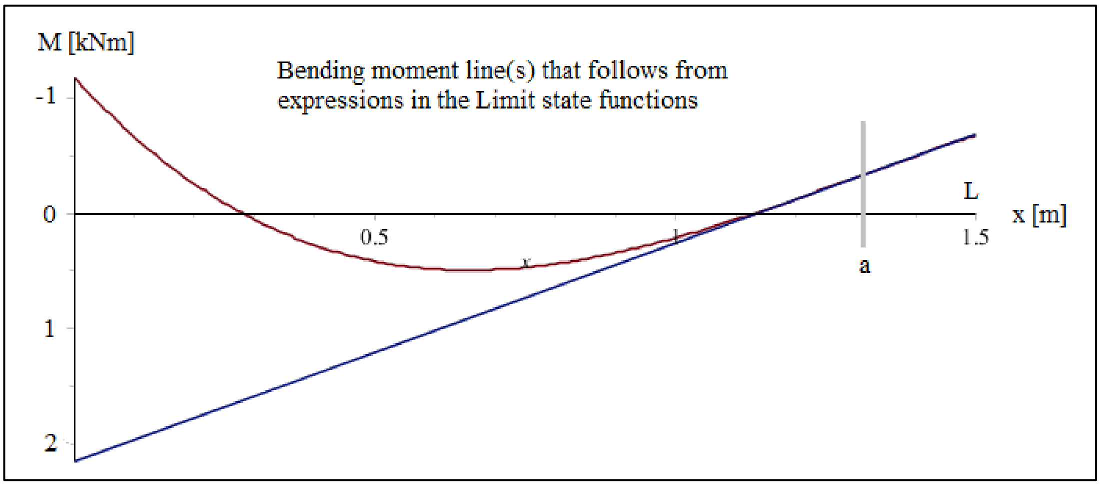


Figure 113: Bending moment diagrams from expressions (red line for x < a, blue line for x > a)

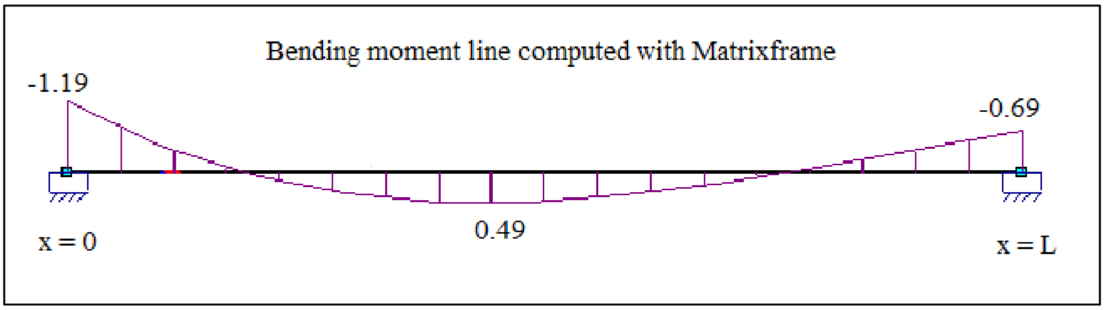


Figure 114: Bending moment diagram computed with Matrixframe

As can be seen, the bending moment diagram that was derived for the limit state functions correspond very well to the bending moment diagram as computed with Matrixframe. Additionally, the bending moments at the supports and in the field were computed with Maple. The results are shown in Table 68. Comparison with the values of the bending moments as shown in Figure 114 confirms that expressions for the bending moment diagram were derived correctly.

Location	x	Bending moment [kNm]
Submerged support	0	-1.188
Field	0.66	0.492
Dry support	L	-0.694

Table 68: Bending moments at supports and field computed with expressions in the limit state functions

The limit state functions are composed of more than only expressions for the bending moment diagram. For instance, the stresses at the glass surface are also computed. However, it is not possible to verify if these expressions are correct as software to compare the results is not available. By validating expressions for the bending moments, the most important parts of the limit state functions were validated.