GEOWALL®

as a

Quay wall

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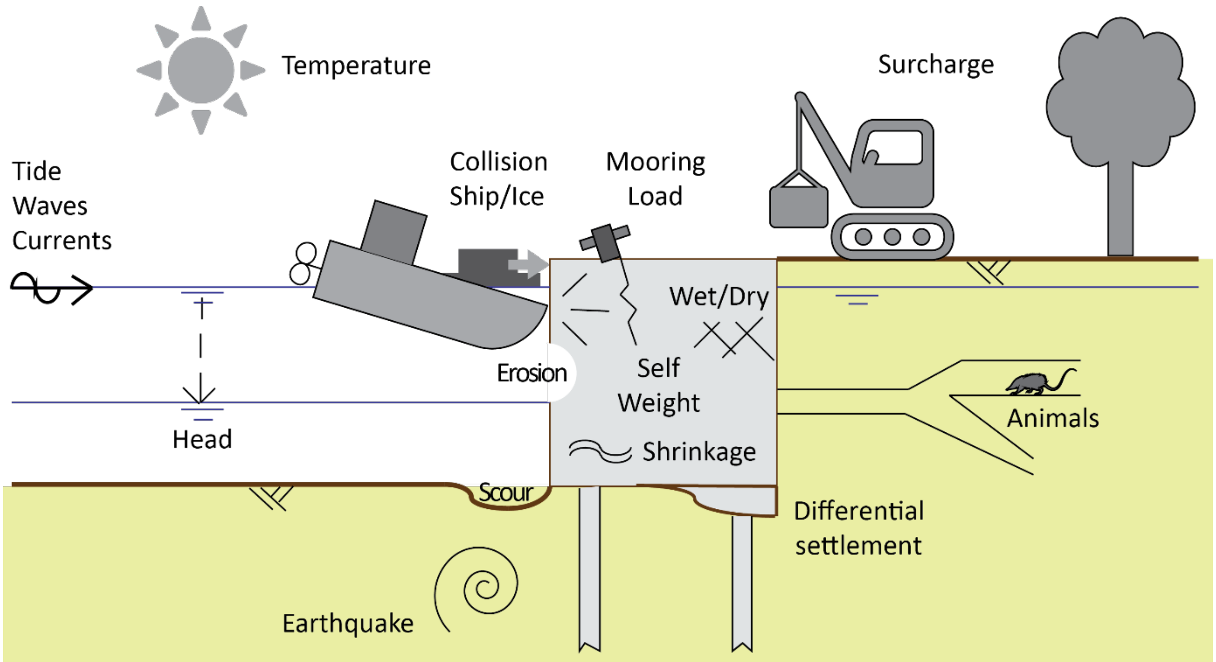
NETICS
THE INNOVATION ENGINEERS



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ABSTRACT

Bank protections are currently constructed with stone, wood, concrete and steel. NETICS has introduced a sustainable and commercially very attractive alternative called the GEOWALL. The goal of this study is to find a viable field of application within the quay structures by considering the arising challenges of a larger scale wall. Additionally several potential design improvements are described to cope with the challenges. The GEOWALL could fail due to sliding, overturning, exceeding internal stresses, large deformations, vertical instability, overall instability and piping. The safety of the structure will be assessed with predefined general factors of safety and permissible internal stresses, these are the test values. The safety for sliding, overturning, large deformations, vertical stability and piping is calculated with an analytical method. The safety for overall stability, internal strength and large deformations is calculated with a finite element method. There are two main variables in the analyses: the retaining height and the type of soil. The retaining height varies between one and five metres. The soils are simplified to sand, clay and peat. From the results of the analytical analysis and the finite element analysis it can be concluded that sand-GEOWALLs on sand between one and three metres and clay-GEOWALLs on clay between one and three metres are the most viable scenarios for a larger scale GEOWALL with current design. For GEOWALL quay walls larger than three metres and peat-GEOWALLs on peat suitable design improvements are required to meet the stringent safety requirements. The design improvements should be able to limit the possibility of failure due to sliding, overturning and failure trough large deformations. There are three categories of potential improvements: material improvements, geometrical design improvements and structural design improvements. The embedded wall is expected to be the most promising geometrical design improvement. Adding a drainage system is expected to be the most promising structural design improvement at this moment. It is recommended to study the effects and efficiency of the proposed design improvements in future research. It is also recommended for NETICS to continue their research on the strength and resistance of different GEOWALL types and on the influence of every possible load. In case of a GEOWALL design for a specific location it is recommended to determine the safety factors for that specific location and to check the analytical results with more advanced modelling programmes. Lastly it is recommended to consider not only the technical aspects, but also the practical and commercial aspects if one is investigating the design improvements.

PREFACE

In May 2015 I started with this master thesis for the faculty of Hydraulic Engineering at the Technical University of Delft. I worked at this project for nine months, it was both a challenging and fun period where many lessons were learned. Currently I am proud to present the final version of my thesis.

Many people assisted me along the way in the process of my thesis. First of all, I would like to thank the graduation committee: Ir. Henk Jan Verhagen, Dr. ir. Jarit de Gijt, Dr. ir. Leon van Paassen, Ir. Hugo Ekkelenkamp and Ing. Edert Besseling. Each of the committee members have been supportive and helpful by giving me constructive feedback, valuable information and interesting views on the research project. Special thanks to Hugo en Eldert for providing the opportunity to do this research project at NETICS and for the weekly meetings to discuss the progress of the research.

In addition I would like to thank Wilfred Molenaar for all his help with the structural calculations and Ronald Brinkgreve for all the help with the finite element modelling program Plaxis2D.

Finally, I would like to thank my friends and family. They reviewed parts of my report and supported me throughout this research.

Delft,

January 2016

SUMMARY

Bank protections are currently constructed with stone, wood, concrete and steel. NETICS has introduced a sustainable and commercially very attractive alternative called the GEOWALL. A GEOWALL with a retaining height of 0.72 metres has been constructed in their first pilot project. The goal of this study is to find a viable field of application within the quay structures by considering the arising challenges of a larger scale wall. The field of application determines the boundary of the retaining height. For this study the most viable application is found by comparing the GEOWALL with conventional quay wall structures. Based on this comparison the GEOWALL is identified as a potential alternative for a wooden sheet pile wall up to three metres and a stone gravity wall up to five metres. The GEOWALL is initially applied as a squared gravity type structure in small waterways and ponds within the field of quay walls.

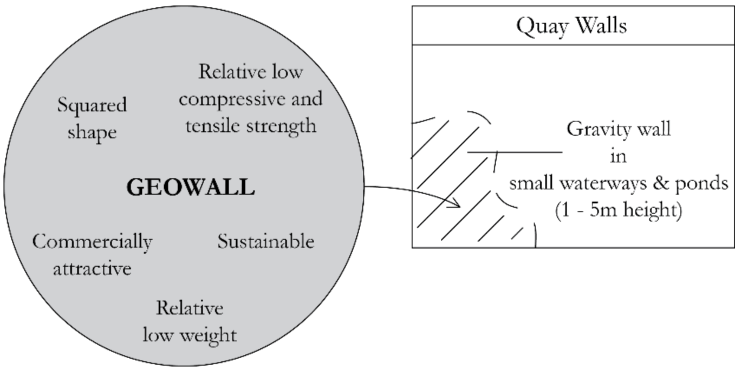


Figure 0-1: GEOWALL field of application

The GEOWALL could fail due to sliding, overturning, exceeding internal stresses, large deformations, vertical instability, overall instability and piping. The safety of the structure will be assessed with a deterministic level I approach. This means predefined minimum factors of safety as the margin between the loads and resistances. The safety factors for sliding, overturning, large deformations, vertical stability and piping are calculated with an analytical method. The safety factors for overall stability and the maximum and minimum stresses for structural failure and large deformations are calculated with a finite element method.

There are two main variables in the analyses: the retaining height and the type of soil. The retaining height varies between one and five metres. For every retaining height there is a width, a water level, a surcharge and a water level difference (head):

Retaining height (H) [m]	Width (B) [m]	Water level (h) [m]	Surcharge (q_v) [kN/m^2]	Head [m]
1.0	1.0	0.8	5	0.2
2.0	2.0	1.6	5	0.4
3.0	3.0	2.4	10	0.6
4.0	4.0	3.2	20	0.8
5.0	5.0	4.0	30	1.0

The soils are simplified to three general types of soil: sand, clay and peat. For every soil type there is a set of soil properties and a set of GEOWALL characteristics:

Identification	Model	Type	γ_{sat} $[kN/m^3]$	γ_{dry} $[kN/m^3]$	E' $[kN/m^2]$	v'	c' $[kN/m^2]$	ϕ' [°]
Sand	Mohr-Coulomb	Drained	20	17	40,000	0.3	0	35
Clay	Mohr-Coulomb	Drained	14	14	5,000	0.3	8	22
Peat	Mohr-Coulomb	Drained	11	11	1,000	0.3	5	15
Sand-GEOWALL	Mohr-Coulomb	Drained	22	20	700,000	0.4	2500	0
Clay-GEOWALL	Mohr-Coulomb	Drained	19	16	700,000	0.4	2500	0
Peat-GEOWALL	Mohr-Coulomb	Drained	16	14	700,000	0.4	2500	0

A scenario is defined by a combination of a retaining height and a type of soil. Every scenario is analysed and returns a set of safety factors and internal stresses. These safety factors are compared with the general factors of safety. These general factors of safety are: sliding – 1.5, overturning – 1.5, vertical stability – 2.5, overall stability – 1.5 and piping – 1.5. The internal stresses are compared with the permissible stresses. The maximum tensile strength is 0.07 MPa and the minimum compressive strength is 3.0 MPa. These values are based on averaged values of compressed stabilised earth blocks. There are three possible outcomes:

1. The calculated values are larger than the test values for the Ultimate Limit State.
2. The calculated values are less than the test values for the Ultimate Limit State and larger than the test values for the Serviceability Limit State.
3. The calculated values are less than the test values for the Serviceability Limit State.

From safety analysis I it can be concluded that the considered conceptual design of the GEOWALL is potentially safe on sand for retaining heights between one and five metres and on clay for retaining heights between one and three metres. The current design of the GEOWALL is unsafe for peaty soils. A design improvement is required for these unsafe scenarios. This design improvement should be able to limit the possibility of failure due to sliding, overturning and large deformations.

From safety analysis II it can be concluded that the considered conceptual design of the GEOWALL is safe on sand and clay for retaining heights between one and three metres in the cross-section. The current design is unsafe for structures larger than three metres. A design improvement is required for these scenarios. This design improvement should be able to limit the possibility of failure due to sliding. The current GEOWALL design is safe for a varying subsoil of clay and sand in the longitudinal direction.

Both analysis combined it can be concluded that sand-GEOWALLs on sand between one and three metres and clay-GEOWALLs on clay between one and three metres are the most viable scenarios for a larger scale GEOWALL. For GEOWALL quay walls larger than three metres and peat-GEOWALLs on peat suitable design improvements are required.

There are three categories of potential improvements: material improvements, geometrical design improvements and structural design improvements. Material improvements are improvements of the construction material. The material can be improved in terms of weight and strength. Both depend on the soil type and many other (local) parameters such as the moisture level and the construction method of the GEOWALL elements. The geometrical design improvements do not require any additional construction materials, such as wood, stone or bricks; they are variations on the original geometry. The four geometrical design improvements are: a wider wall, an embedded wall, a triangular wall and a stepped wall. From these four improvements the embedded wall is expected to be most promising at this moment. Structural design improvements are design improvements which require additional materials. The four structural design improvements are: strengthening of the wall, adding a drainage system, placing foundation and putting armour at the front or on the top of the wall. From these four improvements the drainage system is expected to be most promising at this moment.

The recommendations for further research are brought back to five themes: the simplifications, the material of the GEOWALL, the safety, the way of modelling and the practical & commercial aspects. In case of the simplifications it is recommended to perform research more different types of soil, the difference between a monolithic wall and stacked elements, the optimal shape and size of the elements and the influence of every possible load. Considering the GEOWALL material it is recommended for NETICS to continue their research on the strength and resistance of different GEOWALL types. The study showed that the GEOWALL is most viable for smaller retaining heights (< 3 metres). It is therefore recommended to work with lower Ultimate Limit State test values in case of smaller retaining GEOWALLs. This will result in more safe scenarios. In case of modelling the structure it is recommended to check the analytical results for specific designs with more advanced programs. Lastly it is recommended to consider not only the technical, but also the practical and commercial aspects in the choice of a design improvements.

CONTENTS

LIST OF FIGURES

LIST OF SYMBOLS

Symbol	Definition	Unit
A	Cross-sectional area	[m²]
α	Inclination	[°]
b	Width of the structure	[m]
β	Backfill slope angle	[°]
B	Width of the structure	[m]
c'	Effective cohesion	[kN/m²]
C_p	Lane's constant	[-]
C_p	Primary compression coefficient	[-]
C_s	Secondary compression coefficient	[-]
δ	Wall friction	[°]
e	Eccentricity	[mm]
E'	Effective Young's modulus	[kN/m²]
f	Friction coefficient	[-]
f_s	Local side friction	[MN/m²]
g	Gravitational acceleration	[m/s²]
h	Retaining height / layer thickness / water level	[m] / [m]
H	Horizontal force / retaining height	[kN]
γ	Weight density	[kN/m³]
γ_{sat}	Saturated weight density	[kN/m³]
γ_{dry}	Dry weight density	[kN/m³]
I	Moment of inertia	[m⁴]
K	Soil pressure coefficient	[-]
L	Length of the structure / Total seepage distance	[m]
M	Moment	[kNm]
N_x	Coefficients	[-]
p_b	Maximum bearing capacity	[kN/m²]
φ'	Effective angle of friction	[°]
q	Surcharge	[kN/m²]
q_c	Cone resistance	[MN/m²]
ρ	Mass density	[kg/m³]
σ'	Effective stress	[kN/m²]
$\sigma_{compressive}$	Compressive stress	[kN/m²]
$\sigma_{tensile}$	Tensile stress	[kN/m²]
$\sigma_{k,max}$	Maximum acting stress on the subsoil	[kN/m²]
$\sigma_{k,min}$	Minimum acting stress on the subsoil	[kN/m²]
$\sigma'_{v,i}$	Initial vertical effective stress	[kN/m²]
$\Delta\sigma'_v$	Increase of the vertical effective stress	[kN/m²]
t	time	[s]
u	Displacement	[mm]
V	Vertical Force	[kN]
v'	Effective void ratio	[-]
W	Section modulus	[m³]

Chapter 1

INTRODUCTION

The motivation behind this research is the patented GEOWALL, a retaining wall made of local sediment/silt. The GEOWALL is able to replace standard bank protections and is a low-cost sustainable solution as it works with local sediment as construction material. The initial GEOWALL construction has been developed in a small scale set-up. The objective of this thesis is to investigate and evaluate the challenges of a larger (1.0 – 5.0m) GEOWALL structure.

1.1. THESIS MOTIVATION

Bank protections are currently constructed with stone, wood, concrete and steel. NETICS has introduced an alternative concept called the GEOWALL. The GEOWALL is an in 2014 patented concept by NETICS and is described as 'a retaining wall of compressed sediment/silt'.

The maturity of a new concept is specified with the 'Technology Readiness Level'. There are nine levels, starting at level one knowing the basic principles of the application to level nine which represents an application that is already proven through successful mission operations (US, 2011). The maturity of the GEOWALL is currently estimated by the board of NETICS between level four and level five: Strength tests have been carried out and the first pilot projects have been constructed.

The first pilot is built in Flood Proof Holland, a testing and demonstration site near the Technical University of Delft for innovative temporary embankments. The wall in this test area is made of compressed stabilised earth blocks of 30 x 14 x 8 cm. These blocks are stacked on top of each other on a wooden foundation, resulting in wall which is 2.5 metres long, 0.3 metres deep and 0.72 metres high (see Figure 1-1). The challenge is to construct larger bank protections with this new concept, so retaining walls of compressed soil with a larger height than one metre. To make such large bank protections possible, NETICS has developed a new press which can produce infinite long blocks with a cross-section of 30x20 cm.

Figure 1-1: GEOWALL pilot in Flood Proof Holland

Compared to conventional bank protections the GEOWALL concept holds several advantages which make this concept attractive for further research. The wall is environmental friendly, since locally available soil of the banks is re-used in the construction. This approach prevents unnecessary disposal of sediment/silt as well as unnecessary supply of construction materials to the construction site, resulting in a minimal ecological footprint. The re-use of (free) available soil also supports the commercial attractiveness of the structure. NETICS investigated that approximately 250 million euros is issued by the municipalities in the Netherlands every year to replace wooden vertical bank structures (beschoeiing) while budgets of municipalities become smaller. The amount of hard wood needed for these yearly replacements is equivalent to a forest as large as the surface of the city Delft. A potential technical advantage is the relative small additional settlement of the light structure due to the re-use of the soil. Sediment from the banks is returned in a compacted state to approximately the same location, which is expected to result in limited extra settlements. In conclusion an increasing support from society to invest in sustainable products, high prices for conventional bank structures in combination with tighter budgets of municipalities and the potential minimal settlement, makes it attractive to investigate this concept further.

1.2. THE RESEARCH DESIGN

A research design supports the research project. This research design not only limits the project to a manageable size, it also provides a perfect guidance throughout the project. According to Verschuren, Doorewaard, and Mellion (2010) the research design holds a structured overview of the expected achievements within the research project.

The research design holds three elements.

1. The research objective
2. The research framework
3. The research questions

RESEARCH OBJECTIVE

The research objective clarifies the exact contribution of this research project to the overall project context. The project context is the general idea at the start of the thesis, which is defined in this thesis as 'A larger GEOWALL'. It holds many different challenges and potential solutions which are all very interesting to investigate. Three of these challenges and potential solutions are mentioned in particular. The first, the owners of NETICS are interested in the technical, commercial and practical feasibility of the GEOWALL. They want to have a design for the structure and knowledge about the construction technique. They expect that tensile forces are governing in higher retaining walls and propose to look into a reinforced wall. The second challenge is the relative settlement of the GEOWALL. It is expected that a small difference in weight between the removed soil of the bank and the placed blocks of the compressed soil will lead to a negligible settlement. A foundation would in such case not be necessary and would make the technology more commercially attractive. A third one suggested to design an embedded and stepped wall. The background behind this concept is the relative low weight of the wall which is expected to results in horizontal displacement (sliding) of the structure. It can be concluded that the project holds a wide variety of views on the subject. The research objective will therefore clarify the contribution of this research project to the general context: 'A larger GEOWALL'.

The research objective has been found with the step-by-step approach (Verschuren et al., 2010) which is further elaborated in Appendix A. This approach starts by identifying the type of research: theory-oriented research or practice-oriented research. This thesis has been identified as a practice-oriented research project. A practice-oriented research considers the so-called 'intervention cycle', which is a predefined set of steps to solve a practical problem:

1. Challenge
2. Diagnosis
3. Design
4. Change
5. Evaluation

A research project can contribute to each of these five steps. It is very tempting to approach this thesis as a design-oriented research and to focus directly on an optimal GEOWALL design. However a design-oriented research can only be carried out if the previous two steps, the challenge and the diagnosis are clearly defined. As mentioned earlier it is still quite unclear what the actual challenges are for a larger GEOWALL. These challenges of a larger GEOWALL should first be determined (step 1). Identifying the challenges is followed by a diagnosis (step 2). The diagnosis focusses on the background and the causes of the challenges. It also includes a course of action that needs to be taken in order to find potential design solutions (step 3). The objective of this thesis covers the first three steps of the intervention cycle: Defining the challenges, performing a diagnosis and considering potential design solutions. The two remaining steps of the intervention cycle are not covered due to time limitations, however they could be a focus for another thesis.

The first objective is to determine the most viable field of application of the GEOWALL within the field of quay walls by assessing the characteristics of the GEOWALL and the field of quay walls. The second objective is to investigate the arising challenges of a larger scale GEOWALL on different soil types by investigating the safety of the wall for seven different failure mechanisms.

RESEARCH FRAMEWORK

A research framework is a schematic representation of the research objectives. The research framework can be used in various ways. First, it is used to get from the objectives to a set of relevant research questions. Second, the research framework provides comprehensive information on the focus of the research project. Third, the framework is used as blueprint for the structure of the report. The latter is explained next.

The research framework is obtained by following the step-by-step approach (Verschuren et al., 2010). A detailed elaboration of the steps are presented in Appendix A. The essence of the framework is based on the confrontations between a research object and a research perspective. The research object is the phenomenon under study and the research perspective is the way that the object will be studied. One research object and one research perspective are defined per research objective. Applying the above approach leads to the following structure of this thesis:

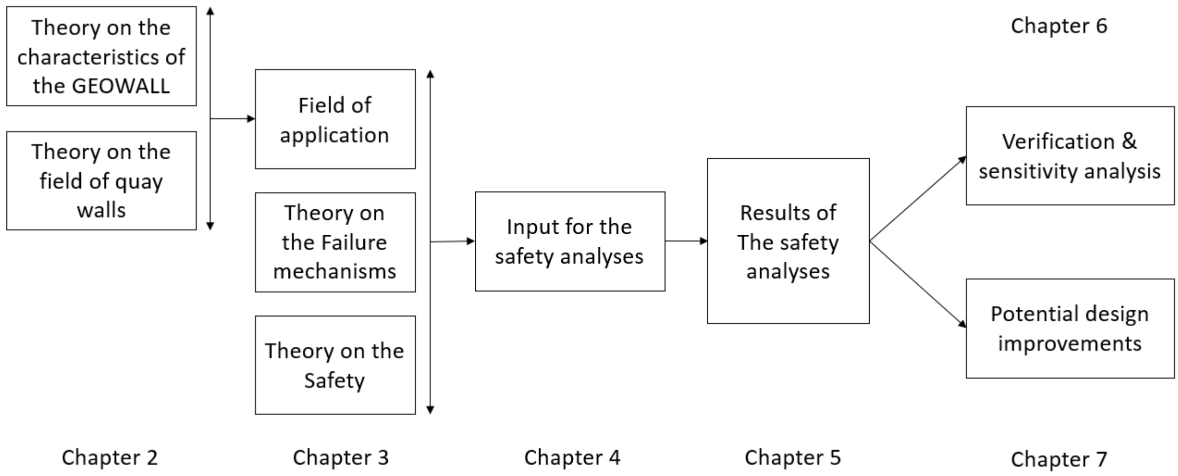


Figure 1-2: Research framework

Chapter 2 holds the required theory on the characteristics of the GEOWALL and the field of quay walls. The theory is used to find a viable field of application for the GEOWALL. Additional theory on failure mechanisms and safety is given in chapter 3. In chapter 4 this theory is quantified to function as input for the safety analyses. The results of the safety analyses are presented in chapter 5. From these results it can be determined which scenarios are critical, e.g. the scenarios that lead to failure of the GEOWALL. Chapter 6 provides a verification and a sensitivity analysis of the two safety analyses. Chapter 7 investigates several potential design improvements. Chapter 8 is the conclusion of the thesis and provides a set of recommendation for further research.

RESEARCH QUESTIONS

Research questions are used as a tool to reach the main objective of the thesis. The questions have been answered during the research period. Each answers contributes by throwing light on parts of the subject. The total of all answers can be used to answer the main question of the thesis:

> What is the most viable field of application of the current GEOWALL within the field of quay walls and what are the arising challenges of a larger GEOWALL on different soil types for seven different failure mechanisms?

The previous paragraph schematised the research project with the help of a research framework. For every confrontation in this framework there is a research question. This method results in the following research questions for this thesis:

1. What is the most viable field of application for the current GEOWALL within the field of quay walls?
    a. What is the identity of the GEOWALL in terms of appearance, weight, strength, type of material and type of structure?
    b. How does the field of quay walls look like and where to place the GEOWALL compared to conventional quay walls?
2. How to assess the safety of the GEOWALL for combinations of a varying retaining height and different soil types (scenarios)?
    a. What are the failure mechanisms of the GEOWALL leading to a collapse of the structure and how can these failure mechanisms be calculated?
    b. Which calculation program(s) are required to calculate the failure mechanisms and how are they used to obtain a set of safety factors per scenario?
3. Which input is required to investigate the safety of the wall for different scenarios and what are their values?
    a. How is the input quantified and how are all input values linked to the varying retaining height of the structure and the varying soil types?
    b. What are the minimum required factors of safety per failure mechanism?
4. What are the critical scenarios of a larger scale GEOWALL?
    a. What are the results from the safety analyses and how are these results interpreted?
    b. How are the results from the safety analysis used to say something about the critical scenarios of a larger scale GEOWALL?
5. What are potential design adjustments and which of these adjustments are most promising to cope with the critical scenarios?
    a. How could the current design be adjusted and what are the consequences of such an adjustment in respect to the failure mechanisms?
    b. What is the most promising design adjustment to cope with the critical scenarios?

1.3. SCOPE

The research on a larger GEOWALL is brought to a manageable size by means of a clear scope. This scope is formed by a set of predefined limitations or boundaries. Most of these limitations originate from engineering experience, obtained in meetings with supervisors and researcher.

Safety. It is advised to use the deterministic design approach (level 0) instead of a semi-probabilistic design approach (level I). The argument is that a partial factor is a result of many tests and known uncertainties. At this moment the GEOWALL cannot have a partial factor over the strength as there is not enough data available from field tests.

Time dependency. Excess pore pressures due to rapid settlement just after construction is taken out of the scope. Only the settlement after approximately 30 years (10,000 days) in the longitudinal direction is taken into account. This value of 10,000 days is a commonly used value. It represent the settlement after infinite time and it is an easy value to calculate with (In the formula of Koppejan: $\log(\Delta t) = \log(10{,}000) = 4.0$).

Construction material. The characteristics of a GEOWALL element depend on many aspects. From these aspects only a variation in the type of soil is taken into account in this thesis.

Construction method. There is yet no definitive construction method specified. This thesis will focus on what eventually is being constructed not how it is constructed. Therefore only the moment before and after construction are taken into consideration.

Feasibility. There is the commercial, practical and technical feasibility. The commercial feasibility and practical feasibility are not considered in the objective of the thesis and are therefore out of the scope. They can be taken into account for argumentation, however the focus of this research is on the technical feasibility of the GEOWALL for the use of a quay wall.

Loads. A larger GEOWALL is defined as a GEOWALL with a varying height between one and five metres. The width of the structure and loads onto the structure are determined per height.

Chapter 2

FIELD OF APPLICATION

The central question of this chapter is: Where is the field of application of a GEOWALL used as quay wall? This question is answered with information on the material properties, the type of structure and the lay-out of the GEOWALL. The construction material is compressed stabilised locally available soil. This is comparable with compressed stabilised earth blocks used in masonry structures. The conceptual design for this study is chosen to have a rectangular shape. The GEOWALL will be considered as a gravity type structure which is most likely to be applied in small waterways and ponds. Especially in situations with limited heights and limited forces, the GEOWALL seems to be a viable alternative for existing quay structures.

2.1. INTRODUCTION

The title 'GEOWALL as a Quay wall' holds the two main aspect in this research, namely the GEOWALL and the Quay wall. The initial goal of this chapter is to provide more information on upscaling the GEOWALL, the kind of material and the type of structure. The GEOWALL is defined as a wall made of elements of compressed stabilised locally available earth. In contrary to the quay wall little is known about upscaling GEOWALLs. The shape, height and type of material of such a quay structure are not yet strictly defined. The construction method is also under development and the potential construction costs are in many cases still assumptions.

The secondary goal is to identify different categories in the field of quay walls and to provide the relevant information on these categories. The definition of a quay wall is according to SBRCURnet (2014) an earth-retaining structure for the berthing of ships and the transhipment of goods. This study defines the quay wall as a vertical structure between land and water. Quay walls can be constructed in all kind of forms, heights and materials. They are constructed in different ways and have varying construction costs. Over the years many different types of quay walls have been constructed, therefore there is much information available.

The third and final goal in this chapter is to conclude with a viable field of application of the GEOWALL within the field of quay walls. The title 'GEOWALL as a quay wall' also insinuates that the GEOWALL is seen as an application within the field of quay walls. Based on the identification of the GEOWALL in combination with a categorised field of quay walls something can be said about this field of application.

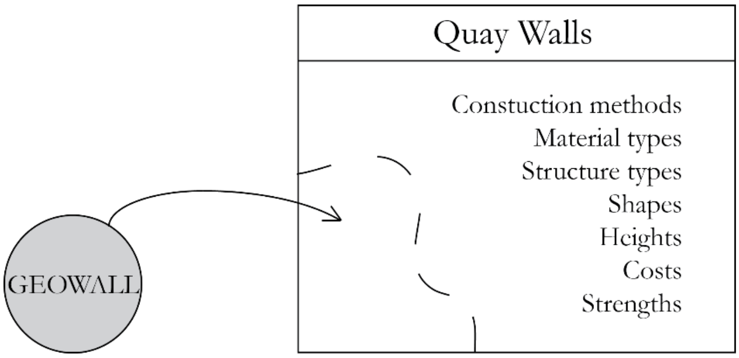


Figure 2-1: GEOWALL field of application

2.2. GEOWALL MATERIAL

The main ingredient of the GEOWALL is removed soil from the bank where the wall is going to be constructed. This soil can vary per location. There is a wide range of different soil types, ranging from 100% sand to 100% clay to 100% silt. Within this range all sorts of combinations are found: silty clay, clayey sand, loamy sand, etc. These different soil types are shown in a soil texture triangle (see Figure 2-2). A soil texture triangle is a classification tool to visualise the classes for soils (Shirazi & Boersma, 1984), each class with its relative portion of sand, clay and silt.

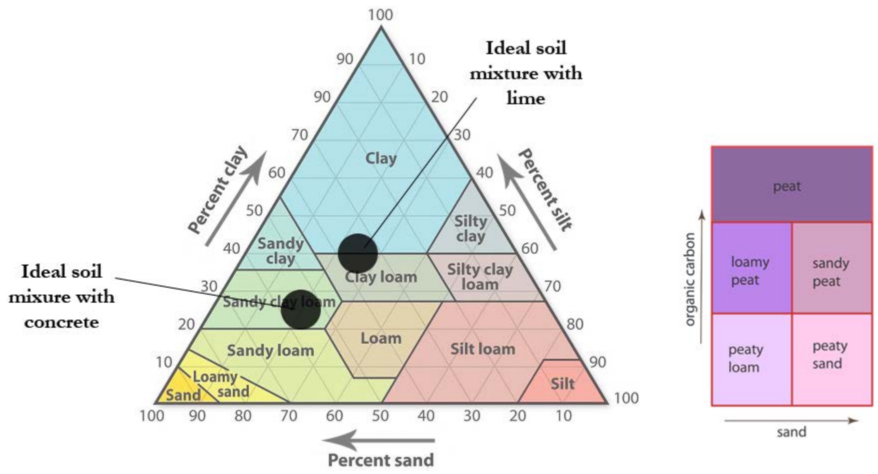


Figure 2-2: Soil texture triangle in combination with the location of two ideal soil mixtures [Based on] (Shirazi & Boersma, 1984) & (Cranfield, 2015)

The GEOWALL structure is obtained by compacting this removed soil from the banks into elements. Compacting the soil is similar to the production technique of compressed stabilised earth bricks. Pulverized moist earth is poured into a press and then compressed with either a manual press or a motorized press. The mixture is stabilized with additives. As a rule of thumb, cement is generally used in sandy soils and hydraulic lime is predominantly used in clay soils. It has to be noted that compacting different types of soil will result in different types of GEOWALLs.

According to Adam and Agib (2001) not only the type of soil and the type of stabiliser influence the characteristics of a compressed element. Four other aspects are identified[2]. First of all there is the water content of the soil. Second aspect is the quantity of stabiliser added to the soil. Third aspect is the magnitude and duration of the applied pressure and fourth aspect is the curing period for strengthening after production. Varying these four aspects will result in elements with different weights and different strengths. Unfortunately, there is little information available from literature about the weight and strength of different compressed soil elements. Some information on this subject is found in the literature on compressed stabilised earth bricks for masonry structures. This information is summarised in Appendix B.

[2] NETICS uses also other parameters for their recipe model. This information remains confidential.

One of the researches is performed by Maini (2010). He conducted several tests to find the optimal mixture for an earthen masonry block. He varied in two of the previously described variables: the type of soil and the type of stabiliser. The type of soil is a mixture of gravel, sand, clay and silt. The types of stabiliser are cement and lime. De tests of Maini showed that a mixture of 15% gravel + 50% sand + 15% silt + 20% clay + cement and a mixture of 15% gravel + 30% sand + 20% silt + 35% clay + lime results in the strongest blocks. These ideal mixtures are also visualised in the soil texture triangle (see Figure 2-2).

The characteristics of these 'ideal' blocks are given in the following table.

Identification	γ $[kN/m^3]$	E' $[MN/m^2]$	v'	$\sigma_{compressive,dry}$ $[MN/m^2]$ After 28d	$\sigma_{compressive,wet}$ $[MN/m^2]$ After 28d	$\sigma_{tensile}$ $[MN/m^2]$ After 28d
Ideal CSEB	17 - 20	700 - 1000	0.35 – 0.5	5 - 7	3 - 4	0.5 - 1

2.3. GEOWALL STRUCTURE

The GEOWALL as defined in this study is a vertical bank structure, or in simple words 'a vertical wall between water and land'. To determine the identity of the GEOWALL structure a parallel is drawn with conventional vertical bank structures. First, the construction material of the GEOWALL is compared with the construction material of conventional bank structures. Second, background information is given on quay wall construction types and third the construction costs are given of conventional construction types.

CONSTRUCTION MATERIAL

The construction materials which are currently used in vertical bank structures are concrete, fired brick, wood and steel. The specific weight and strength of the GEOWALL are compared with the weight and strength of conventional construction materials, see the table below. CSEB has the second lowest specific weight and the lowest compressive and tensile strength. There is one construction material which has an overlapping strength with the CSEB, namely fired bricks. The lower boundary of fired bricks is comparable with the upper boundary of compressed stabilised earth bricks.

	CSEB	Wood	Brick	Concrete	Steel
Specific weight [kg/m3]	1800	400	2000	2200-2500	7750-8050
Compressive strength [Mpa]	5-7	30-50	7-105	12-105	500
Tensile strength [Mpa]	0.5 – 1.0	20-40	0.7-10.5	1.2 – 3.5	500

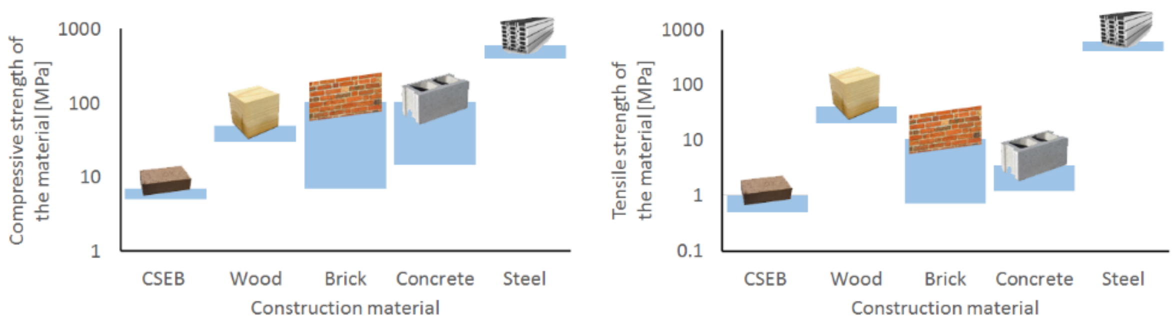


Figure 2-3: Compressive and tensile strength of different quay wall construction materials

CONSTRUCTION TYPE

Four types of quay walls are specified by de Gijt (2010): gravity walls, sheet pile walls, structures with relieving platforms and open berth quays. The latter two construction types are found in larger quay walls, such as the quay walls in the Port of Rotterdam. In these situations steel is predominantly used as construction material to cope with the large bending moments. The GEOWALL has a relative low tensile and compressive strength and is therefore without any improvements such as reinforcements not suited to compete with these kind of structures. The first two types, the sheet pile wall and the gravity wall, are more relevant to investigate.

Sheet pile walls are relative small structures which are partly embedded in the subsoil to obtain their stability. These structures can be constructed with wood, concrete or steel. Over the centuries wooden sheet piles have always been constructed. Currently they are predominantly used for small retaining heights, since larger retaining heights introduce too large bending forces. Concrete walls are seldom used, due to installation problems (de Gijt, 2010). Steel sheet piles are most preferred, since they can bear large forces and can easily be installed. For limited retaining heights (less than 3 metres) no anchor is required (de Gijt, 2010). For larger retaining heights an anchor is necessary to limit the deformations and to improve the stability of these systems. The current GEOWALL is not able to cope with large bending forces. At this moment the GEOWALL can be seen as a potential substitute for conventional sheet pile walls up to three metres. In particular for wooden sheet pile walls.

Gravity walls are relative wide structures between the water and the retained soil. These structures can be constructed with wood, concrete, steel or bricks (Appendix B). According to Haseltine (1991) brickwork is mainly used in shorter gravity retaining walls. Gravity walls are commonly built on soil types with a proper bearing capacity, for example on sand. All the forces and loads acting on a gravity wall, both horizontal and vertical, are transferred to this bearing stratum. The friction between the structure and the bearing stratum has to sustain the horizontal forces. The vertical forces and the weight of the structure itself are directly transferred to the subsoil. The horizontal forces on the structure also generate a moment which must be absorbed by the bearing stratum with only compressive stresses (de Gijt, 2010). At this moment the GEOWALL is a potential substitute for conventional smaller gravity type structures. In particular for brick gravity walls.

CONSTRUCTION COSTS

The costs of a sheet pile structure and a gravity type structure are compared with the estimated costs of a GEOWALL. The costs of the GEOWALL are roughly estimated by NETICS with 80, 100 and 120 euros per cubic metres of wall for a retaining height of respectively 1-3 metres, 4-7 metres and 8-10 metres. As can be seen in the graph a GEOWALL is according to this rough estimate commercially always more interesting than conventional quay wall structures. It should be noted that little can be concluded from this analysis, except the potential commercial advantages of the GEOWALL in comparison with conventional quay walls.

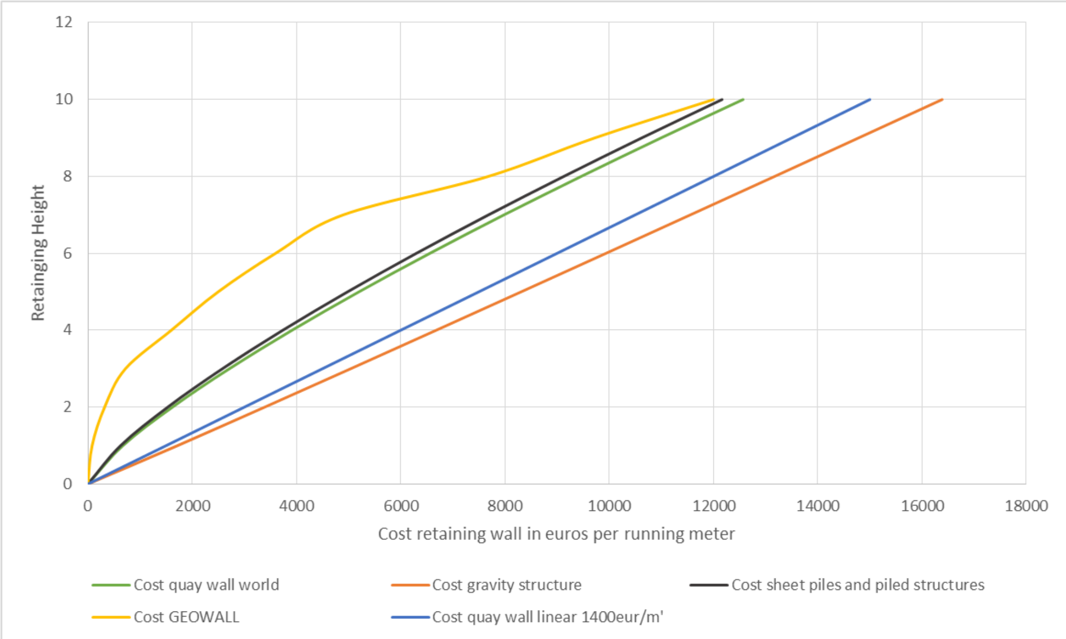


Figure 2-4: Costs of retaining walls in euros per running metre [based on] (de Gijt, 2011)

2.4. FIELD ANALYSIS

The goal of this paragraph is to provide an overview of the structures which are currently used as vertical bank protection for retained heights between one and five metres. Four general situations are identified:

Small waterways and ponds | Urban areas | Rivers and Lakes | Quay walls in ports

BANK STRUCTURES IN SMALL WATERWAYS AND PONDS

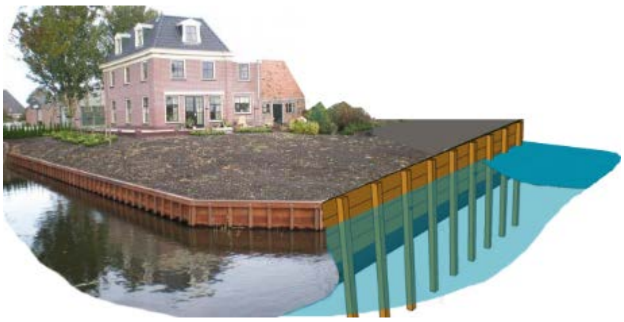


Figure 2-5: Wooden bank structure [based on] pictures of buitenwarenhuis.nl and kjtromp.nl

The bank structures in small waterways and ponds are often constructed with wooden, concrete or plastic sheet piles, due to the presents of soft soil. The sheet piles are constructed with wood, concrete or plastic and additional anchoring can be added for extra strength. Gravity type structures on the other hand are not ideal for soil with a low bearing capacity. A lack of proper foundation may lead to differential settlement which consequentially results in large tensile forces in the construction. The retained height of such a structure is in the order of zero to three metres. The surcharge is assumed to be 5 kPa (Bal & Van 't Wout, 2014). The GEOWALL would have a major advantage if it proves to be able to cope with this differential settlement and would therefore not need a foundation as gravity type wall.

BANK STRUCTURES IN URBAN AREAS

The stone vertical bank structures which are often found in urban areas date from the 13th and 14th century (de Gijt, 2013). In those days, wood and brick was predominantly used since one did not have the technique to build with concrete or steel. The load increased over time, due to traffic, storage and temporary supporting constructions. The small available areas in cities results mostly in solutions with sheet piles instead of gravity type structures. The retaining height varies between two and five metres and the most important function is bearing the load due to traffic, storage and temporary supporting constructions. In urban areas a uniform load of 30 kPa is taken into account (de Gijt, 2013).

Figure 2-6: Quay wall in an urban area

BANK STRUCTURES IN RIVERS AND LAKES

Bank structures in rivers and lakes are often more prone to currents and waves than in small waterways and in urban areas, due to water that is flowing towards an ocean, sea or lake and due to ships passing by. Thus, the main function is a protection against erosion. Bank structures in rivers and lakes are mainly protected by means of a revetment. Rigid structures such as sheet piles are usually considered when a steep slope is wanted due to limited available space. In that case, the structure has the characteristics of an earth retaining structure (Schiereck, 2012). Gravity wall type structures are also possible. A good example is a construction with gabions. These units can be piled up and are flexible and conform to changes in the ground surface due to settlement (Schiereck, 2012). The heights are between one and ten metres and the surcharge is estimated to be around 10 kPa.

QUAY WALLS IN PORTS

Gravity walls in ports often consist of prefabricated reinforced concrete elements (block wall, L-wall, caissons), while sheet pile walls, sheet pile with relieving platforms and pile supported platforms (jetty) are predominantly constructed with steel. The berthing of ships leads to retaining heights between five and thirty metres and introduce major loads, causing enormous tensile forces due to mooring and collisions. The surface load on the active side behind quay walls should be calculated with at least 30 kPa (de Gijt & Broeken, 2013).

SUMMARY FIELD ANALYSIS

The four general situations can be summarised in the following graph. It shows the relation between the application of a construction material and the retaining height of the quay wall. From a pragmatic point of view the CSEB construction material has been added into this graph. This has been carried out on an indicative basis as the field applications of such a wall as quay wall are lacking.

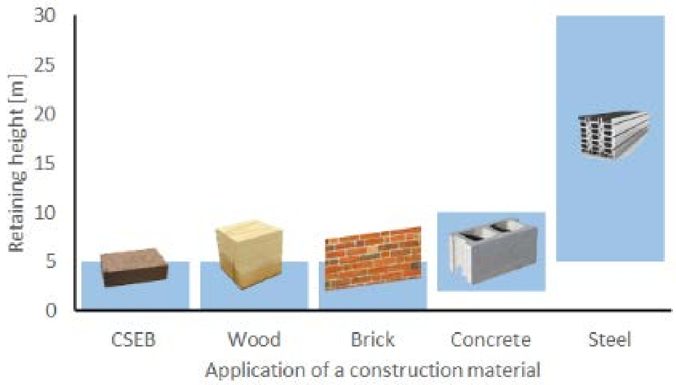


Figure 2-7: Application of different quay wall construction materials

2.5. CONCEPTUAL DESIGN

The previous paragraphs showed that the GEOWALL can be interpret as gravity type structure and that the material is comparable with brickwork based on weight and strength. The lay-out of 'gravity type structures' and 'brickwork structures' are therefore investigated. Vardon (2015) introduced a very general geometry for a gravity wall which can be seen in Figure 2-8a. A brickwork retaining wall is found in the brickwork design guide of Haseltine (1991) and is shown in Figure 2-8b. Taller gravity retaining walls are increasingly built as composite gravity walls, for example the gabion wall. Gabions are cages filled with earth or stones, which are used in building structures such as dams or dikes (Freeman, May 2000). The lay-out of Vardon and Haseltine are comparable to the geometry of the GEOWALL: A gravity wall which is placed onto the bottom of the lake/pond/river, between the water and the retained soil.

This design is the practical most simple and commercially most attractive solution. It is a practical simple solution, because it is a matter of stacking compressed elements on top of each other. And it is commercially attractive, since no extra resources are required besides the stabilised blocks. Besides these practical and commercial arguments, the conceptual design comprises another purpose, namely to analyse the failure of the structure for a varying height. The lay-out is therefore chosen as a squared massive block, which is a relative general solution. This concept design can be seen in Figure 2-8c. Note that this design can easily be changed if needed to fulfil the technical requirements. For example, adding a foundation or adding reinforcement to the structure.

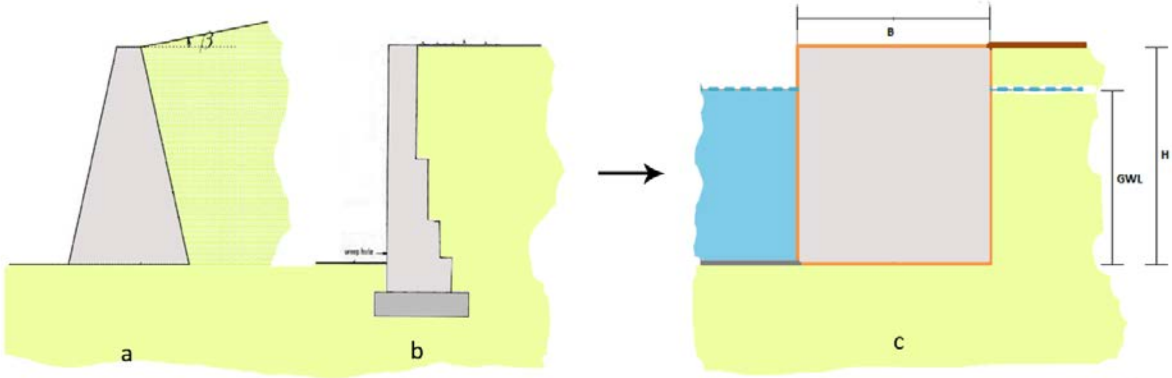


Figure 2-8: a. Gravity retaining wall, source: (Vardon, 2015), b. Brick retaining wall, source: (Haseltine, 1991), c. Concept design

2.6. CONCLUSION

Bank protections are currently constructed with stone, wood, concrete and steel. NETICS has introduced a sustainable and commercially very attractive alternative called the GEOWALL. A comparison is carried out between the GEOWALL and conventional quay wall structures to find a suitable field of application. Based on this comparison the GEOWALL is identified as a potential alternative for a wooden sheet pile wall up to three metres and a stone gravity walls at this moment up to five metres. The GEOWALL could be applied as a squared gravity type structure in small waterways and ponds within the field of quay walls.

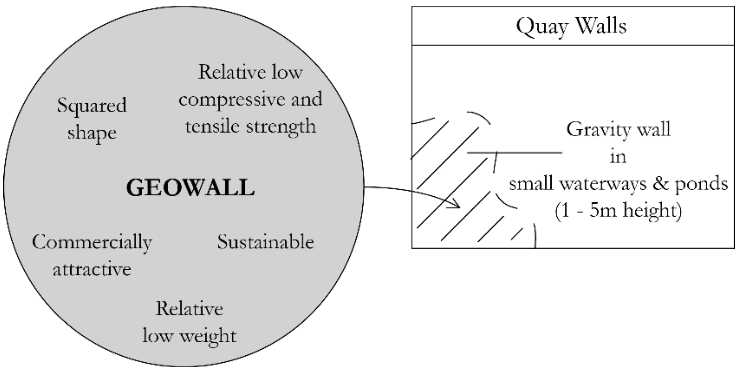


Figure 2-9: GEOWALL field of application

Chapter 3

METHODOLOGY

The goal of this chapter is an overview of the methods to assess the safety of a GEOWALL. It will look into the mechanisms by which a GEOWALL structure may fail to perform. Seven underlying failure mechanisms are investigated, but before doing so the concept of a failure mechanism is clarified. This chapter concludes by investigating the items related to the safety of the GEOWALL structure.

3.1. INTRODUCTION

The previous chapter concluded with the most viable field to apply the GEOWALL. This chapter will provide the theory on the failure mechanisms and the safety of the structure. There are several very general ways a quay wall can fail, also known as limit states. These limit states are investigated in the second paragraph along with the paths that lead to these states. The paths are known as the failure mechanisms and are identified by means of a fault tree. This is carried out in the third paragraph. The fourth paragraph provides theory on the probability of failure, also known as the safety of the structure. Two safety analyses are required to assess the failure mechanisms. These are elaborated in the fifth paragraph. The sixth paragraph is the conclusion.

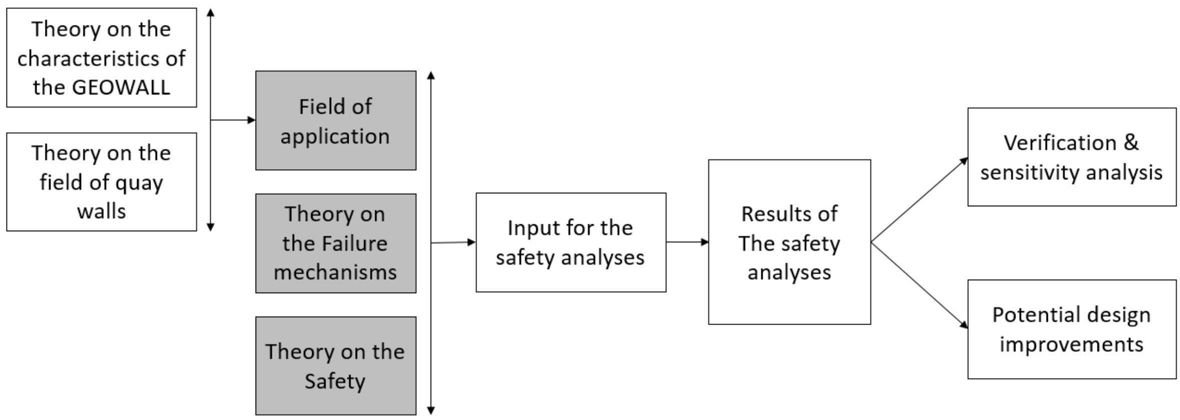


Figure 3-1: Chapter 3 in the research framework

3.2. LIMIT STATES

A structure or parts of a structure which are not able to fulfil their functions are known to be failing. The state on the border of failure and non-failure is called a limit state (CUR, 1997). Two types of boundaries are distinguished. There is the Serviceability limit state (SLS) and the Ultimate limit state (ULS). The serviceability limit state only indicates a disruption of normal use while the Ultimate limit state indicates a collapse of all or part of the structure (Vrijling et al., 2011). The latter, the ultimate limit state is separated in the five following failure modes:

EQU	Equilibrium	Loss of static equilibrium of the structure, considered as a rigid body
GEO	Geotechnical	Failure or excessive deformations of the subsoil
FAT	Fatigue	Fatigue failure of the structure
HYD	Hydraulic	Soil failure due to internal erosion (piping/hydraulic gradients)
UPL	Uplift	Uplift of the structure

The failure modes are reached by different failure mechanisms. Further elaboration on these failure mechanisms is carried out in the following paragraph.

3.3. FAILURE MECHANISMS

A failure mechanisms is a description of the way a structure fails. All failure mechanisms which can lead to collapse of a GEOWALL are organised in a fault tree (see Figure 3-2). The top of the fault tree shows ultimate failure, namely failure of the GEOWALL. The second level from the top holds the previously described failure modes. The layer beneath these failure modes holds the different failure mechanisms. The fault tree also includes several OR-gates. An OR-gate indicates that an event occurs if one or more actions prior to the OR-gate take place.

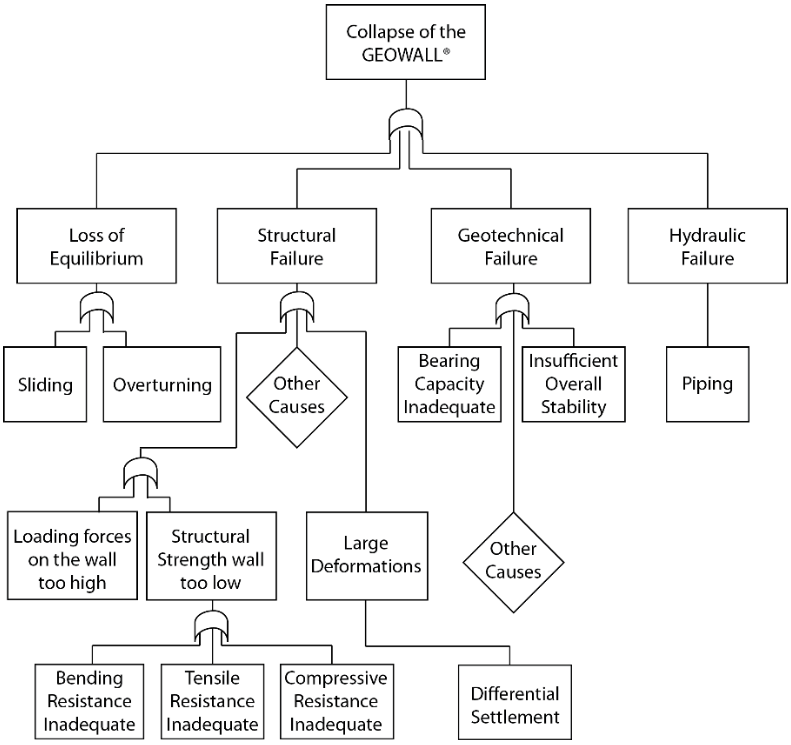


Figure 3-2: Fault tree GEOWALL

The GEOWALL is identified as gravity structure in chapter 2.3. The following list of failure mechanisms is found in the handbook Quay walls (de Gijt & Broeken, 2013) for a gravity wall:

1. Horizontal bearing capacity (sliding) [EQU]
2. Overturning [EQU]
3. Structural strength failure [STR]
4. Failure trough very large deformations [STR]
5. Vertical bearing capacity [GEO]
6. Overall stability [GEO]
7. Under and back seepage and piping [HYD]

The fault tree shows that all these seven failure mechanisms are also applicable on the GEOWALL. In the following the failure mechanisms are categorised per failure mode and discussed in more detail.

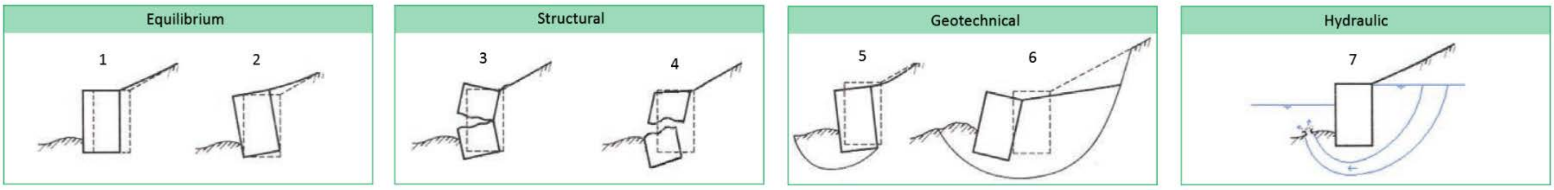


Figure 3-3: Failure mechanism of a gravity wall [based on] (de Gijt & Broeken, 2013)

EQUILIBRIUM

Equilibrium failure is the loss of static equilibrium of the structure. This static equilibrium is simplified to two stability functions: Horizontal stability and rotational stability:

- $\sum H = 0$
- $\sum M = 0$

The structure fails when the total of all forces acting on the structure do not meet these two requirements. Failure of the horizontal equilibrium is called sliding and failure of the rotational equilibrium is called overturning. The GEOWALL is assumed to be a rigid structure, so this can be interpret as the entire structure that moves over the soil in case of sliding and the entire structure tipping over in case of overturning.

Horizontal stability (sliding)

The total horizontal forces acting on the GEOWALL are transferred to the subsoil. The friction force of the subsoil should resist the resulting horizontal force. This friction force is determined by the total of the forces acting on the GEOWALL in the vertical direction, multiplied by a friction coefficient 'f':

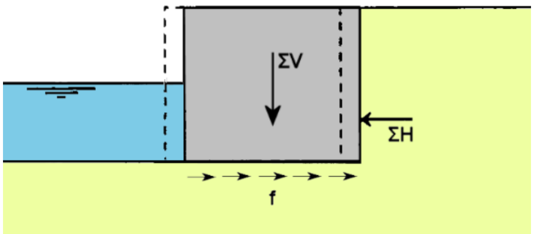


Figure 3-4: Horizontal stability [Based on] Manual Hydraulic structures (Vrijling et al., 2011)

$$\sum H < f \sum V$$

Where:

$\sum H$	[kN]	= Total of the acting horizontal forces
$\sum V$	[kN]	= Total of the acting vertical forces
f	[-]	= Friction coefficient

Rotational stability (overturning)

According to the Manual hydraulic structures (Vrijling et al., 2011), no tensile stresses are allowed, since tensile stress cannot be provided by the subsoil. It is therefore stated that soil stresses necessary for rotational stability may only be compressive. The resulting action force (R) should intersect the core of the GEOWALL. The core is defined as the middle one third of the width of the bottom (see Figure 3-5).

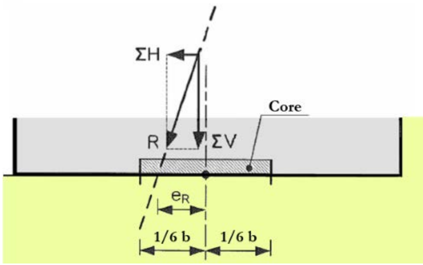


Figure 3-5: Rotational stability [Based] on Manual Hydraulic structures (Vrijling et al., 2011)

$$e_r = \frac{\sum M}{\sum V} \leq \frac{1}{6} \cdot b$$

Where:

e_r		= distance from the moment centre (K) to the intersection point of the resulting force and the bottom line of the structure
$\sum V$	[kN]	= Total of the acting vertical forces
$\sum M$	[kN]	= Total of the acting moments
b	[m]	= Width of the structure

It should be noted that a resultant force outside the core does not mean direct failure of the wall. It only results in a larger vertical stress, since only part of the soil below the structure will contribute to bearing (Vrijling et al., 2011). The maximum soil stress might in that case not be exceeded and the situation is not really problematic.

STRUCTURAL

Structural failure, also known as internal failure, occurs when stress in the structure exceed the maximum that the wall can bear. Excessive shear stresses, compressive stresses or tensile stresses can all lead to a structural failure. Structural failure is separated in two failure mechanisms: Structural strength wall and failure trough very large deformations.

Structural strength failure (cross section)

Structural strength failure is failure due to the exceedance of the permissible internal stresses. Lateral pressure is taken in by the structure causing internal stresses. These stresses should be less than the permissible stresses, otherwise cracks will develop. The internal stresses are not easily calculated analytically and require a more advanced calculation program.

Structural deformation failure (longitudinal direction)

Structural failure trough large deformations is a result of differential settlement in the longitudinal direction. After the construction of a GEOWALL the subsoil has to cope with the additional weight. This extra load induces shear stresses and normal stresses in the subsoil which re-orientates the grain structure. This is also known as compaction. (Molenaar & Houben, 2002). The amount of extra weight, determines the degree of compaction. In practice the type of subsoil varies over the length of the wall, which will result in a variation of the amount of settlement after construction. This variation in settlement is also known as differential settlement (see Figure 3-6). This differential settlement in the longitudinal direction is expected to result in increased tensile stresses in the upper part of the structure and increased compressive stresses in the lower part of the structure.

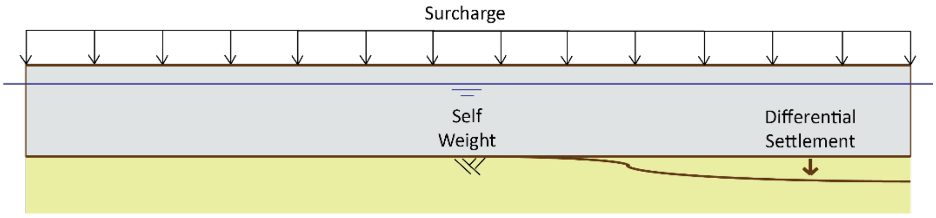


Figure 3-6: Differential settlement in longitudinal direction

The initial settlement occurs relatively fast after applying the load on the subsoil and is called consolidation or primary settlement. The slower and ongoing settlement under a constant load is called secondary settlement. This secondary settlement is also known as 'creep' or secular effect (Vrijling et al., 2011).

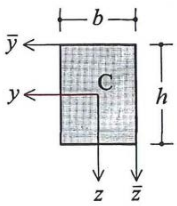
The compressibility is calculated with the formula of Koppejan (Koppejan, 1948). Koppejan combines the equation for primary compression from Terzaghi with the equation for creep from Buisman and is as follows:

$$\frac{\Delta h}{h} = \left(\frac{1}{C_p} + \frac{1}{C_s} \cdot \log(t)\right) \cdot \ln\left(\frac{\sigma'_{v,i} + \Delta\sigma'_v}{\sigma'_{v,i}}\right)$$

Where:

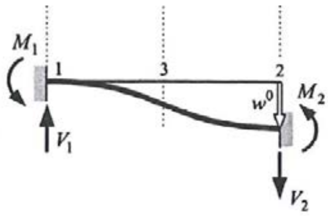
Δh	[m]	= compression
h	[m]	= layer thickness
C_p	[-]	= Primary compression coefficient
C_s	[-]	= Secondary compression coefficient
t	[s]	= Time
$\sigma'_{v,i}$	[kPa]	= Initial vertical effective stress
$\Delta\sigma'_v$	[kPa]	= Increase of the vertical effective stress

The translation from the variation in compression over the length to the tensile and compressive stresses inside the construction is carried out with the moment-displacement formula for a statically indeterminate beam. The displacement w_0 in this formula is the difference between the maximum occurring compression Δh_{max} and the minimum occurring compression Δh_{min}. The length l is the distance between this maximum and minimum. The GEOWALL is assumed as one rigid structure for the calculation of the second moment of area of the wall I_{zz} (see Figure 3-7). In combination with the Young's modulus (E) it is possible to calculate the internal moment in the structure (see Figure 3-8). The maximum tensile and compressive stresses in the wall are consequentially calculated by dividing the moment from the moment-displacement formula over the area of the wall.



$$I_{zz} = \frac{1}{12} bh^3$$

Figure 3-7: Second moment of area of a block (Hartsuijker & Welleman, 2007)



$$M_1 = M_2 = \frac{6EI}{l^2} w^0$$

Figure 3-8: Moment-displacement formula for a statically indeterminate beam (Hartsuijker & Welleman, 2007)

GEOTECHNICAL

Geotechnical failure is the failure of the subsoil beneath the wall (vertical stability) or failure over a failure line through the surrounding soil (overall stability). Failure of the subsoil beneath the wall is a result of large vertical forces pushing the structure into the soil. This is only expected to take place if the bearing capacity of the subsoil is not sufficient to withstand the downward forces of the structure. The equilibrium between the vertical downward force and the bearing capacity of the soil is denoted as the vertical stability:

- $\sum V = 0$

Overall stability is a result of extensive shear stresses in the soil. A slip circle occurs when the shear stresses in the soil exceed the maximum permissible stress in one line. The structure plus the soil is then expected to slide away over this line.

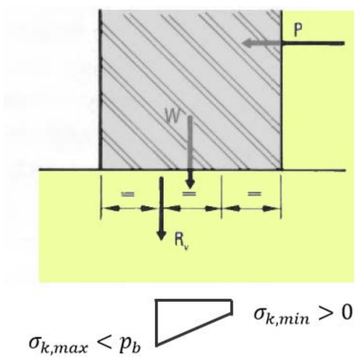
Vertical stability

The vertical forces are transferred to the subsoil. The vertical effective stresses in the subsoil are required to resist this pressure. The maximum effective stress a soil can bear is called the maximum bearing capacity p_b. This value differs greatly per soil type. The structure is likely to fail if the maximum acting stress on the subsoil $\sigma_{k,max}$ exceeds the maximum bearing capacity of the soil.

$$\sigma_{k,max} < p_b$$

Soil can also not cope with tensile forces, therefore:

$$\sigma_{k,min} > 0$$



$$\sigma_{k,max} = \frac{F}{A} + \frac{M}{W} = \frac{\sum V}{b \cdot l} + \frac{\sum M}{\frac{1}{6} \cdot l \cdot b^2}$$

$$\sigma_{k,min} = \frac{F}{A} - \frac{M}{W} = \frac{\sum V}{b \cdot l} - \frac{\sum M}{\frac{1}{6} \cdot l \cdot b^2}$$

Where:

$\sum V$	[N]	= Total of the acting vertical forces
A	[m2]	= Area of the bottom plate
W	[m3]	= Section modulus of the area of the bottom plate
b	[m]	= Width of the structural element
l	[m]	= Length of the structural element
$\sum M$	[kNm]	= Total of the acting moments

Figure 3-9: Vertical stability [based on] (Haseltine, 1991)

The bearing capacity p_b is calculated per soil type with the Brinch Hansen method (Verruijt, 2010):

$$p_b = c'N_c + q'N_q + 0{,}5\gamma' B \cdot N_\gamma$$

Where:

c'	[kPa]	= Effective cohesion
B	[m]	= Width of the structure
Nc/Nq/Ny		= Coefficients

Overall stability

The overall stability is the failure of the surrounding soil. This type of failure can be calculated with a limit state equilibrium model or a strength reduction model. There are multiple software programs available with one of these two calculation methods as background. A limit equilibrium method is based on Fellenius or Bishop. This method uses circular slip surfaces. By dividing the earth mass in several slices is it possible to calculate the driving and resisting moment in the soil. The equilibrium between the two moments determines the safety factor for overall stability. The advantage is the relative small amount of input parameters and the relative fast calculation time.

A finite element program is used to perform deformation and stability analysis. The stability analysis is carried out with a strength Reduction Method, also known as the c-phi reduction method. The shear strength parameters tan(φ) and c of the soil are successively reduced until failure occurs (PLAXIS, 2015).

HYDRAULIC

A water level difference between the water level in the waterway and the water level in the soil is able to cause piping. According to Vrijling et al. (2011) piping is a flow of water through a soil layer which has been caused by internal erosion. This phenomenon mainly occurs just beneath the bottom of the structure (see Figure 3-10). The formula of Lane is used to describe the critical situation in which piping occurs for water retaining structures (Lane, 1935):

Figure 3-10: Piping

$$L \geq \gamma \cdot C_L \cdot \Delta H$$

$$L = \sum L_{vertical} + \sum \frac{1}{3} L_{horizontal}$$

Where:

L	[m]	= Total seepage distance
C_L	[-]	= Lane's constant
ΔH	[m]	= Differential head across structure
γ	[-]	= Safety factor

3.4. SAFETY

The safety of the GEOWALL is the object under study if one analyses the challenges of a larger structure. The structure is safe when it is adequately protected against serious danger or hazard (Vrouwenvelder & Vrijling, 1995). This safety is quantified by looking at the reliability of the structure. The reliability according to the CUR (1997) is described as the margin between the resistance to failure and the loads. This margin can be determined with a deterministic approach (level 0) and a semi-probabilistic approach (level I). Both approaches are evaluated below. The most suitable is chosen for this study.

Deterministic design approach (level 0)

The margin between the total of the loads and the total of the resistances is taken into account by means of a general factor of safety. According to Vrijling et al. (2011) these factors are based on experience or engineering judgement. The estimation of these safety factors are not based on a quantification of the uncertainties.

Semi-probabilistic design approach (level I)

According to Vrijling et al. (2011) loads and resistances are variables treated as stochasts in the semi-probabilistic design approach. This means that the value of a load or resistance is assumed to have a certain distribution with a mean value μ and a standard deviation σ. The margin between load and resistance is taken into account by means of partial factors. The characteristic values, also known as the mean values, are multiplied with a partial factor. The partial factors are predefined for every type of load and resistance. The values for these factors are found in the Eurocode (EN, 2002) . The transformation results in the design values. A design resistance larger than the design load is considered to be a safe structure.

$$R_d \geq S_d \rightarrow \frac{R_{rep}}{\gamma_R} \geq \gamma_s \cdot S_{rep}$$

Both design approaches are evaluated. A deterministic design approach provides less accurate results as it is based on experience or engineering judgement. A semi-probabilistic approach is more accurate as it is based on known distributions for the loads and resistances. Nevertheless, strength distributions of the GEOWALL are not available. It is therefore concluded to assess the safety of the structure with the deterministic level I design approach.

3.5. SAFETY ANALYSES

The goal of a safety analysis is to assess the safety of the structure for different scenarios. A scenario is defined by a combination of a geometry, type of soil material and load. The combination of these three aspects is known as the 'input' of the analysis. This input is subsequently used to calculate the safety factors per failure mechanism. The result of the analysis is a set of safety factors per scenario. These safety factors can then be compared with the required general factors of safety to point out the critical scenarios.

The analysis is divided in two separate analyses. One analysis is calculated with an analytical method and one with a finite element method. The two methods are complementary: by combining both methods all failure mechanisms can be calculated. The two methods are further explained in the following.

The analytical calculation method can be described as ‘quick and dirty’: A fast and simple calculation method which provides rough outcomes. In theory the analytical calculation method is able to calculate all failure mechanisms. However, the accuracy is limited for failure mechanisms which have to deal with displacements and stresses inside the structure. The following four failure mechanisms are therefore ideal for this method: sliding, overturning, vertical bearing capacity and piping. Failure trough large deformations is also calculated in the analytical analysis, however is expected to provide less accurate results than the finite element method.

The failure mechanisms: structural failure, overall stability and failure trough very large deformations require an advanced method to obtain more accurate results. Two methods are evaluated: the limit equilibrium method and the finite element method. A limit equilibrium method (such as D-Geo stability) provides information about the overall stability of the structure. A finite element method (such as Plaxis2D) is able to calculate the overall stability of the structure and is additionally able to model the stresses inside the construction and the soil. A finite element program can also be used to investigate more detailed designs, thus making it possible to investigate the impact of different design improvements. A final advantage is the option of performing a sensitivity analysis. Based on previous arguments it is concluded to use a finite element method for the calculation of the remaining failure mechanisms. An overview for the two analyses and the corresponding mechanisms is given in the table below:

Safety Analysis I [Analytical method]	Safety Analysis II [Finite element method]
1. Sliding	1. Structural failure
2. Overturning	2. Overall stability
3. Failure trough very large deformations	3. Failure trough very large deformations
4. Vertical bearing capacity	
5. Piping	

Plaxis 2D is a two-dimensional finite element program used to perform deformation and stability analysis for geotechnical applications. There is a clear separation between the cross section calculations and the longitudinal calculations. Modelling the GEOWALL is therefore simplified to these two 2D model planes. Detailed information on Plaxis2D can be found in Appendix G.

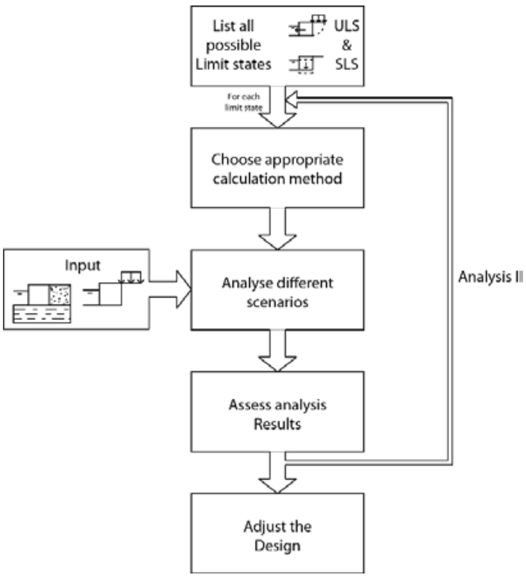


Figure 3-11: Analyses flow diagram

Figure 3-11 shows the flow diagram of the two analyses. The flow diagram starts with the seven failure mechanisms. For each failure mechanism the appropriate calculation model is chosen.

Note the loop in this flow diagram. Prior to the start of the second analysis the results of the first analysis are checked. These results are divided in safe, potentially safe and unsafe scenarios. The first two, safe and potentially safe scenarios, proceed to the second analysis. This second analysis starts at the stage ‘choose appropriate calculation model’. Instead of choosing the analytical method, the finite element method is chosen in this stage. Both analyses combined will result in an overview of safe, potentially safe and unsafe scenarios.

3.6. CONCLUSION

The GEOWALL can fail due to sliding, overturning, exceeding internal stresses, large deformations, vertical instability, overall instability and piping. The safety of the structure will be assessed with a deterministic level I approach. This approach uses predefined general factors of safety as margin between the loads and resistances. The safety for sliding, overturning, large deformations, vertical stability and piping are calculated with the analytical method. Structural failure, overall stability and large deformations are calculated with a finite element method.

Chapter 4

INPUT SAFETY ANALYSES

The goal of this chapter is an overview of the input for the two safety analyses. There are two main variables in these analyses: the retaining height and the type of soil. The retaining heights vary between one and five metres. The soils are simplified to three soil types: sand, clay and peat. All other input variables are either connected to the value for the retaining height or to the type of soil. The factors of safety are quantified for different failure mechanisms to set the target value for what is considered to be a safe structure.

4.1. INTRODUCTION

The previous chapter concludes with two analyses to assess the safety of the GEOWALL. This chapter quantifies the input for the two analyses. The input for analysis I is divided into four parts: the geometry, the material properties of the soil and the GEOWALL, the loads and the general factors of safety. The input for analysis II has the same four elements plus information on the modelled mesh of the geometry.

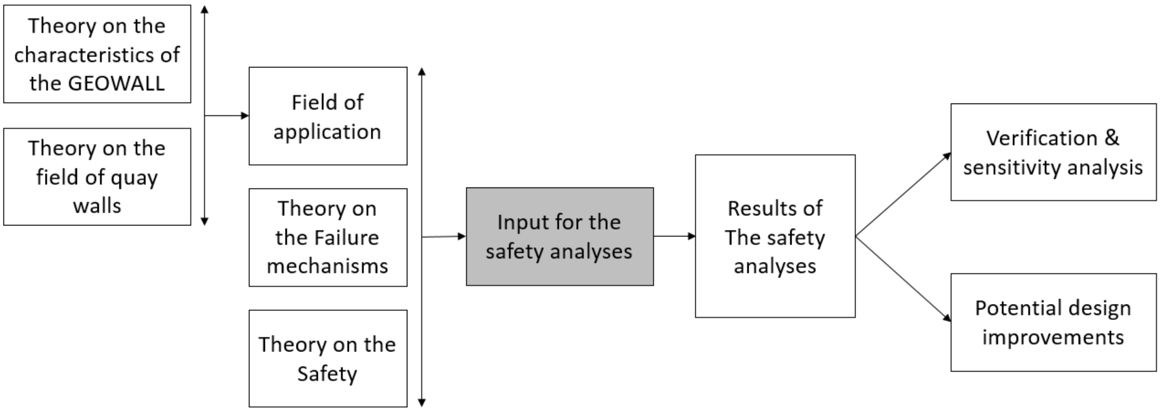
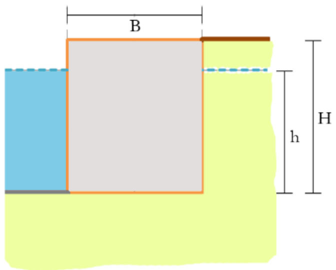


Figure 4-1: Chapter 4 in the research framework

4.2. INPUT ANALYSIS I

GEOMETRY

The geometry in the cross-section and longitudinal direction is as follows:



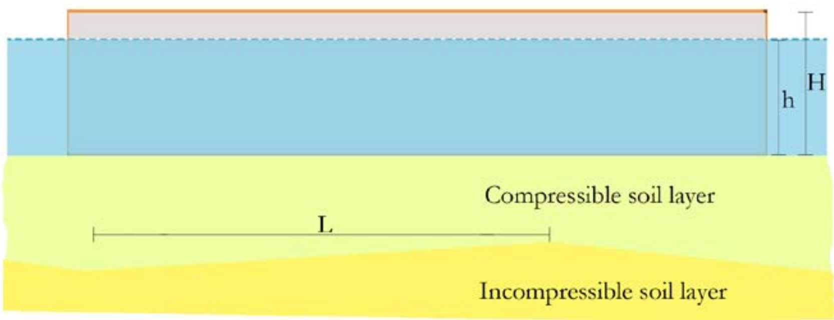


Figure 4-2: Geometry cross-section Figure 4-3: Geometry longitudinal direction

The geometry in the cross section is similar to the presented conceptual design. The geometry holds three basic geometrical variables, namely the retaining height (H), the water level (h) and the width (B). The retaining height is a varying value which is bounded by one and five metres. The interval in this range is one metre. The water level and width are predetermined varying parameters. This means that for every retaining height there is one predefined width and one predefined water level. The width of the structure is similar to the retaining height of the structure. This value for the width follows from the initial technical analysis which is described in Appendix C. The water level is 80% of the retaining height. In case of a head difference the water level in the waterway is lower than the ground water level.

Retaining height (H) [m]	Width (B) [m]	Water level (h) [m]
1.0	1.0	0.8
2.0	2.0	1.6
3.0	3.0	2.4
4.0	4.0	3.2
5.0	5.0	4.0

The geometry in the longitudinal direction also holds three basic geometrical variables, namely the retaining height (H), the water level (h) and the length (L). As explained in chapter 3.3 the geometry in the longitudinal direction is used to investigate the compressive and tensile stresses in the structure as a result of differential settlement. This differential settlement is taken into account by including a varying compressible soil thickness in the geometry. The length (L) is the length between the thickest part of the compressible soil layer and the thinnest part of the compressible soil layer (Figure 4-3).

MATERIAL PROPERTIES

This research simplifies all soil types into three main categories, namely sand, clay and peat. The motivation behind this simplification is that each soil category represents a specific part of soil. Sand represents the stronger soil layers. It has a high bearing capacity and water can flow easily through its structure. Clay is a representation for all high cohesive soils that also have some strength. Peat represents the soil with relative little strength. The three soil types are placed in the soil texture triangle of chapter 2.2 (see Figure 4-4). The soil type 'Silt' is not taken into account.

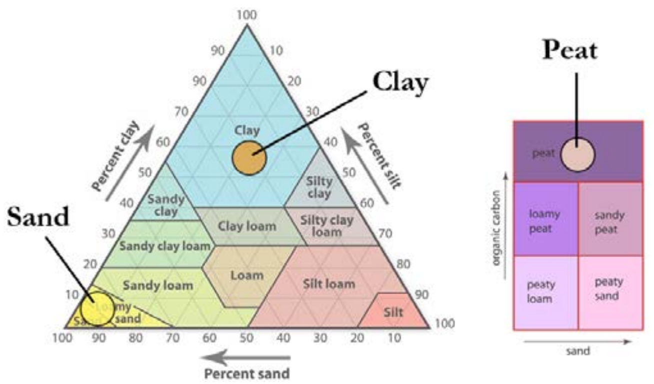


Figure 4-4: Sand, clay and peat in the soil texture triangle [Based on] (Shirazi & Boersma, 1984) & (Cranfield, 2015)

The handbook 'Constructing with soil' (CUR, 1993) and the website Geotechdata.info are both used to obtain the characteristics of the three soil types. The saturated weight γ_{sat}, the dry weight γ_{dry}, the effective cohesion c' and the effective friction angle ϕ' of the three soils are presented in the following table:

Identification soils	γ_{sat} $[kN/m^3]$	γ_{dry} $[kN/m^3]$	c' $[kN/m^2]$	ϕ' [°]
Sand	20	17	0	35
Clay	14	14	8	22
Peat	11	11	5	15

Each of these soils can be used for a GEOWALL element. As explained in chapter 2.2 the characteristics of the element depends on five aspects. From these five aspects only the type of the compressed soil is investigated in this study. The characteristics necessary for this analysis are the dry weight γ_{dry}, the saturated weight γ_{sat}, the effective Young's modulus E', the tensile strength $\sigma_{tensile}$ and the compressive strength $\sigma_{compressive}$. The dry and saturated weight of the GEOWALL are given by NETICS. The effective Young's modulus, tensile strength and compressive strength are taken from the research of Satprem Maini (Maini, 2010) on earthen masonry blocks. A range for these values is presented in chapter 2.2. The lower side of the range is taken as input for this analysis. Strength tests done by NETICS indicate that these values are representative for the GEOWALL elements. The characteristics of the GEOWALL elements are summarised in the table below:

Identification GEOWALLs	γ_{sat} $[kN/m^3]$	γ_{dry} $[kN/m^3]$	E' $[kN/m^2]$	$\sigma_{compressive,dry}$ $[MN/m^2]$ After 28d	$\sigma_{compressive,wet}$ $[MN/m^2]$ After 28d	$\sigma_{tensile}$ $[MN/m^2]$ After 28d
Sand-GEOWALL	22	20	700,000	5.0	3.0	0.5
Clay-GEOWALL	19	16	700,000	5.0	3.0	0.5
Peat-GEOWALL	16	14	700,000	5.0	3.0	0.5

The failure mechanism for horizontal equilibrium requires a value for the friction coefficient f. This coefficient is the smallest of the three following mechanisms according to Vrijling et al. (2011):

	Mechanism	Soil	ϕ	f
1	Friction GEOWALL – subsoil from Bal and Van 't Wout (2014)			0.58
2	Internal friction of the subsoil $f = \tan(\varphi)$	Soil	ϕ	$\tan(\varphi)$
		Sand	35	0.70
		Clay	22	0.40
		Peat	15	0.27
3	A deeper soil layer with a low sliding resistance			N/A

The failure mechanism for the soil bearing capacity is calculated with the Brinch Hansen method. This method requires the additional dimensionless coefficients Nc, Nq and Nγ. These coefficients are given in the soil mechanics book (Verruijt, 2010) for different values of effective soil angle ϕ'. The values for sand, clay and peat are presented in the following table:

Soil	ϕ' [°]	N_c	N_q	N_γ
Sand	35	46.124	33.296	45.228
Clay	22	16.833	7.821	5.512
Peat	15	10.977	3.941	1.576

The failure mechanism failure trough very large deformation is calculated with the settlement calculations of Koppejan. This method requires the addition of the coefficients Cp and Cs, which are respectively the primary and secondary compression coefficient. These coefficients are given in the CT3040 lecture notes (Molenaar & Houben, 2002) for different soil types. The values for sand, clay and peat are presented in the following table:

Identification	Cp	Cs
Sand	600	-
Clay	20	240
Peat	6.2	25

In addition to these coefficients the values for the layer thickness h, the time t, the initial vertical effective stress $\sigma'_{v,i}$ and the increase of the vertical effective stress $\Delta\sigma'_v$ are required. The values for the layer thickness vary over the length of the structure and are based on the soil analysis of the embankment near The Donk (appendix F). Note that this is a simplification by only considering the layer thickness variation at one location. At this location six soundings have been carried out. Each sounding holds geotechnical information to a depth of 20 metres. The interval between the soundings is approximately 50 metres and the overall length is 229 metres. The plane between the compressible and incompressible layer is presented in the graph below. The largest variation in the compressible layer is found over the length L (see Figure 4-5). The compressible layer thickness changes from 7,20 metre to 5,60 metre over a length of 22 metres.

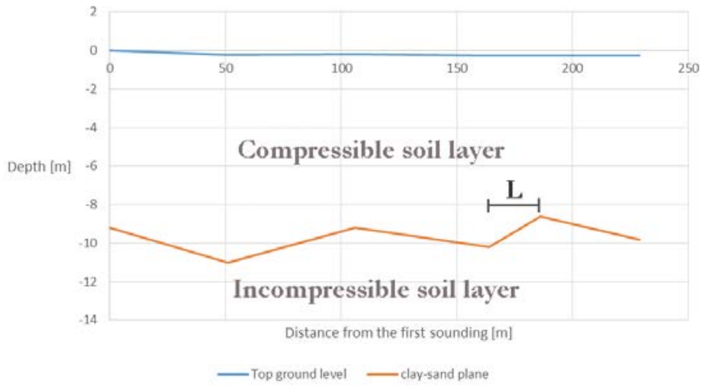


Figure 4-5: Geotechnical analysis of the soil near The Donk, The Netherlands

As stated in the scope, only long term effects are considered. Therefore a period of 10000 days is chosen, which is approximately 30 years. The initial vertical effective stress is calculated with the weight of the original bank. The increase of the vertical effective stress is the difference between the weights of the GEOWALL and the weight of the original bank.

LOADS

The GEOWALL is prone to many external influences. These influences are known as loads or actions. On the opposite side of the loads are the resistances. Resistances are mostly materials or structures with a certain strength. The GEOWALL has too many loads influencing the structure, however only part of them are taken into account in the defined field of application. Figure 4-6 provides an overview of all the loads acting on a GEOWALL. The table on the right of this figure shows a separation between the included loads and the excluded loads in respectively black and grey. The loads which are excluded are further explained.

The first excluded load is the mooring load. The mooring boulders are not placed in a constant sequence over the length of the GEOWALL. Usually there are some specific locations along a waterway or lake where ships can moor their ships. It is therefore important to include the mooring load only in the designs for these specific locations. This study considers a more generic situation, thus the structure should not be dimensioned with a constant mooring load acting over the full length of the wall. The collision of ships/ice, currents, waves and tides are only important in rivers and large lakes. In such waters, ships are of greater size and waves are able to develop due to a long fetch. Nevertheless, this structure will not be constructed in such areas at this moment and these loads are therefore not considered in this study. The influences of erosion, whetting and drying, shrinkage and temperature onto the wall are all related to the material characteristics of the wall and local situation. The influence of these loads is assumed not to be very dominant and is interesting to investigate in further research.

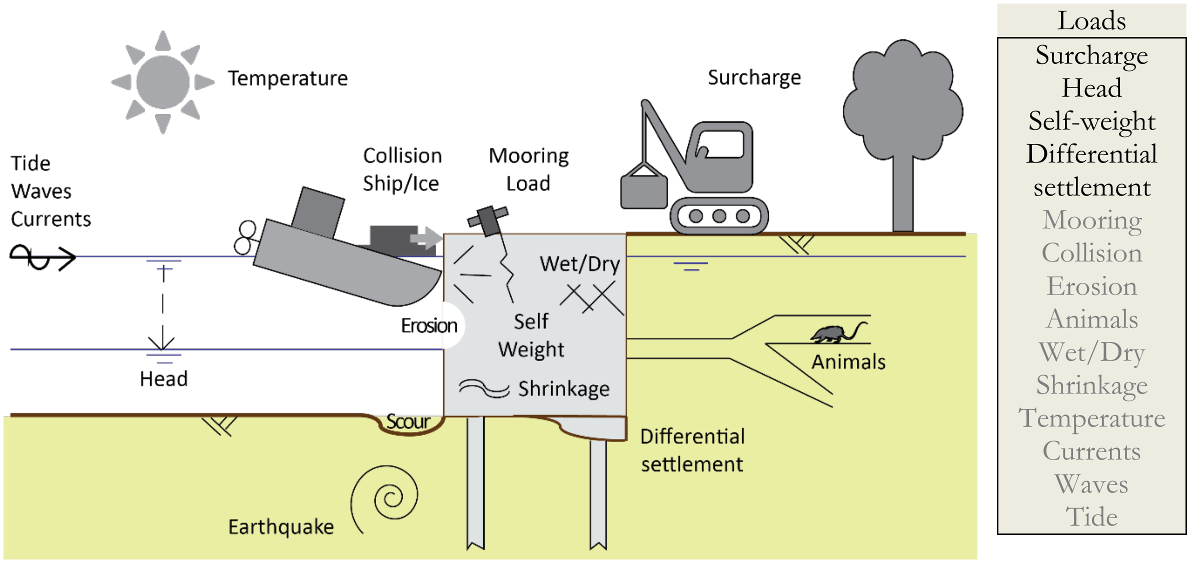


Figure 4-6: Loads on the GEOWALL

The following loads are considered in the cross-section and in the longitudinal direction:

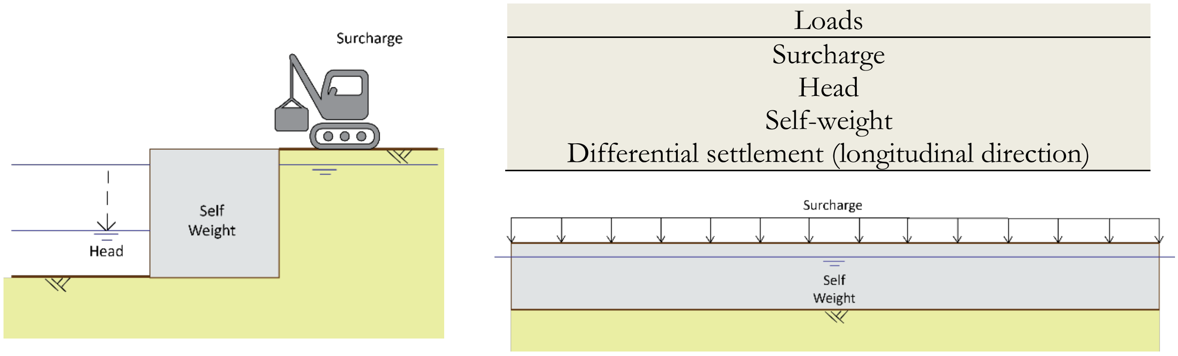


Figure 4-7: Loads on the GEOWALL in Cross-section an longitudinal direction

The first load is the surcharge. A surcharge per area of application is given in the field analysis of chapter 2.4. These are summarised in the following table:

Field	Retaining height (h) [m]	Surcharge (q) [kN/m^2]
Small waterways and ponds	0-3	5
Urban areas	1-5	30
Rivers and lakes	1-10	10
Quay walls in ports	5-30	>30

The CUR 166 (CUR, 2012) prescribes a representative surcharge of 5-10 kPa for bank protections.

The second load is the head difference. A head difference is defined as the water level difference between the water level in the waterway and the phreatic level in the retained soil. According to de Gijt (2013) a value of 0.5 metres is used as water level difference for the design of quay walls in urban areas. The third load results from the weight of the GEOWALL. The weights for the different types of GEOWALL are given in the previous section of this paragraph: Material properties.

The retaining height and soil types are the only variables in the analyses. The loads are linked to the variation in the retaining height. One surcharge and one head difference is determined per retaining height. The previous described sources for the surcharge and the head difference in combination with an expert opinion of one of the supervisors of this study supervisor results in the following list of loads per retaining height.

Retaining height [m]	Surcharge (q_v) [kN/m^2]	Head [m]
1	5	0.2
2	5	0.4
3	10	0.6
4	20	0.8
5	30	1.0

TEST VALUES

The analytical analysis performs several calculations. The result is a set of safety factors and a bending stress. The formulas for the calculation of the safety factors are given in the table below. These calculated values are compared with predefined test values. The test values are mainly general factors of safety. The factors of safety for the ultimate limit state and the serviceability limit state are specified for the required failure mechanisms. These values as well as their source are also given in the table below.

Failure mechanism	Formula	Factor of safety [ULS]	Factor of safety [SLS]	Source
Sliding	$SF, sliding = \frac{f * \sum W}{\sum H}$	1.5	1.0	Expert opinion + (Caltrans, 1990)
Overturning	$SF, overturning = \frac{\frac{1}{6} \cdot B}{e_r}$	1.5	1.0	Expert opinion + (Caltrans, 1990)
Vertical bearing capacity	$SF, bearing = \frac{p_b}{\sigma_{k,max}}$	2.5	1.0	Expert opinion + (Verruijt, 2010)
Piping	$SF, piping = \frac{C_L \cdot \Delta H}{L}$	1.5	1.0	(Vrijling et al., 2011)

The general factor of safety for vertical bearing capacity (only 2.5) does not account for settlement. Settlement is considered in a separate mechanism, namely failure trough large deformations. This failure mechanism is checked in both analyses. Instead of a safety factor, the analysis provides a rough estimate for the bending stress. The test value for this bending stress is chosen to be 0.5 MPa. This is similar to the tensile strength of a single compressed stabilised earth block.

4.3. INPUT ANALYSIS II

GEOMETRY

The analysis is carried out by a finite element model. The geometry in the cross and longitudinal direction for this finite element model are given below. The geometry in the cross-section is similar to the previous analysis. The geometry in the longitudinal direction is an 80 metre long wall, such that the largest variation in the compressible layer falls within the domain. Beneath this compressible clay layer is an incompressible soil layer. The results from the first analysis for the failure mechanisms 'failure trough very large deformations' turned out to be safe for sand, potentially safe for clay and unsafe for peat. This second analysis will therefore only consider a compressive subsoil of clay and an incompressive sandy soil. The thickness of the compressible clay layer varies over the length. This variation is similar to the previous analysis based on the geotechnical information of the soil near The Donk (Appendix F).

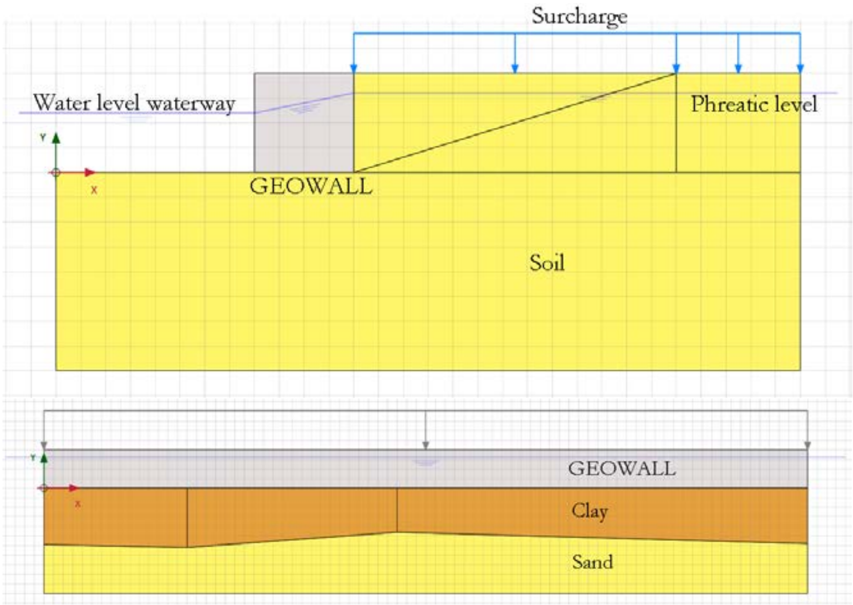


Figure 4-8: Modelling the GEOWALL in Plaxis2D

MESH

The analysis is carried out with a 'fine' mesh. The mesh is refined for parts of the area such as the GEOWALL and the soil surfaces adjacent to the wall.

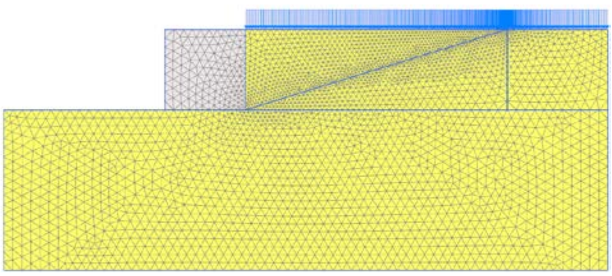


Figure 4-9: Mesh of the GEOWALL model

The soil layers are modelled with 15-node triangular elements. Each element has fifteen displacement points and twelve stress points (PLAXIS, 2015). The displacement points are used to describe the settlement of the structure. The stress points are used for the internal stresses and the overall stability calculations.

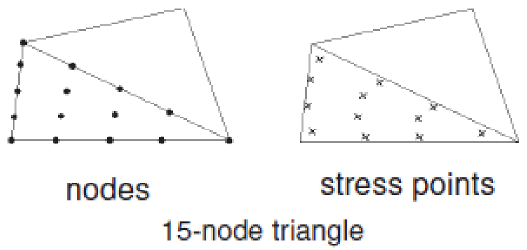


Figure 4-10: Position of nodes and stress points in elements (PLAXIS, 2015)

MATERIAL PROPERTIES

The material properties given for safety analysis I are equal to the material properties for this analysis. Once again there is the simplification to three soil types: sand, clay and peat. These soil types require some additional information for the finite element method, namely the way they are modelled, the effective Young's modulus E' and the effective Poisson's ratio v'. All soil types are modelled with Mohr-Coulomb. The Mohr-Coulomb model is a linear elastic perfectly plastic model. A constant averaged stiffness is estimated for the soil layers which results in relative fast computations. There are more advanced material models, for example the hardening soil model for more accurate deformation and stress patterns, however such models are not necessary for this case. The soils are set to 'drained', because it is chosen to analyse the long-term (drained) response. The effective Poisson's ratios and effective Young's moduli are also found in the handbook 'Constructing with soil' (CUR, 1993).

The total of all soil characteristics are summarised in the table below:

Identification	Model	Type	γ_{sat} $[kN/m^3]$	γ_{dry} $[kN/m^3]$	E' $[kN/m^2]$	v'	c' $[kN/m^2]$	ϕ' [°]
Sand	Mohr-Coulomb	Drained	20	17	40,000	0.3	0	35
Clay	Mohr-Coulomb	Drained	14	14	5,000	0.3	8	22
Peat	Mohr-Coulomb	Drained	11	11	1,000	0.3	5	15

The GEOWALL is modelled as soil. The compressive strength and tensile strength of the GEOWALL are given in the previous paragraph and are respectively 5.0 MPa and 0.5 MPa. The compressive and tensile strength can be expressed in the drained soil strength parameters: effective cohesion c' and effective friction angle ϕ'. A 1-axial compressive stress for the GEOWAL in combination with an angle of internal friction of 0 degrees results in an effective cohesion which is half the one-axial compressive strength:

$$compressive\ strength\ (kPa) = \frac{2c\cos\varphi}{1-\sin\varphi} \quad \rightarrow \quad c_{GEOWALL} = 0.5 * compressive\ strength$$

The soil void ratio and the permeability of the GEOWALL used for this study are found at earth-auroville.com which holds information on compressed stabilised earth blocks. The data is obtained for blocks which are stabilized with 5 to 10% cement and which are compressed with a pressure of 2-4 MPa. The soil void ratio is 0.4, this coincides with a porosity of 0.29. The permeability is 1.0E-8 m/s.

The GEOWALL properties are summarised in the table below:

Identification	Model	Type	γ_{sat} $[kN/m^3]$	γ_{dry} $[kN/m^3]$	E' $[kN/m^2]$	v'	c' $[kN/m^2]$	ϕ' [°]
Sand-GEOWALL	Mohr-Coulomb	Drained	22	20	700,000	0.4	2500	0
Clay-GEOWALL	Mohr-Coulomb	Drained	19	16	700,000	0.4	2500	0
Peat-GEOWALL	Mohr-Coulomb	Drained	16	14	700,000	0.4	2500	0

LOADS

The loads are similar to the loads of analysis I. For the five retaining heights the same surcharge and water level differences are taken into account.

TEST VALUES

The finite element method returns a safety factor for overall stability and a visualisation of the internal stresses. The safety factor for overall stability is calculated with the c/phi reduction method. The shear strength parameters of the soil (tan phi and c) as well as the tensile strength are successively reduced until a slip circle developed. The strength parameters are reduced by a selected amount of steps. $\sum Msf$ is a parameter in the finite element program which represents the safety of the structure. As the strength is reduced the value for this parameter increases. This is true until the structure collapses. From this point on the value of $\sum Msf$ remains constant. The factor of safety is similar to this constant value:

$$SF = \frac{available\ strength}{strength\ at\ failure} = value\ of\ \sum Msf\ at\ failure$$

The general factor of safety for overall stability in ultimate limit state is 1.5. The general factor of safety for overall stability in serviceability limit state is 1.0.

In addition to the overall stability the finite element program is able to visualise the variation in stresses over the longitudinal and cross-sectional plane of the structure. This includes the maximum (tensile) stresses and minimum (compressive) stresses. These are checked in the cross-section for structural failure and in the longitudinal direction for failure trough large deformations. The permissible stresses for a single compressed stabilised earth brick are known, however the tensile strength of a stacked GEOWALL structure has never been calculated in previous research. The brickwork retaining wall design guide (Haseltine, 1991) provides the permissible tensile stress for brickwork walls in longitudinal direction, which is 0.07 N/mm^2. The permissible compressive stress is similar to the maximum saturated compressive stress of a single compressed stabilised earth brick, which is 3.0 N/mm^2.

4.4. CONCLUSION

There are two main variables in the analyses: the retaining height and the type of soil. The retaining height varies between one and five metres. For every retaining height there is a width, water level, surcharge and water level difference:

Retaining height (H) [m]	Width (B) [m]	Water level (h) [m]	Surcharge (q_v) [kN/m^2]	Head [m]
1.0	1.0	0.8	5	0.2
2.0	2.0	1.6	5	0.4
3.0	3.0	2.4	10	0.6
4.0	4.0	3.2	20	0.8
5.0	5.0	4.0	30	1.0

The soils are simplified to three general types of soil: sand, clay and peat. For every soil type there is a set of soil properties and a set of GEOWALL characteristics:

Identification	Model	Type	γ_{sat} [kN/m^3]	γ_{dry} [kN/m^3]	E' [kN/m^2]	v'	c' [kN/m^2]	ϕ' [°]
Sand	Mohr-Coulomb	Drained	20	17	40,000	0.3	0	35
Clay	Mohr-Coulomb	Drained	14	14	5,000	0.3	8	22
Peat	Mohr-Coulomb	Drained	11	11	1,000	0.3	5	15
Sand-GEOWALL	Mohr-Coulomb	Drained	22	20	700,000	0.4	2500	0
Clay-GEOWALL	Mohr-Coulomb	Drained	19	16	700,000	0.4	2500	0
Peat-GEOWALL	Mohr-Coulomb	Drained	16	14	700,000	0.4	2500	0

A scenario is defined by a combination of a retaining height and a type of soil. Every scenario is analysed and returns a set of safety factors and internal stresses. These safety factors are compared with the general factors of safety. These general factors of safety are: sliding – 1.5, overturning – 1.5, vertical stability – 2.5, overall stability – 1.5 and piping – 1.5. The internal stresses are compared with the permissible stresses. The maximum tensile strength is 0.07 MPa and the minimum compressive strength is 3.0 MPa. These values are based on averaged values of compressed stabilised earth blocks.

Chapter 5

RESULTS

This chapter describes the output of the safety analyses. With this data the safe, potentially safe and unsafe situations can be clarified. For the unsafe structures the challenge is to find suitable design improvements. Based on the result, specific failure mechanisms can be pointed out which should be counteracted in the improvement of the design.

5.1. INTRODUCTION

The previous chapter concludes with a list of input variables for the two safety analyses. This chapter will go into the results of these two analyses. The chapter is divided into four paragraphs. The first paragraph is this introduction. The second paragraph describes the results of the safety analysis I. The third paragraph presents the results of safety analysis II. The fourth and last paragraph is the conclusion of the chapter.

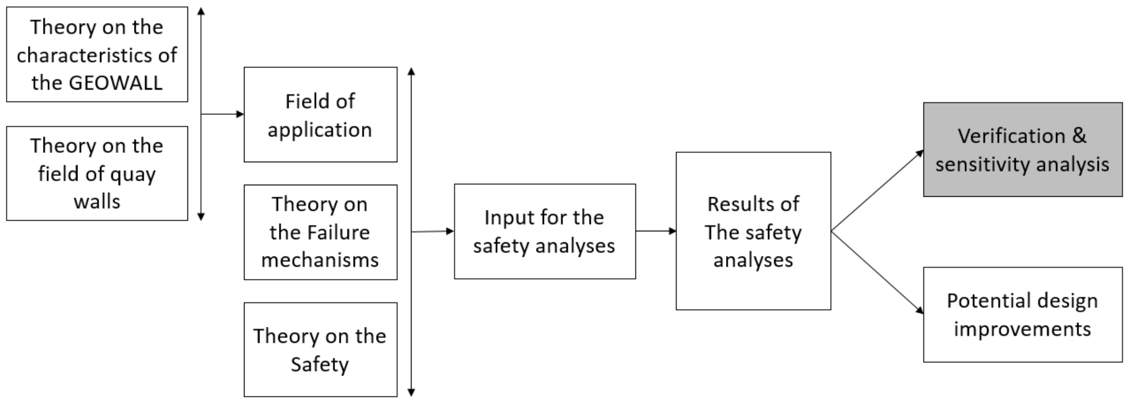


Figure 5-1: Chapter 5 in the research framework

The safety factors and the internal stresses from the analyses are compared with the test values: the general factors of safety and the permissible stresses. The general factors of safety and the permissible stresses are given in chapter 4.2 and 4.3. There are three possible outcomes:

4. The calculated values are larger than the test values for the Ultimate Limit State.
5. The calculated values are less than the test values for the Ultimate Limit State and larger than the test values for the Serviceability Limit State.
6. The calculated values are less than the test values for the Serviceability Limit State.

A value in the first category means that the current GEOWALL design is safe. A value in the second category means that the GEOWALL is potentially safe. Little cracks might appear, however there is no direct failure. A value in the third category means that the current GEOWALL design is unsafe. For these scenarios a design improvement is required. The three groups are summarised in the table below with the corresponding colours green, blue and red:

Larger than the test value [ULS]	Safe
Between 1.0 [SLS] and the test value [ULS]	Potentially safe
Less than 1.0 [SLS]	Unsafe → Design improvement required

5.2. RESULTS ANALYSIS I

The input for safety analysis I is provided in the previous chapter: five retaining heights and three types of soil. For every combination the analysis returns four safety factors and one bending stress. An overview of the results is given in the table on the following page. Every column represents a failure mechanism and every row represents a scenario (one soil type + one retaining height). The four safety factors are the values for sliding, overturning, vertical bearing capacity and piping. The bending stress is the output for large deformations. The values are compared with the predefined test values which are given in bold in the top of the table. The outcomes are presented with the tree colour as explained in the introduction.

Height	cross-section				Longitudinal direction
	Sliding	Overturning	Vertical bearing capacity	Piping	Bending Stress [N/mm^2]
	1.5	**1.5**	**3**	**1.5**	**0.5**
Sand					
1.0	**1.4**	**1.0**	**17.2**	**1.1**	**$0.34\cdot10^{-3}$**
2.0	**1.6**	**1.2**	**21.1**	**1.1**	**$1.36\cdot10^{-3}$**
3.0	**1.5**	**1.1**	**20.9**	**1.1**	**$3.06\cdot10^{-3}$**
4.0	**1.4**	**1.0**	**19.8**	**1.1**	**$5.44\cdot10^{-3}$**
5.0	**1.3**	**0.9**	**19.2**	**1.1**	**$8.51\cdot10^{-3}$**
Clay					
1.0	**1.0**	**0.9**	**8.1**	**1.1**	**0.04**
2.0	**1.6**	**1.5**	**6.6**	**1.1**	**0.17**
3.0	**1.2**	**1.2**	**4.9**	**1.1**	**0.38**
4.0	**0.9**	**0.9**	**3.7**	**1.1**	**0.68**
5.0	**0.8**	**0.7**	**3.2**	**1.1**	**1.06**
Peat					
1.0	**0.5**	**0.4**	**3.3**	**1.1**	**0.35**
2.0	**0.7**	**1.0**	**2.3**	**1.1**	**1.38**
3.0	**0.6**	**0.6**	**1.6**	**1.1**	**3.11**
4.0	**0.4**	**0.3**	**1.2**	**1.1**	**5.53**
5.0	**0.4**	**0.3**	**0.9**	**1.1**	**8.64**

Several conclusions can be drawn from these results for the considered conceptual design of the GEOWALL. First it can be concluded that this GEOWALL is potentially safe on sand with retaining heights between one and five metres, despite one safety factor beneath 1.0. Second it can be concluded that this GEOWALL is potentially safe on clay with retaining heights between one and three metres, despite one safety factor beneath 1.0. Third it can be concluded that the current design of the GEOWALL is unsafe for clay soils larger than three metres and peaty soils. A design improvement is required for these scenarios. This design improvement should be able to limit the possibility of failure due to sliding, overturning and failure trough large deformations. The objective of the research stated to find the most viable field of application for the given conceptual design. With this predefined tunnel vision, it is decided to exclude part of the scenarios. Only the safe and potentially safe scenarios proceed to safety analysis II. The selection is visualised in the following figure:

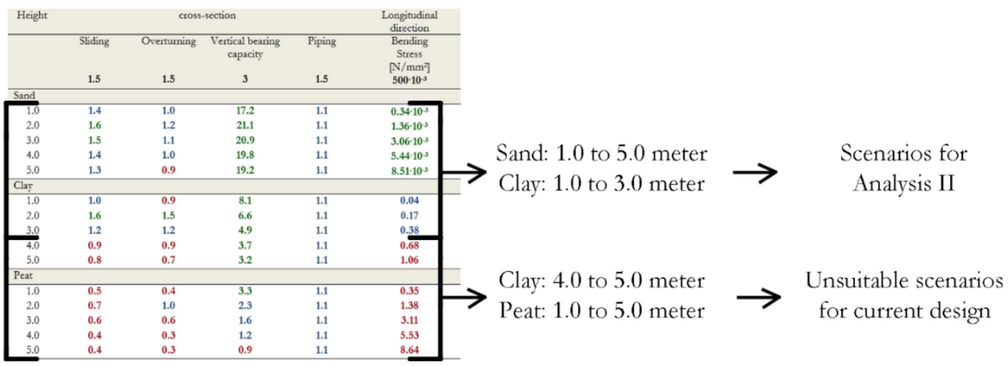


Figure 5-2: Scenarios for analysis II

5.3. RESULTS ANALYSIS II

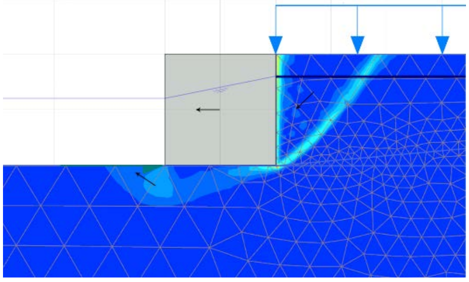
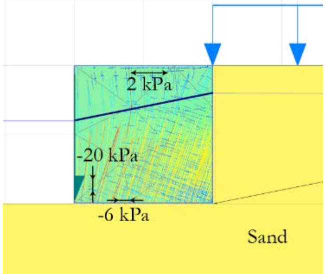
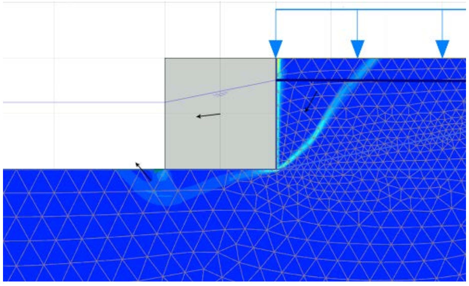
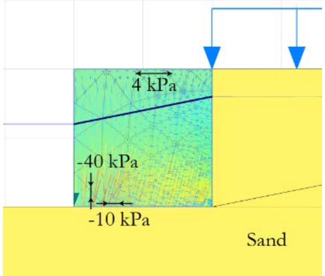
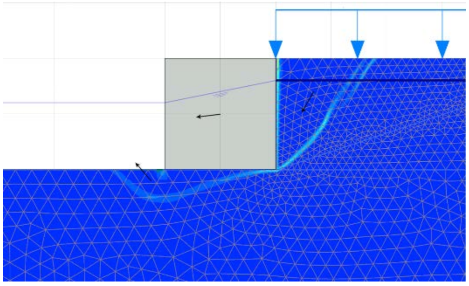
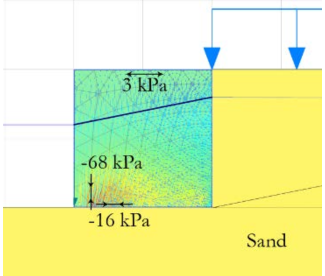
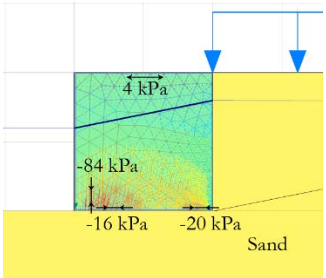
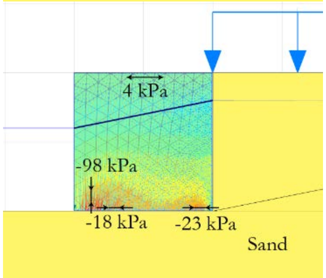
Safety analysis II calculates the safety for a one to five metre retaining wall on sand and a one to three metre retaining wall on clay. The input for all scenarios is provided in the previous chapter. The calculations are separated in two groups: calculations in the cross-section and calculation in the longitudinal direction. Overall stability and structural failure are checked in the cross-section and failure due to large deformations is checked in the longitudinal direction.

CROSS-SECTION

Overall stability and structural failure are checked in the cross-section. The results are presented in the table on the following page. Failure due to overall stability is visualised with a light blue line. As stated in chapter 4.3 the overall stability is calculated by reducing the shear strength parameters of the soil until a slip circle has developed. This light blue line is a representation of the slip circle at the moment of failure. In addition arrows are added to the plot. These arrows provide the direction of the structure and the soil at the moment of failure.

The internal stresses are visualised by plotting the principal effective stresses in the structure. Red indicates the larger stresses and blue indicates the smaller stresses. The maximum and minimum principal effective stresses are mentioned at the right side of the table on the following page. In addition horizontal and vertical stresses are indicated in the plots with arrows. The arrows pointing away from each other are tensile stresses and arrows pointing towards each other are compressive stresses. The arrows indicate locations with large tensile stresses and large compressive stresses.

The outcomes of analysis II are presented with the tree colour as explained in the introduction.

Retaining Height	Overall stability [slip circle at failure] **1.5**	Horizontal and vertical internal effective stresses [kPa]	Max / Min principal effective stresses σ'_3 (tensile)	σ'_1 (compr.)
			70	**-3000**
1m	 **SF = 1.3**		**2.26**	**-22.40**
2m	 **SF = 1.3**		**5.27**	**-46.07**
3m	 **SF = 1.2**		**7.08**	**-72.02**
4m			**11.69**	**-89.54**
5m			**13.17**	**-105.0**

According to the results from the table on the previous page the considered conceptual design of the GEOWALL is potentially safe on sand for retaining heights between one and three metres and the current design is unsafe for structures larger than three metres. The results are quite unexpected, in particular the results for overall stability. Sand is known to be a strong soil, therefore the low values for overall stability seem very unlikely. An explanation is given based on theory and the plots.

Overall failure develops over the line where the shear stresses exceed the critical shear stress. The formula for the critical shear stress (τ_f) in a soil body is according to Verruijt (2010):

$$\tau_f = c + \sigma_n' \cdot \tan(\varphi)$$

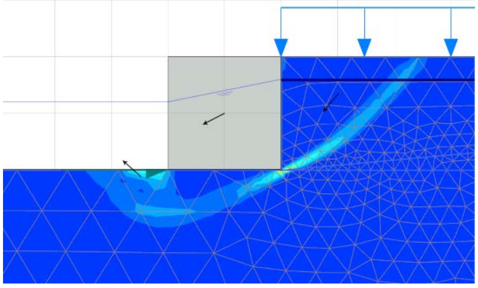
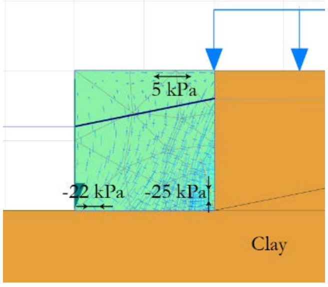
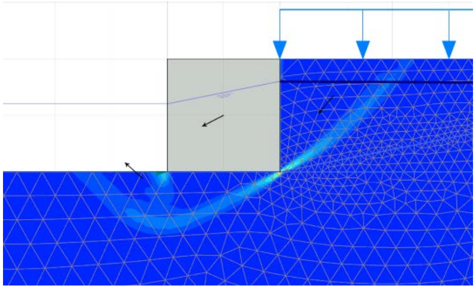
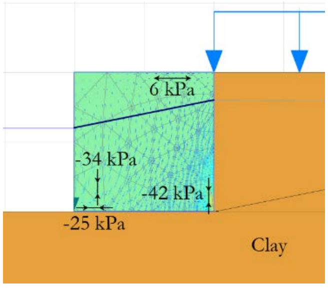
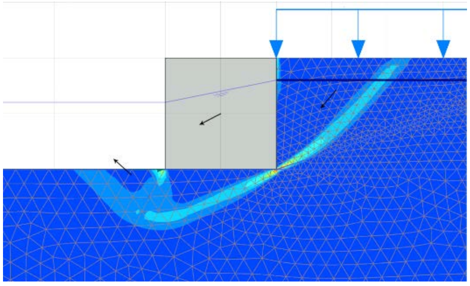
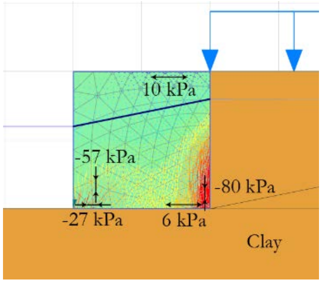
The critical shear stress is a sum of the cohesion (c) and the normal effective stress (σ_n') at the considered plane times the tangent of the angle of internal friction(φ). For sand the cohesion is zero. In addition, the effective normal stress is minimal at the plane between the bottom of the structure and the soil and increases over the depth. The critical shear stress is therefore easily reached near the bottom of the wall. This explains the failure line (light blue in the plots) close to the bottom of the structure.

The failure line suggests that the finite element method is not showing overall failure, but horizontal equilibrium failure. The slip circles in the overall stability failures plots are situated close to the bottom of the structure and the orientation of the arrows in these plots are nearly horizontal. To check whether this is true, safety factors for sliding from the previous analysis are compared with the results of the finite element method.

Retaining height (sand) [m]	Sliding analytical method	Finite element model	Difference
1.0	1.4	1.3	7.1%
2.0	1.6	1.3	19%
3.0	1.5	1.2	20%
4.0	1.4	1.0 or lower	29%
5.0	1.3	1.0 or lower	23%

From this comparison it cannot be concluded that the finite element method is showing horizontal failure. The safety factors for sliding are very different from the overall stability safety factors in almost any case.

In all scenarios the internal stresses are never exceeding the permissible stresses. The internal stresses are much lower than the maximum tensile stress of 0.07 MPa and the minimum compressive stress of 3.0 MPa. An explanation for this is that the structure is considered as a rigid structure. The maxima of the principal effective stresses are found in the bottom left corner of the structure. This is as expected, since the lateral soil pressure is pushing the wall in that specific direction.

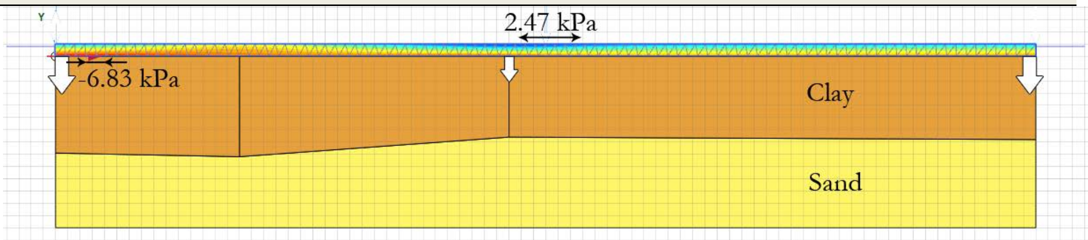
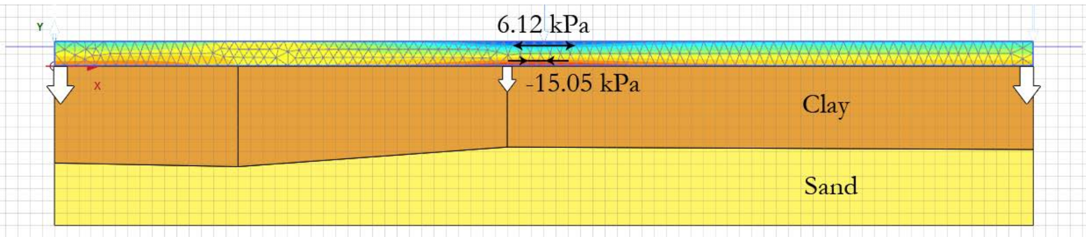
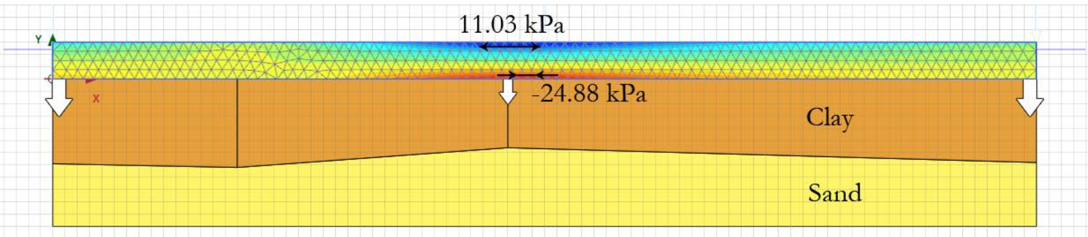
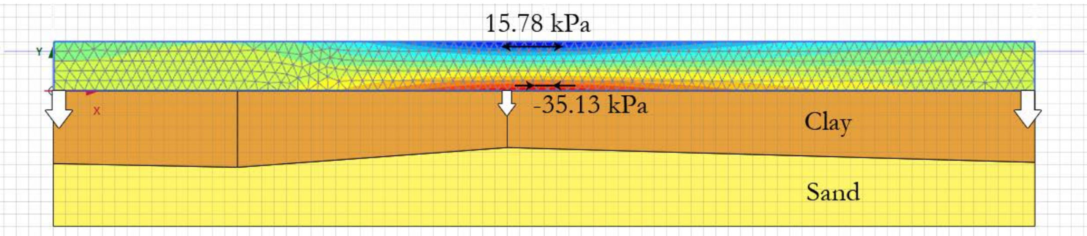
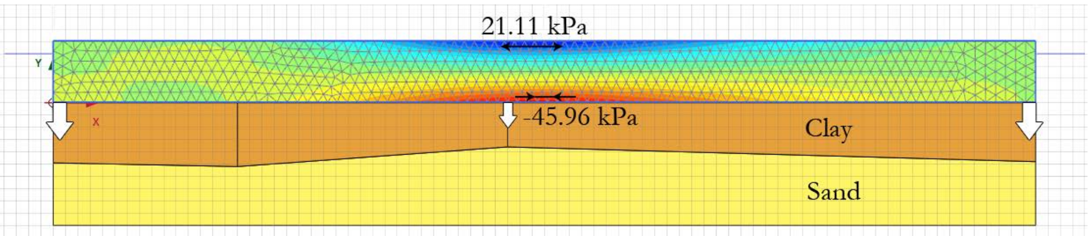
Retaining Height	Overall stability [slip circle at failure]	Horizontal and vertical internal effective stresses [kPa]	Max / Min principal effective stresses σ'_3 (tensile)	σ'_1 (compr.)
	1.5		**70**	**-3000**
1m	 **SF = 3.3**		**5.16**	**-76.49**
2m	 **SF = 2.2**		**6.90**	**-111.5**
3m	 **SF = 1.7**		**48.33**	**-167.7**

According to the results from the table above the considered conceptual design of the GEOWALL is safe on clay for retaining heights between one and three metres. The slip circles in the overall stability failure plots are situated within the soil layer. At the moment of failure, the arrows indicate a combination of a horizontal and rotational movement of the structure.

The principal effective stresses are much lower than the permissible tensile stress of 0.07 MPa and the permissible compressive stress of 3.0 MPa. The large vertical compressive stresses on the active side (right) of the structure are unexpected large. This can be explained by wall friction. The surcharge is pushing the soft cohesive clay next to the structure in a downward direction. The vertical effective stresses in the soil are transferred to the right side of the structure. This vertical wall friction holds a friction component and a cohesive component. The friction component is stress dependent: zero at the surface and linearly increasing over the depth. The cohesive component is stress independent and constant next to the wall. As the stresses increase over the depth the shear stress is maximum near the bottom at the active side of the wall. This consequentially results in the large vertical compressive stresses in the right corner of the wall.

LONGITUDINAL DIRECTION

Failure due to large deformations is checked in the longitudinal direction. The finite element model gives the stresses in the structure and the settlement after approximately 30 years. The results are presented in the table below. The settlement is visualised with the white arrows. The size of the arrow indicate the relative amount of settlement. The internal stresses are visualised by plotting the effective horizontal stresses. Red indicates compressive stresses and blue indicates tensile stresses. The maximum and minimum horizontal effective stresses are given. They are indicated with by arrows. The arrows pointing away from each other are tensile stresses and arrows pointing towards each other are compressive stresses.

Retaining Height [m]	Horizontal effective internal stresses + settlement
1m	
2m	
3m	
4m	
5m	

According to the results from the table on the previous page the considered conceptual design of the GEOWALL is safe for retaining heights between one and five metres on a varying subsoil of clay and sand. The horizontal effective stresses are much lower than the permissible tensile stress of 0.07 MPa and the permissible compressive stress of 3.0 MPa. Interesting to notice is the correlation between the thickness of the clay layer, the amount of settlement and the location of the maximum stresses. In the considered longitudinal domain the clay layer is maximal at the edges of the domain and minimal in the middle. A larger compressible layer coincides with more settlement. This is also in agreement with the formula of Koppejan (see chapter 3.3). As a result of the variation in the shape of the layer enormous moments are created in the middle of the domain. These moments are taken in by the cross-sectional surface of the wall causing a tensile stress in the upper part and a compressive stress in the lower part of the wall.

5.4. CONCLUSION

The goal of this chapter is an overview of the challenges that have to be encountered in larger GEOWALLs. This goal is achieved by comparing the results from the analyses with the test values and the maximum permissible stresses. There are three possible outcomes:

1. The calculated values are larger than the test values for the Ultimate Limit State.
2. The calculated values are less than the test values for the Ultimate Limit State and larger than the test values for the Serviceability Limit State.
3. The calculated values are less than the test values for the Serviceability Limit State.

From safety analysis I it can be concluded that the considered conceptual design of the GEOWALL is potentially safe on sand for retaining heights between one and five metres and on clay for retaining heights between one and three metres. The current design of the GEOWALL is unsafe on clay soils larger than three metres and peaty soils. A design improvement is required for these unsafe scenarios. This design improvement should be able to limit the possibility of failure due to sliding, overturning and failure trough large deformations.

From safety analysis II it can be concluded that the considered conceptual design of the GEOWALL is safe on sand and clay for retaining heights between one and three metres in the cross-section. The current design is unsafe for structures larger than three metres. A design improvement is required for these scenarios. This design improvement should be able to limit the possibility of failure due to sliding. For a varying subsoil of clay and sand in the longitudinal direction a GEOWALL is always safe.

Both analysis combined it can be concluded that sand-GEOWALLs on sand between one and three metres and clay-GEOWALLs on clay between one and three metres are the most viable scenarios for a larger scale GEOWALL. For GEOWALL quay walls larger than three metres and peat-GEOWALLs on peat suitable design improvements are required.

Chapter 6

VERIFICATION AND SENSITIVITY ANALYSIS

This chapter describes the verification of the finite element model. The verification is carried out by cross-checking the analytical calculation with the finite element calculations. In addition to the verification a sensitivity analysis is carried out to investigate the influence of a variation in the GEOWALL weight to the potential safe scenarios.

6.1. INTRODUCTION

The previous chapter presented the results of the two analyses. This chapter focuses on the verification of these results. In particular the results from the finite element analysis. Additionally a sensitivity analysis is carried out to determine the influence of a variation in the weight of a GEOWALL element on the safety against sliding and overturning for sand, clay and peat.

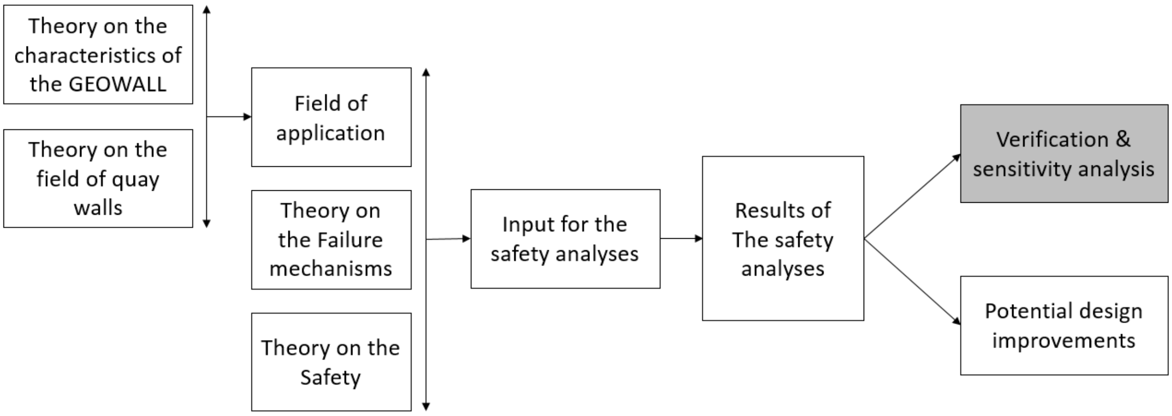


Figure 6-1: Chapter 6 in the research framework

6.2. VERIFICATION

A verification is carried out to verify the finite element model for the modelled GEOWALL structure. This is done by checking the results from the analytical calculations and a Limit equilibrium model with the results from the finite element model. The horizontal and vertical stresses and forces are checked with the analytical calculations. The overall stability is checked with the Limit Equilibrium Model D-Geo stability. The output of both methods is placed next to each other. By comparing both outputs, something can be said about the verification of the finite element model.

HORIZONTAL AND VERTICAL STRESSES AND FORCES

The horizontal and vertical stresses and forces are checked with the analytical method. As input the following two scenarios are applied: A two metre retaining wall on sand and a two metre retaining wall on clay. The two results are checked on the following aspects:

- The vertical bearing stresses and the distribution of these stresses over the width
- The equivalent vertical force
- The horizontal effective soil stresses and the distribution of these stresses over the height
- The equivalent horizontal force

Scenario 1

The first scenario is a sand-GEOWALL with a retaining height of two metre placed on a sandy subsoil. As can be seen in the results below the horizontal and vertical forces are very similar. The horizontal stresses are also very similar, however the development of the vertical effective stresses differ over the width.

Analytical method **Finite element method**

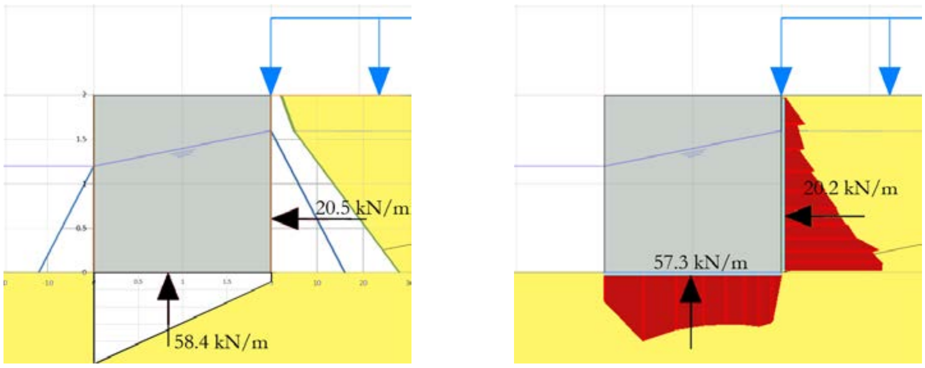


Figure 6-2: Horizontal and vertical stresses and equivalent forces on the sand-GEOWALL

Scenario 2

The second scenario is a clay-GEOWALL with a retaining height of two metre placed on a clay subsoil. As can be seen in the results below the horizontal and vertical forces are less similar than the previous scenario. In addition the development of the horizontal and vertical effective stresses are very different from each other. Both differences can be explained by the friction between the wall and the soil. The finite element model takes this wall friction into account where the analytical method ignore this phenomenon. The analytical calculations therefore results in a perfect distribution over the height and the width of the structure.

Analytical method **Finite element method**

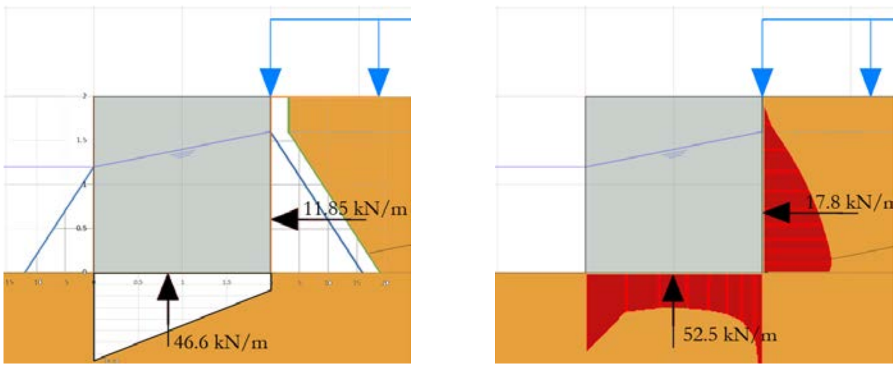
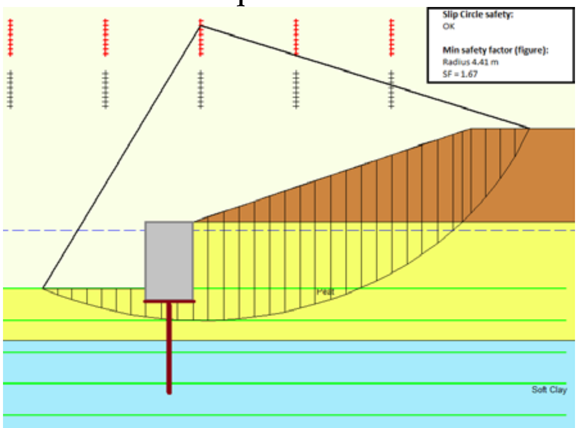


Figure 6-3: Horizontal and vertical stresses and equivalent forces on the clay-GEOWALL

OVERALL STABILITY

The overall stability is also calculated with a linear equilibrium model (D-Geo Stability). The situation is based on a case for the second pilot at Flood Proof Holland (Appendix J). The results for the overall stability are very similar (see Figure 6-4). It can therefore be concluded that the finite element model also works for the overall stability calculations of the GEOWALL.

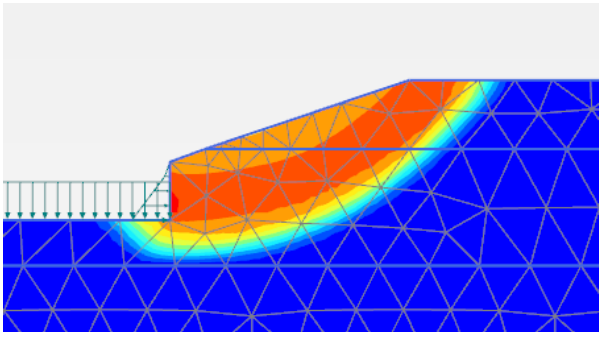
Linear equilibrium model



Safety Factor = 1.67

Figure 6-4: Overall stability by D-Geo Stability

Finite element model



Safety Factor = 1.63

Figure 6-5: Overall stability by Plaxis2D

6.3. SENSITIVITY ANALYSIS

The relation between the weight of a GEOWALL element and the safety against sliding and overturning for sand, clay and peat is given in the three table below. The basis case is a two metre high retaining structure. The basis cases are highlighted in grey. The difference between a new situation and the basis case divided by the new case is called the relative difference. This relative difference shows whether the safety factor is going up or down.

	Sand			
	Sliding		Overturning	
Weight Wet/dry	Safety Factor	Relative Difference	Safety Factor	Relative Difference
18/16	1.2	-27%	0.9	-27%
20/18	1.4	-14%	1.1	-14%
22/20	1.6	0%	1.2	0%
24/22	1.9	14%	1.4	14%
26/24	2.1	27%	1.6	27%

A lighter sand element results in smaller safety factors for both sliding and overturning. A heavier sand element results in larger safety factors for sliding and overturning.

Clay				
	Sliding		Overturning	
Weight Wet/dry	Safety Factor	Relative Difference	Safety Factor	Relative Difference
15/12	**1.0**	**-35%**	**1.0**	**-35%**
17/14	**1.3**	**-18%**	**1.2**	**-18%**
19/16	**1.6**	**0%**	**1.5**	**0%**
21/18	**1.8**	**18%**	**1.8**	**18%**
23/20	**2.1**	**35%**	**2.0**	**35%**

A lighter clay element results in a smaller safety factors for both sliding and overturning. A heavier clay element results in larger safety factors for sliding and overturning.

Peat				
	Sliding		Overturning	
Weight Wet/dry	Safety Factor	Relative Difference	Safety Factor	Relative Difference
12/10	**0.4**	**-47%**	**0.5**	**-47%**
14/12	**0.5**	**-23%**	**0.8**	**-23%**
16/14	**0.7**	**0%**	**1.0**	**0%**
18/16	**0.9**	**23%**	**1.2**	**23%**
20/18	**1.0**	**47%**	**1.5**	**47%**

A lighter peat block results in a smaller safety factors for both sliding and overturning. A heavier peat block results in larger safety factors for sliding and overturning. Peat elements with a dry and wet saturated specific weight of respectively 18 kN/m^3 and 20 kN/m^3 are potentially safe. For all other weights the scenarios with peat block remain unsafe.

6.4. CONCLUSION

Based on the comparison between the analytical results and the results from the finite element model of a two metre retaining wall on sand, it is concluded that the finite element model is able to give a good representation of the horizontal and vertical forces. Based on the comparison on clay, it is concluded that the finite element model also includes wall friction in the calculations.

The relation between the weight of a GEOWALL element and the safety against sliding and overturning for sand, clay and peat in the cross-section is summarised in two graphs. One graph for sliding and one for overturning. In both graphs the variation of the wet specific weight of the elements is found on the x-axis and the corresponding safety factor on the y-axis. The slope determines the sensitivity. A steep slope corresponds with a large sensitivity and a gentle slope corresponds with a small sensitivity. What also can be seen is the required weight for a safe or potentially safe situation.

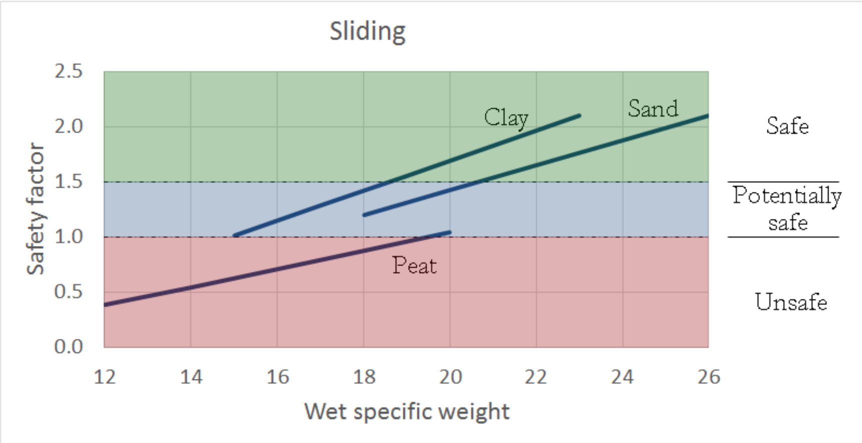


Figure 6-6: Sensitivity analysis - variation in weight vs SF for sliding

Peat elements with a dry and saturated specific weight of respectively 18 kN/m^3 and 20 kN/m^3 are potentially safe. For all other weights the scenarios with peat block remain unsafe. The variation in the weight of clay has most influence on the safety factor, than sand and finally peat. In percentage the relative sensitivity (difference in weight/difference in safety factor) is respectively 14.0%, 11.0% and 8.0%.

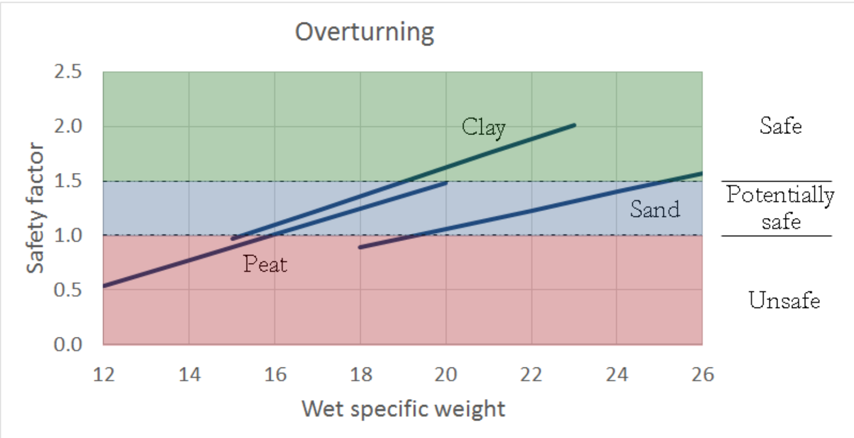


Figure 6-7: Sensitivity analysis - variation in weight vs SF for overturning

Peat elements with a dry and saturated specific weight of respectively 16 kN/m^3 and 14 kN/m^3 are potentially safe. This is less weight than for sliding. The variation in the weight of clay has most influence on the safety factor, than peat and finally sand. In percentage the relative sensitivity (difference in weight/difference in safety factor) is respectively 13.0%, 12.0% and 8.0%.

It can be concluded that a variation in the weight of clay has most influence on the safety factors sliding and overturning for current GEOWALL design. A variation in the weight of peat has least influence on the safety factor for sliding. A variation in the weight of sand has least influence on the safety factor overturning.

Chapter 7

POTENTIAL DESIGN IMPROVEMENTS

The goal of this chapter is to find potential design improvements to cope with the challenges from chapter five. Design improvements are variations on the conceptual design. It has to be noted that all design improvements are created from a technical point of view. They all have the potential to improve the original conceptual design. A qualitative analysis is carried out for eight different adjustments. This analysis evaluates the designs with respect to the failure mechanisms of a GEOWALL.

7.1. INTRODUCTION

The conceptual design is presented in chapter 2.5. This conceptual design is stated to be practically simple and commercially attractive. The design is subsequently analysed and the unsafe situations for such a design are concluded in chapter 5. The goal of this chapter is to find potential design improvements to cope with these challenges. This goal is reached by evaluating several potential design improvements. The impact of each potential design improvement with respect to the seven failure mechanisms is qualitatively described.

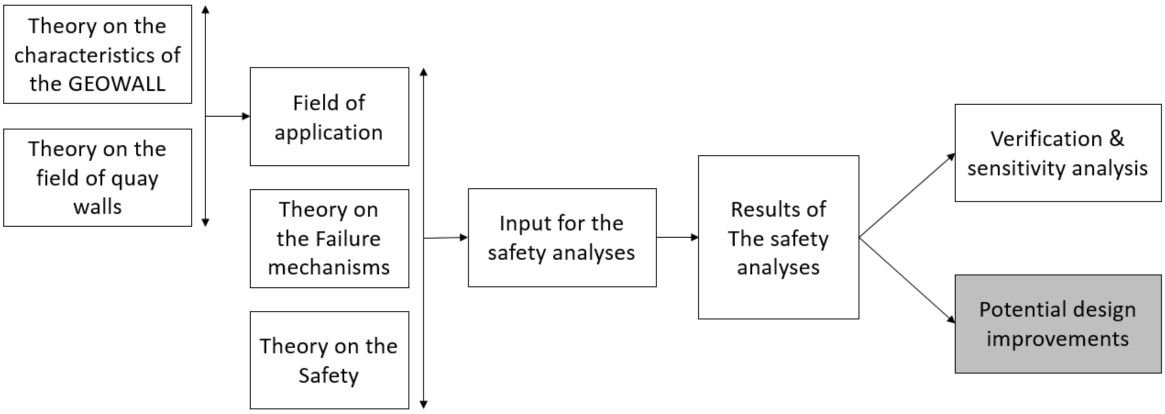


Figure 7-1: Chapter 7 in the research framework

The design improvements are obtained from a brainstorm session with NETICS and the brickwork retaining walls design guide (Haseltine, 1991). The different improvements are categorised in three classes: material improvements, geometrical design improvements and structural design improvements. The first, material improvements is initially taken outside the scope of this study and will be described shortly in the second paragraph. The two design improvements are elaborated in respectively paragraph three and four.

7.2. MATERIAL IMPROVEMENTS

Material improvements are improvements of the construction material. The material can be improved in terms of weight and strength. Weight can be increased or decreased depending on the types of available soil. The strength can be increased by improving the connections between the elements in the structure or by increasing the internal strength of the elements. The connections can be improved with natural binders such as 'hennep'. The internal strength of the elements is influenced by various parameters such as the type of soil, the type and amount of stabiliser, the water content of the soil, the magnitude and duration of the applied pressure and the curing period. NETICS is continuously studying these effects. Their model contains besides the strength, the resistance of the element: the resistance against erosion, whetting/drying, shrinkage, temperature changes and currents/waves/tides.

7.3. GEOMETRICAL DESIGN IMPROVEMENTS

The improvements presented in this paragraph do not require any additional construction materials, such as wood, stone or bricks. The designs are modifications in the geometry of the conceptual design. A variation in the volume of used GEOWALL material is not excluded in these adjustments. The following four geometrical design improvements are qualitatively analysed:

1. Wider wall
2. Embedded wall
3. Triangular wall
4. Stepped wall

WIDER WALL

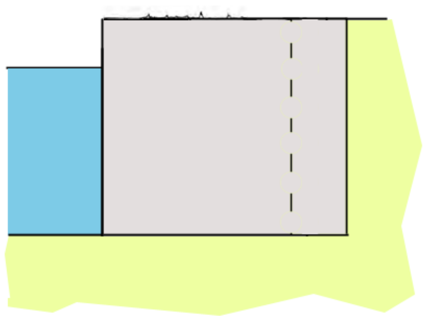


Figure 7-2: Design improvement - wider wall

A wider GEOWALL is created by constructing additional elements on the soil side of the wall (see Figure 7-2). Due to the extra weight the structure is less likely to slide away. Furthermore the length of the interface between the wall and the soil is larger than the conceptual design. This results in extra friction and extra length to counteract piping. From a commercial point of view the extra elements and the extra labour raises the costs of the wall per running metre. From a practical point of view additional width is required which is not always available.

EMBEDDED WALL

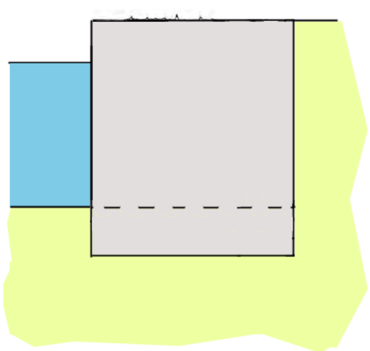


Figure 7-3: Design improvement - embedded wall

An embedded GEOWALL is created by sinking part of the wall into the subsoil (see Figure 7-3). A passive soil pressure can now develop at the waterside of the wall. Due to this passive soil pressure, an embedded construction is very useful in counteracting failure against sliding and overturning. It has to be noted that the friction at the dotted line is supposed to be larger than the soil-wall friction. Otherwise the structure will slide away internally over this boundary. A commonly used material in brickwork retaining walls is mortar. Mortar is able to bind the individual elements in the structure. This consequently increases the internal friction. The emerging challenge of an embedded wall an increase of the internal stresses. The lower part wall is clamped into the soil, while lateral soil pressure is pushing the upper part of the wall in a horizontal direction. This results in additional tensile stresses in the lower right part of the wall. The length of the interface between the wall and the soil is larger, which makes it easier to counteract piping. From commercial point of view extra elements are necessary and more subsoil should be excavated from the bank. This extra material and labour raises the cost of the wall per running metre.

TRIANGULAR WALL

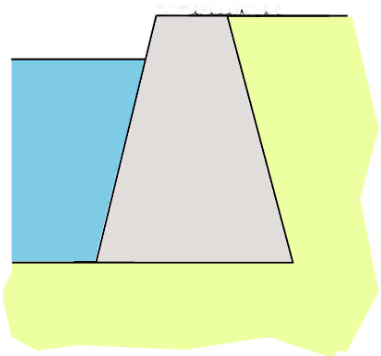


Figure 7-4: Design improvement - triangular wall

A triangular wall is characterised by a smaller upper part and a wider lower part (see Figure 7-4). This design can be explained by the variation of the horizontal soil pressure over the height of the wall. The horizontal pressure increases with the depth. The thickness of the wall increases with the same rate as the horizontal soil pressure over this depth. In addition wall friction under an angle will result in active pressures acting in a more vertical than horizontal direction. The triangular design improves the resistance against sliding and overturning. It is unclear if more or less material is required compared to the conceptual design. From a practical point of view it is harder to construct a wall under an angle than a vertical wall.

STEPPED WALL

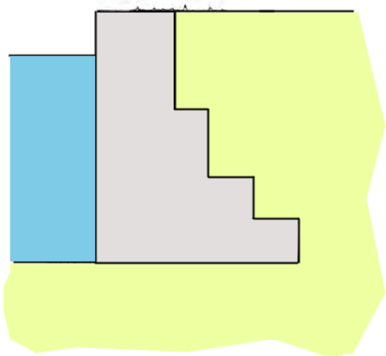


Figure 7-5: Design improvement - stepped wall

A stepped wall is characterised by a stepped shape at the active side of the wall (see Figure 7-5). Soil pressure can now act in both horizontal and vertical direction against the wall. The horizontal soil stresses are similar to the conceptual design. The vertical soil stresses 'pushes' the right side of the structure in a downward direction, counteracting the overturning moment. Note that this is only possible in case of a monolithic structure. A wall with separate elements is not able to transfer the vertical soil pressure properly to the rest of the structure. Thus making a stacked blocked wall without any internal connections unsafe for this improvement. Besides, this design will result in larger internal stresses and is therefore more prone to structural failure.

7.4. STRUCTURAL DESIGN IMPROVEMENTS

Structural design improvements are design improvements which require additional materials. This varies from steel tension bars to plastic weep holes. The following four structural design improvements are qualitatively analysed:

1. Strengthened walls [reinforcement, anchoring, pre-stressing]
2. Drainage system [weep holes, granular material, geotextile]
3. Foundation
4. Armoured wall [front & top]

STRENGTHENED WALL

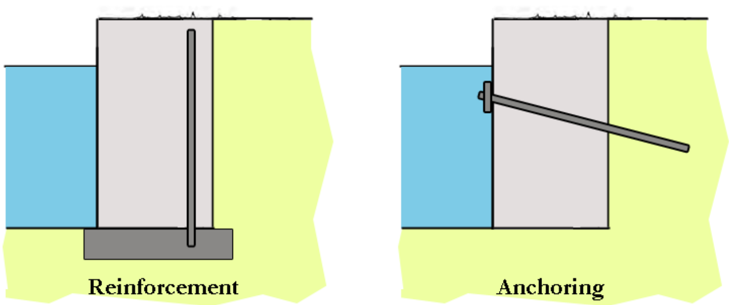


Figure 7-6: Design improvement – strengthened wall

A strengthened wall is a wall with reinforcements or anchoring or it can be prestressed elements. A reinforced wall is a wall with reinforcing bars to cope with internal stresses. Figure 7-6 shows a tensile bar coupled to a footing. This tensile bar can withstand the tensile forces in the structure and directly binds the structure. It has to be noted that such a costly and unsustainable improvement discards the whole idea of a GEOWALL: "a sustainable and commercially attractive quay wall made of locally available sediment". An anchored wall is identified by an anchor which connects the wall with ground behind the wall (see Figure 7-6). This solution is commonly used in sheet pile walls. It is very effective to cope with bending moments in the structure and to cope with equilibrium failure. Note that internal stresses might exceed the permissible stresses at the point where the anchor is connected to the structure. Another disadvantage of an anchored wall is the need for sufficient space; soil has to be dug out behind the wall to place the anchor. A prestressed wall is a method to overcome tensile stresses in the structure. Extra compressive stresses are applied prior to installation. This extra compressive stress consequentially balances the occurring tensile stresses after installation as a result of the bending moments. The disadvantage of all three improvements is the additional costs for the extra material and the equipment for the instalment.

DRAINAGE SYSTEM

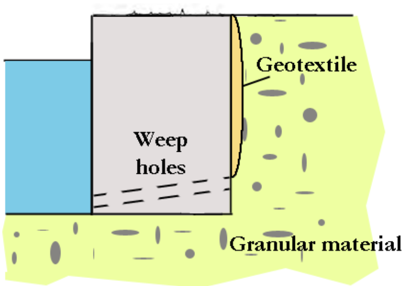


Figure 7-7: Design improvement - drainage systems

The goal of a drainage system is to reduce the lateral soil pressure on the active side of the wall. This is reached by diminishing the water level difference (head) between the ground water and the waterway. There are several drainage systems available which are able to reach this goal. Three systems are discussed: weep holes, granular filter and geotextile (see Figure 7-7). A weep hole is a pipe through the lower part of the structure which allows water to flow easily from the active side to the waterway. Granular material is material with an open structure, for example sand or rubble. The open structure enables the water to flow through the pores of the granular layer. The material replaces the soil behind the wall and beneath the wall. A geotextile is a permeable material which can be used as filter, protection and drainage. The design improvement introduces a geotextile fabric between the wall and the retained soil. Figure 7-7 shows how the ground water is directed to the weep hole. The limited lateral stresses as a result of these three design improvements provide extra safety against sliding and overturning. Above all adding a drainage system eliminates the failure due to piping. A major disadvantage is the possibility of clogging: sediment blocking the flow through the weep hole. From a practical point of view adding a geotextile is expected to be least labour intensive. From a commercial point of view a weep hole is probably the least expensive solution.

FOUNDATION

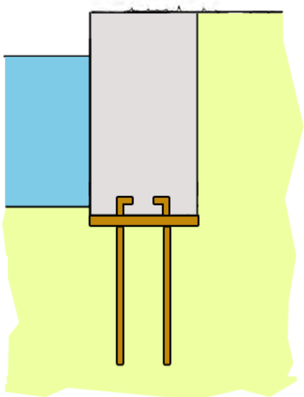


Figure 7-8: Design improvement - foundation

A foundation is a structure under the wall which transfers the weight/loads of the structure to the subsoil. This is only necessary in case of weak soils such as clay or peat. This design improvement is able to counteract geotechnical failure as well as equilibrium failure. The most important advantage of a foundation is its resistance against differential settlement. Due to the foundation there is less differential settlement in the longitudinal direction which diminishes the bending stresses inside the structure.

ARMOURED WALL

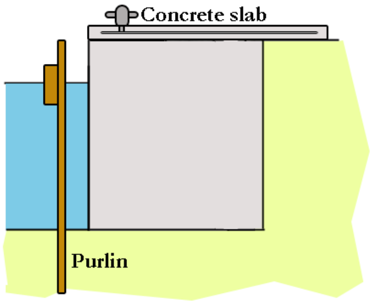


Figure 7-9: Armoured wall

An armoured wall is characterized by additional elements on the top or at the front of the structure. Two types of armour are evaluated: A concrete slab and a purlin (see Figure 7-9). A concrete slab is a horizontal reinforced plate which is placed on top of the wall. Boulders can be attached to the reinforcement inside the slab to cope with the mooring loads. Note that these mooring forces need to be transferred to the wall which results in larger internal stresses. Due to the extra weight of such an element on top of the structure equilibrium failure is less likely to occur. However the subsoil has to cope with extra pressure and the structure has to cope with larger stresses, increasing the chance of failure as a result of vertical bearing capacity or internal failure. A purlin is a horizontal beam along the length of the wall. This beam provides extra strength against collision of berthing boats.

7.5. CONCLUSION

The goal of this chapter is to find potential design improvements to cope with the increased loads when upscaling the conceptual GEOWALL. There are three categories of potential improvements: material improvements, geometrical design improvements and structural design improvements.

Material improvements are improvements of the construction material. The material can be improved in terms of weight and strength. Both depend on the soil type and many other (local) parameters such as the moisture level and the construction method of the GEOWALL elements.

The geometrical design improvements do not require any additional construction materials, such as wood, stone or bricks; they are variations on the original geometry. The four geometrical design improvements are: a wider wall, an embedded wall, a triangular wall and a stepped wall. From these four improvements the embedded wall is expected to be most promising at this moment.

Structural design improvements are design improvements which require additional materials. The four structural design improvements are: strengthening of the wall, adding a drainage system, placing foundation and putting armour at the front or on the top of the wall. From these four improvements the drainage system is expected to be most promising at this moment.

It is recommended to study the effects and efficiency of the proposed design improvements in further research.

Chapter 8

CONCLUSIONS AND RECOMMENDATIONS

This chapter holds the conclusions and recommendations. The conclusion provides answer to the central question of this research project. The recommendations provide a guidance for further research on this topic. The recommendations are separated in five themes. Every theme holding one or more aspects which are recommended to investigate in future research.

8.1. CONCLUSIONS

> What is the most viable field of application of the considered conceptual design of the GEOWALL within the field of quay walls and what are the arising challenges of a larger scale GEOWALL for a varying retaining height restricted by the field of application and for different types of soil?

The GEOWALL is sustainable and commercially very attractive and will initially be constructed as a squared shaped gravity type structure. The considered GEOWALL is identified as a potential alternative for a wooden sheet pile wall up to three metre and a stone gravity wall up to five metres. The GEOWALL defined in this study will be applied as gravity type structure in small waterways and ponds within the field of quay walls.

The arising challenges are found by assessing different scenarios. A scenario is defined by a combination of a retaining height and a type of soil. The retaining heights follow from the field of application and vary between one and five metres. The types of soil are simplified to sand, clay and peat. The retaining height and the type of soil determine all other input values. The output of both analyses form a set of safety factors and internal stresses. There are three possible outcomes:

1. The calculated values are larger than the test values for the Ultimate Limit State.
2. The calculated values are less than the test values for the Ultimate Limit State and larger than the test values for the Serviceability Limit State.
3. The calculated values are less than the test values for the Serviceability Limit State.

Every scenario is analysed and returns a set of safety factors and internal stresses. These safety factors are compared with the general factors of safety. These general factors of safety are: sliding – 1.5, overturning – 1.5, vertical stability – 2.5, overall stability – 1.5 and piping – 1.5. The internal stresses are compared with the permissible stresses. The maximum tensile strength is 0.07 MPa and the minimum compressive strength is 3.0 MPa. These values are based on averaged values of compressed stabilised earth blocks.

From the results of the analytical analysis and the finite element analysis it can be concluded that sand-GEOWALLs on sand between one and three metres and clay-GEOWALLs on clay between one and three metres are the most viable scenarios for a larger scale GEOWALL. For GEOWALL quay walls larger than three metres and peat-GEOWALLs on peat suitable design improvements are required. The design improvements should be able to limit the possibility of failure due to sliding, overturning and failure trough large deformations.

There are three categories of potential improvements: material improvements, geometrical design improvements and structural design improvements. The embedded wall is expected to be the most promising geometrical design improvement. Adding a drainage system is expected to be the most promising structural design improvement at this moment.

8.2. RECOMMENDATIONS

This paragraph provides several recommendations for further research. The recommendations regard the simplifications, the material of the GEOWALL, the safety, the way of modelling and the practical & commercial aspects.

SIMPLIFICATIONS

The research includes four main simplifications: simplifications of the soils, simplification of the initial design, simplification of the longitudinal geometry and simplification of the loads. This research simplifies all soil types into three main categories, namely sand, clay and peat. However, there is a wide range of different soil types, ranging from 100% sand to 100% clay to 100% silt. Within this range all kinds of combinations are found: silty clay, clayey sand, loamy sand, etc. It is recommended to consider more different types of soil in future research. The conceptual design for this study is simplified to a massive squared block. In practice several elements are stacked on top of each other. The difference between a monolithic wall and stacked elements is interesting to investigate as well as what the optimal shape and size of the elements. The longitudinal geometry is based on one location in The Netherlands. In practice the longitudinal geometry varies in layer thicknesses, type of soils, angle of the plane between the layers, etc. The loads are limited to the surcharge, head difference, self-weight and differential settlement. The rest of the loads are assumed to have little influence. For further research on the GEOWALL it is recommended to investigate the influence of the other governing loads.

GEOWALL MATERIAL

The research considers a GEOWALL element with the strength of an averaged compressed stabilised earth block. Within this study this strength is used for all types of compressed GEOWALL elements: the sand-GEOWALL, clay-GEOWALL and peat-GEOWALL. The strength is dependent on many aspects such as the type of soil, type and amount of stabiliser, water content, magnitude and duration of the applied pressure and the curing period. NETICS is continuously studying these effects. Their model contains besides the strength, the resistance of the element: the resistance against erosion, whetting/drying, shrinkage, temperature changes and currents/waves/tides.

SAFETY

Currently three groups are identified for the results of the analysis: safe, potentially safe and unsafe. The groups are categorised based on the factors of safety of the Ultimate Limit State and the Serviceability Limit State. The Ultimate Limit State test values determine the difference between a safe and potentially safe situation. These test values depend on the location and the application. If the consequences of failure are minimal, the factors of safety can be low. If the consequences are high, the factors of safety should be higher. The values for this research originate from literature on quay wall design which are very conservative for the application of the GEOWALL. The study showed that the GEOWALL is most viable for smaller retaining heights (< 3 metres). It is therefore recommended to work with lower Ultimate Limit State test values in case of smaller retaining GEOWALLs. This will result in more safe scenarios.

Potentially safe scenarios are situated between the ultimate limit state and the Serviceability Limit State. This means that for example cracks might occur, but without total failure of the wall. It depends on the location and the application if potentially safe scenarios can still be applied. If the consequences of failure are minimal, the factors of safety can be low. If the consequences are high, the factors of safety should be higher. It is recommended to investigate the consequences for every specific situation on its own. Doing so, it can be decided whether the potentially safe scenarios are still applicable or not.

MODELLING

The analyses are carried out with an analytical and a finite element model. The analytical calculations provide a good estimate and can be used to answer the research questions of this study. However, the design of a GEOWALL for specific locations should be checked with a more advanced modelling programme. Aspects such as wall friction and inclination are currently taken out of the analytical calculation and are expected to have a proportional influence on the results.

THE PRACTICAL AND COMMERCIAL ASPECTS

The proposed design improvements in this study are created from technical point of view. The design improvement should not only be technically effective, but also practically possible and commercially attractive. Technical effective means that adjustments to the conceptual design make unsafe situations possible. Reinforcement for example is a technical effective solution for exceeding tensile stresses in a larger structure. Practical possible means that the structure can be built on a certain way. A wider GEOWALL without sufficient construction space is from practical point of view not the optimal improvement. Commercially attractive means that the improvement is not too expensive compared to where the structure is going to be used for. Something can be said about this aspect when prices are coupled to the improvements. The effect of the improvement in combination with the price that has to be paid for the adjustment leads to the commercially most efficient result. It is recommended to take not only the technical, but also the practical and commercial aspects in consideration in the choice of the design improvements.

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APENDICES

Appendices A t/m J

APPENDIX A – PROJECT DESIGN

STEP-BY-STEP APPROACH FOR THE RESEARCH OBJECTIVE

Step 1: Project type

Is it a theory-oriented or practice-oriented research project?

This thesis is identified as a practice-orientated research project, since the researcher is commissioned by an external party. It is also meant to provide knowledge and information that can contribute to a successful intervention in order to change an existing situation

Step 2: Project context

What problems are involved within the project context?

The construction of the GEOWALL is very similar to other constructions, however NETICS told about many challenges that they have to be overcome. They want to know when it is safe to construct a GEOWALL, what the design looks like, what the decisive properties are knowledge about the construction technique.

Feasibility | Design | Strength | Construction technique

What is the background to these problems?

The background to these problems is that there is little known about the compositions of different sediment types and additives and their coinciding strength. Buildings have been built with compressed earth blocks (CEBs) of ideal sediment; however a retaining bank structure is a new application, introducing other forces on the construction material and different soil characteristics.

What solutions are the stakeholders considering?

The solution NETICS is considering is a reinforced wall to cope with the tension forces, built with a mechanic press, using basic design formula for concrete/brick gravity walls as bank structure. Mister Verhagen told me to focus on differential settlement and the ''messing-groeven" solution. Jarit told me to make an embedded stepped design to solve the problem of the GEOWALL. There was one common fault everyone made. The assumption of the problem you will have with a vertical composed of CSEBs.

Step 3: Research type

A practice-orientated research has globally the following steps:

1. Problem
2. Diagnosis
3. Design
4. Change

5. Evaluation

The object of the thesis covers the first three steps. As stated before it is quite unclear what the actual problems are in larger scale GEOWALLs. The problem of a larger scale GEOWALL should therefore first be determined. The problem is followed by a diagnosis. The diagnosis focusses on the background and the causes of the identified problems. It also includes a course of action that needs to be taken in order to find a solution. Only when the diagnosis is clear, a design can be introduced that copes with the actual problems.

Step 4: Research objectives

The first objective is an assessment of the critical failure scenarios of the GEOWALL by analysing the stability of the conceptual design of the GEOWALL on sand, clay and peat for heights between one and five metres. The second objective is an overview of the challenges that have to be encountered in larger GEOWALLs by analysing and quantifying how, when and where the concept design fails for the varying scenarios. The third and last objective is an overview of potential design improvements for the failing larger GEOWALLs by looking into solutions for similar problems in conventional retaining wall design and by introducing out-of-the-box ideas.

STEP-BY-STEP APPROACH FOR THE RESEARCH FRAMEWORK

Step 1: Characterise briefly the objective of the research project

1. Overview of the critical factors
2. Overview of the challenges
3. List of suitable design improvements

Step 2: Determine the object or objects of the research project

1. Analytical failure calculations
2. Larger scale GEOWALL
3. Failing larger scale GEOWALLs

Step 3: Establish the nature of the research perspective

1. Problem-analysing: Possible impact of the critical factors of failure
2. Diagnostic research: Determine a specific area, and within this area look for the possible causes of the problem. For this analytical instruments such as a SWOT analysis, gap analysis, etc. can be used.
3. Design-oriented research: Mostly carried out with a *design model*, think in this case of the story of Eldert with a PvE (safety, quality, material usage, construction method, maintenance, costs), how to select alternatives, analyse data, think of alternatives (MCA + preference), final design, Maintenance plan, Realisation.

Step 4: Determine the sources of the research perspective

1. Theory on the safety of a gravity type structure, theory on the calculations, preliminary research

Step 5: Make a schematic presentation of the research framework by using the principle of confrontation

The challenge is to find the challenges of the GEOWALL instead of a specific solution for a specific problem. In other words, it is not essential to know the optimal design in the case of a river bank only. You want to know the design challenges for the critical situations. Knowing these challenges, will result in a targeted implementation of design improvements.

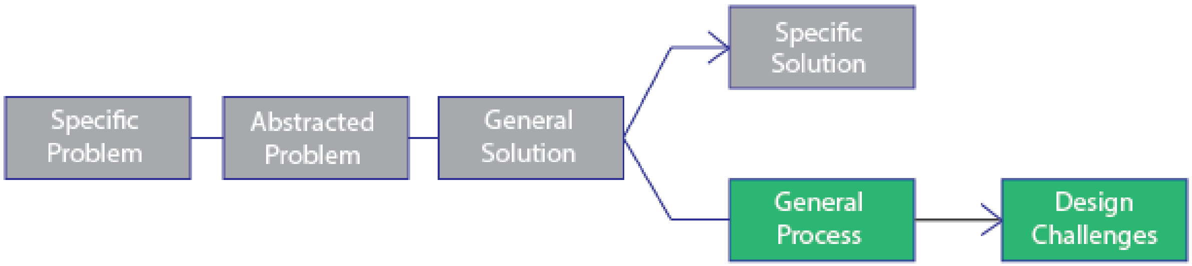


Figure A-1: Methodology

There are two main adjustments that can turn a design challenge of the GEOWALL into a proper modification: Adjusting the design and adjusting the strength. A design change is for example a foundation or a wider basis and a strength adjustment is for example reinforcement or different material. The focus in this thesis is the design, while NETICS is investigating the strength of the GEOWALL.

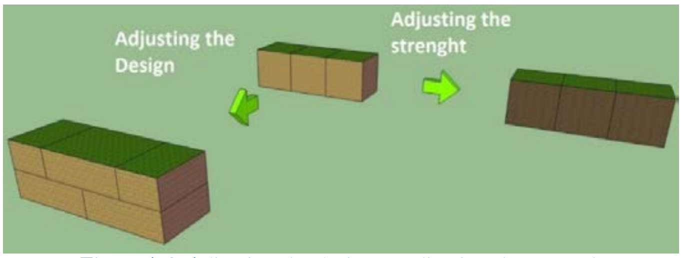


Figure A-2: Adjusting the design vs adjusting the strength

GEOWALL structure – construction type – gravity type structures:

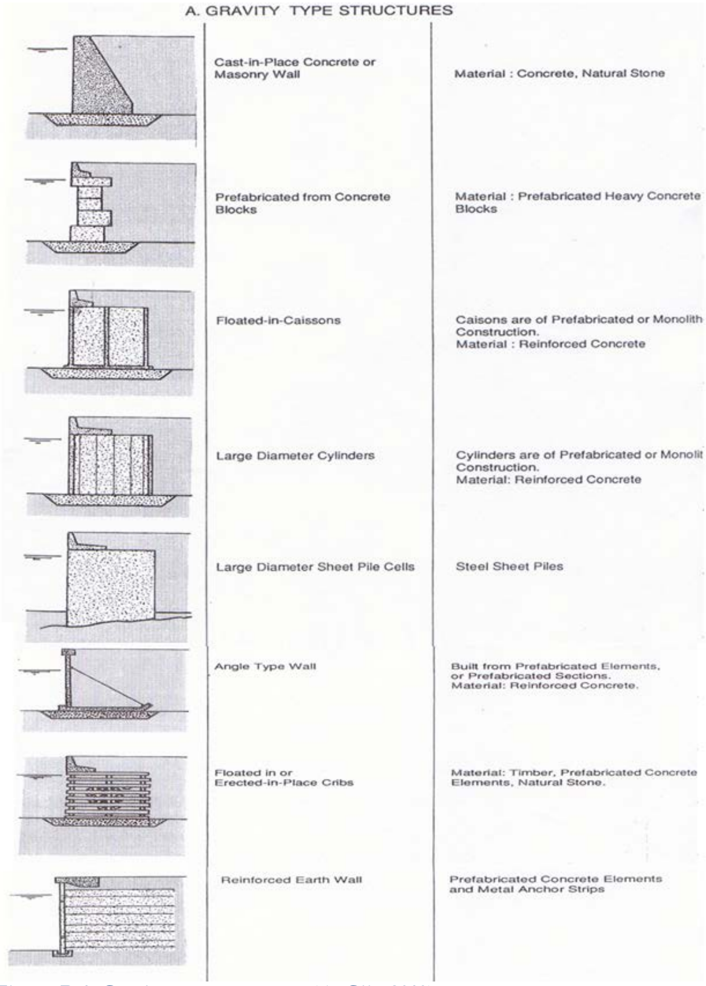


Figure B-1: Gravity type structures (de Gijt, 2010)

GEOWALL structure – construction material:

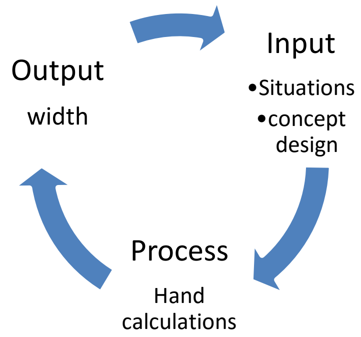
There are three sources that have useful information concerning the characteristics of the GEOWALL. The first a study performed by Maini (2010) which provides information on compressed stabilised earth masonry blocks. The data is obtained for blocks that are stabilized with 5 to 10% cement and that are compressed with a pressure of 2-4 MPa.

PROPERTIES	SYMBOL	UNIT	CLASS A	CLASS B
28 day dry compressive strength (+20% after 1 year)	σ d 28	MPa	5 -7	2 - 5
28 day wet compressive strength (after 24 hours immersion)	σ w 28	MPa	2 - 3	1 - 2
28 day dry tensile strength (on a core)	τ 28	MPa	1 - 2	0.5 - 1
28 day dry bending strength	β 28	MPa	1 - 2	0.5 - 1
28 day dry shear strength	S 28	MPa	1 - 2	0.5 - 1
Poisson's ratio	μ	-	0.15 - 0.35	0.35 - 0.50
Young's Modulus	E	MPa	700 - 1000	-
Apparent bulk density	γ	Kg/m^3	1900-2200	1700-2000
Coefficient of thermal expansion	-	mm/m°C	0.010-0.015	-
Swell after saturation (24 hours immersion)	-	mm/m	0.5 - 1	1 - 2
Shrinkage (due to natural air drying)	-	mm/m	0.2 - 1	1 - 2
Permeability		mm/sec	1.10^{-5}	-
Total water absorption	-	% weight	5 - 10	10 - 20

Figure B-2: Properties of a compressed stabilised earth masonry block according to Maini (2010)

Another source for the characteristics is the thesis of Bal and Van 't Wout (2014). This study suggest a dry GEOWALL weight of 16 $kN/m^3$and a wet GEOWALL weight is 19.34kN/m^3. The same research proposes a compressive strength of 5-7 MPa and tensile strength of 0.5-0.7 MPa. A third source is the website of geotechdata.info.

An initial technical analysis is carried out to identify the governing failure mechanism with respect to the concept design and to come up with a respectable width for the structure per height. The average width of the GEOWALL is therefore the only variable in this analysis and is iteratively found, such that the structure merely fails. In this iterative approach, no partial factors are included. The iterative process is schematized:



The input load is a constant water pressure and a constant soil pressure, thus waves, currents and ships are not taken into account. The concept design is used as lay-out and the soil set as sand. The geometrical data is shown in the table below.

Description	The surcharge	The height of the wall	The water level	Average width
Symbol	(q)	(H)	(h)	(b)
Unit	kN/m^2	m	m	m
Small waterways & Ponds	5	3	1.8	2.4
Cities	20	10	6	9
Lakes & Rivers	10	15	12	12.6
Ports	30	25	20	22.3

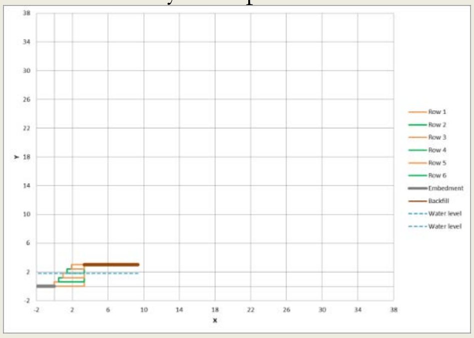
The input is processed by analytical calculations, which limits the analysis to only four failure mechanisms: sliding, overturning, structural strength wall and vertical bearing capacity.

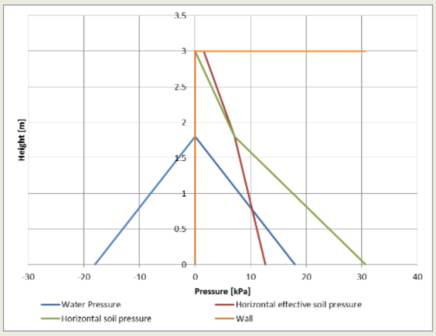
The output is the governing failure mechanism and the required width per situation. The horizontal bearing capacity (sliding) turned out to be the determining factor. This can be explained by a large buoyancy force in combination with the low specific weight of the GEOWALL resulting in a small downward force. Failure due to the structural strength of the wall and overturning were respectively found second and third in all situations. Nearly 50% of the required width is needed to counteract these two mechanisms. The failure mechanism for vertical bearing capacity was only reached for very small widths. This can also be explained by the small vertical downward forces in combination with a wide basis in the stepped design. The dominance of the failure mechanisms is as follows.

The required width to counteract the governing failure mechanism for all situations approximately 80% of the retained height. It has to be noted that no partial factors and no hydraulic head is included in the calculations. The width of the concept GEOWALL varies linearly with the height of the structure. The width of the GEOWALL concept design is the same as the height of the GEOWALL concept design. An overview of the analysis is given below.

Input	Process	Output

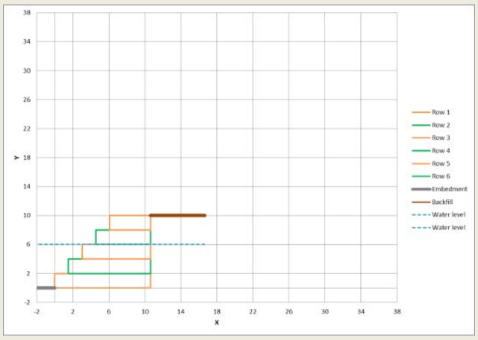
Small waterways and ponds

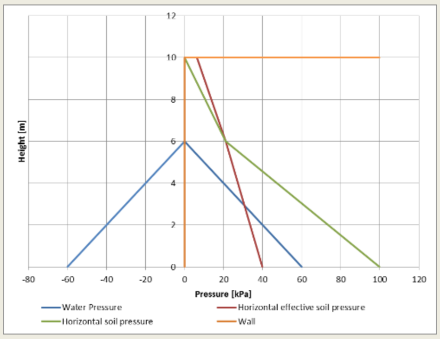




b = 2.2m	• Sliding
b = 1.4m	• structural strength wall
b = 1.3m	• Overturning
b = 0.4m	• Vertical bearing capacity

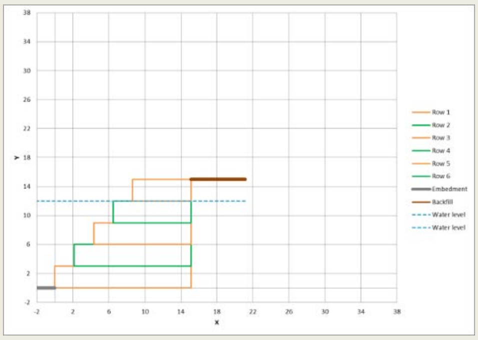
Urban areas

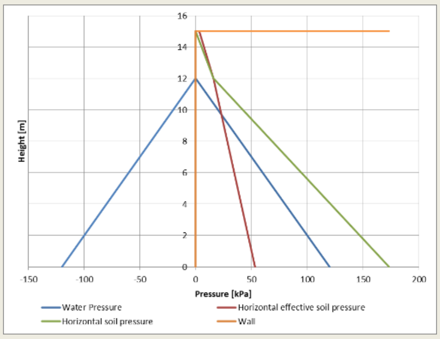




b = 7.6m	• Sliding
b = 4.8m	• structural strength wall
b = 4.6m	• Overturning
b = 2.6m	• Vertical bearing capacity

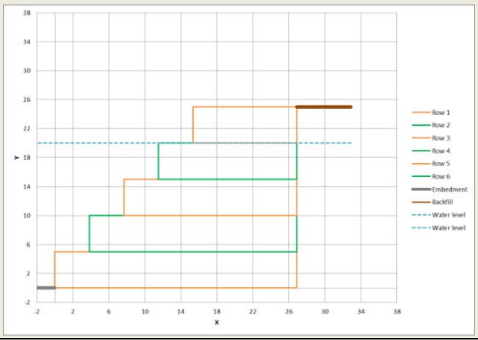
Rivers and lakes

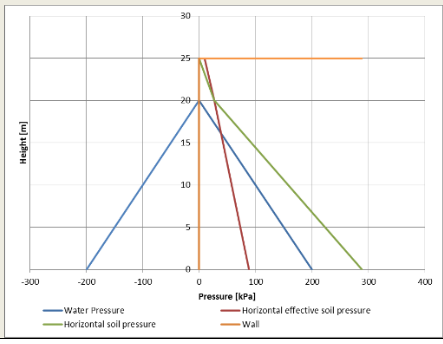




b = 10.5m	• Sliding
b = 6.5m	• structural strength wall
b = 6.7m	• Overturning
b = 4m	• Vertical bearing capacity

Ports





b = 19m	• Sliding
b = 11.6m	• structural strength wall
b = 11.4m	• Overturning
b = 9.4m	• Vertical bearing capacity

This appendix is subdivided in horizontal loading, vertical loading and loads

HORIZONTAL LOADING

There are several methods to make initial calculations for the horizontal soil pressure. According to Vrijling et al. (2011), the horizontal soil stress can be determined with the following formula:

$$\sigma_{soil,h} = \sigma'_h + p$$

The water pressure (p) and the effective horizontal soil pressure σ'_h should be considered separately. The magnitude for water pressure at a certain depth is the same in all directions:

$$p = \rho \cdot g \cdot h$$

The effective horizontal soil pressure has a regularly assumed relation with the vertical effective soil pressure:

$$\sigma'_h = K \cdot \sigma'_v$$

According to Vrijling et al. (2011) the vertical effective pressure (load) for a soil system with n dry layers and m wet layers can be determined:

$$\sigma'_v = \sum_{i=1}^{n} \gamma_{d,i} + \sum_{j=1}^{m} \gamma_{n,j} d_j - p$$

The K-value to calculate the horizontal effective soil pressure can be specified by means of three types of soil behavior. Jacky found a K-value for the vertical and horizontal effective soil stresses at rest, also called the 'neutral stress':

$$K_0 = 1 - \sin(\varphi)$$

Active soil stress occurs when the soil is less compacted than at rest, due to for example the sliding aside of the gravity structure away from the soil body. Passive soil stress occurs when the soil is compressed due to movement of a wall in the direction of the soil. The K-values for these two soil behaviours are:

$$K_a = \frac{cos^2(\varphi + \alpha)}{cos^2(\alpha)\left(1 + \sqrt{\dfrac{\sin(\varphi + \delta)\sin(\varphi - \beta)}{\cos(\alpha - \delta)\cos(\alpha + \beta)}}\right)^2}$$

$$K_p = \frac{cos^2(\varphi - \alpha)}{cos^2(\alpha)\left(1 - \sqrt{\dfrac{\sin(\varphi - \delta)\sin(\varphi + \beta)}{\cos(\alpha - \delta)\cos(\alpha + \beta)}}\right)^2}$$

The described assumptions result in a single K_a-value.

- Vertical wall, inclination $\alpha = 0°$
- Backfill slope angle $\beta = 0°$
- Angle of internal friction $\varphi = 27.5°$
- Wall friction $\delta \approx \frac{2}{3}\varphi = 18.3°$

$$\boldsymbol{K_a = 0.311}$$

This K_a-value in combination with the variable input values can be included in the described formulas, which will result in the horizontal loadings on the structure.

The surcharge is the load on top of the backfill from traffic, storage, temporary constructions, etc. In case of this surface load q, an additional horizontal load can be found against the wall (Vrijling et al., 2011):

$$Q_{h,q} = K_a \cdot q \cdot H$$

VERTICAL LOADING

The total vertical loading is the weight of the wall, reduced with a buoyancy force.

$$\sum W = W_{wall} - W_{buoy} + W_{step}$$

The weight of the wall is a determined by the sum of the dry and wet elements:

$$W_{dry} = \sum_{i=1} h_i b_i \gamma_d$$

$$W_{saturated} = \sum_{i=1} h_i b_i \gamma_s$$

The water is 'pushing' the structure in an upward direction, inducing the upward force W_{buoy}. This W_{buoy} assumes a rectangular structure with a uniform width which is not the case in the stepped preliminary design. The buoyancy force is therefore reduced with the term W_{step} to counteract for this difference.

There are two water levels, namely the outside water level and the ground water level. In the preliminary design, these two water levels are assumed to have the same height. The water levels per situation are based on example designs in recent CUR-publications (de Gijt, 2013), (de Gijt & Broeken, 2013).

BRINCH HANSEN

$$p'_{max} = c'N_c s_c i_c + q'N_q s_q i_q + 0.5\gamma' B \cdot N_\gamma s_\gamma i_\gamma$$

In this formula the coefficients i_c and i_q the inclination factors which correct for the angle in the direction of the vertical load. The coefficients s_c and s_q are the shape factors which correct for the shape of the loaded surface.

LOADS

The mooring forces for a preliminary design are given by Vrijling et al. (2011).

Type of ship	Mooring forces per bolder
Seagoing vessels	100 - 2000 kN
Inland Barges	140 – 280 kN
Yachts	55 kN

Collisions can be caused by either ships or ice. In case of both, are ships determining. The **Berthing** of a **ship** and the collision of a ship against a structure are theoretically the same. The collision force will influence the equilibrium of the structure as well as the structural strength of the wall.

Waves can influence the structure by means of low water levels and large incidental forces. The low waters are important for the equilibrium of the structure and are explained in head. The incidental wave forces are important for the structural strength of the wall. The **Tide** is a combination of water level differences and currents. Water level differences are considered in head and currents are explained separately.

Currents cause erosion what will result of a strength reduction of the structure. **Animals** may dig holes through the structure and is also considered to be a strength reducing load.

The change in **temperature** of an unobstructed object results in a linear increase or decrease of the structure:

$$\frac{\Delta l}{l} = \varepsilon = \alpha \Delta T$$

Material	α
Concrete	1.0E-5
Steel	1.2E-5
Ice	5.5E-5

Water in the construction material will have additionally effects. Freezing and melting of the structure will influence the pressure inside the pores of the saturated part of the construction. High temperatures may also have effects due to the water that evaporates out of the structure. This causes a change in volume and weight.

Influences of loads in relation to the failure modes:

	Equilibrium	Structural	Geotechnical
Material Properties	w	X	X
Geometrical data	X	X	X
Loads	Surcharge Head Self-weight Mooring Waves (low water)	Impact Mooring Erosion Uneven settlement Animals Wet/Dry Shrinkage Temperature Currents Waves (impact) Iceload	Surcharge Head Self-weight

Delft has currently 500 km of quay walls, banks and other walls. The municipality of Delft has 500 km of quay walls, embankments and walls. For the period of 2012 to 2015 Delft has the following lengths of vertical bank protections:

Type of wall in Delft	Height (m)	Material	Length (m^1)
Embankments (slopes)	N/A	N/A	3,900
Wooden Revetments	0-0.3	Wood	119,900
Sheet piling	0-3.0	Steel/Concrete/Wood	10,500
Quay walls	0-3.0	Stone + steel/concrete/wood	18,800
Environmentally friendly embankments	N/A	N/A	107,400
Gabion wall - water	N/A	Gabions	300

Type kade gemeente Delft	Lengte (m^1)
Beschermd Talud	1.200
Beschoeiing	103.900
Damwand	3.400
Geluidsscherm	3.200
Kademuur	14.300
Natuurvriendelijke oever	90.300
Schanskorf – land	1.100
Schanskorf – oever	300
Overig/ onbekend	32.100
Totaal	**249.800**

Kade type	Lengte (m^1)
Beschermd talud	100
Beschoeiing	13.00
Damwand	300
Geluidsscherm	600
Kademuur	4.500
Natuurvriendelijke oever	3.100
Totaal	**21.500**

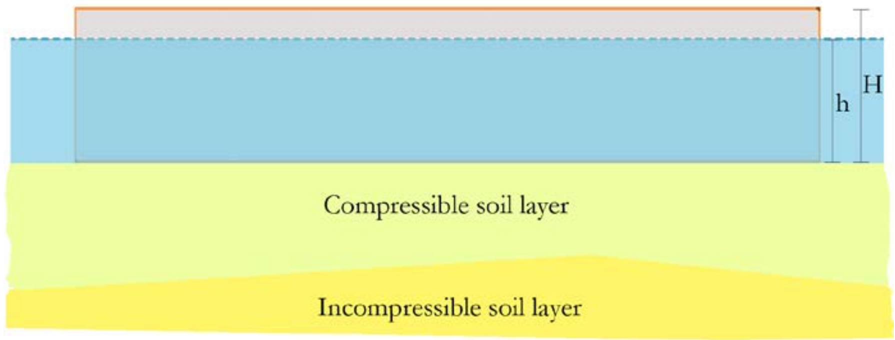


Figure F-1: Sketch geometry longitudinal direction

The goal of the primary analysis in the longitudinal direction is to check the stresses as a result of differential settlement. Differential settlement occurs in the situation where a weak soil layer such as clay or peat settles in a different rate over the length of a structure. This is mainly provoked due to a variation in the height of the compressible layer (see Figure F-1). The analysis will investigate the settlement of three soil types (sand/clay/peat) on top of a strong sandy soil layer which is known to be nearly incompressible.

The input for the analysis requires a depth of the compressible soil layer as well as a certain inclination of the plane between two soil layers. These two geometrical constants for the longitudinal analysis are found by investigating the soundings of several locations in the Netherlands. A sounding is the output of a cone penetration test, which is according to Meigh (2013) a method to evaluate the geotechnical engineering parameters of the soils to assess bearing capacity and settlement. A cone penetration test returns the local side friction (f_s) and the cone resistance(q_c). These ratio between the two values give away the identity of a soil layer. The Plaxis 2D reference manual (PLAXIS, 2015) provides the following overview to retrieve the soil type from cone penetration tests:

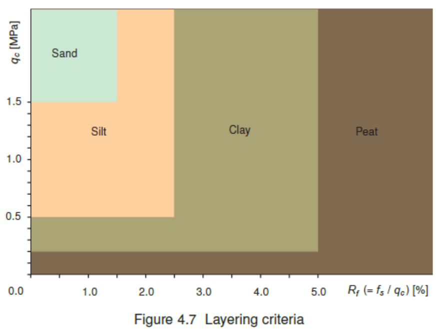


Figure F-2: Layering criteria according to PLAXIS (2015)

Several soundings at multiple locations are investigated of which one is chosen as best representable for this thesis. The location is situated near The Donk in the Netherlands. The compressible layer at this location has the largest variation in the height over the length and is therefore expected to return the highest tensile forces in the structure. At the location six soundings are found over the length of an embankment next to a 40 metre wide waterway. These six soundings hold information on the soil layers to a depth of 20 metres:

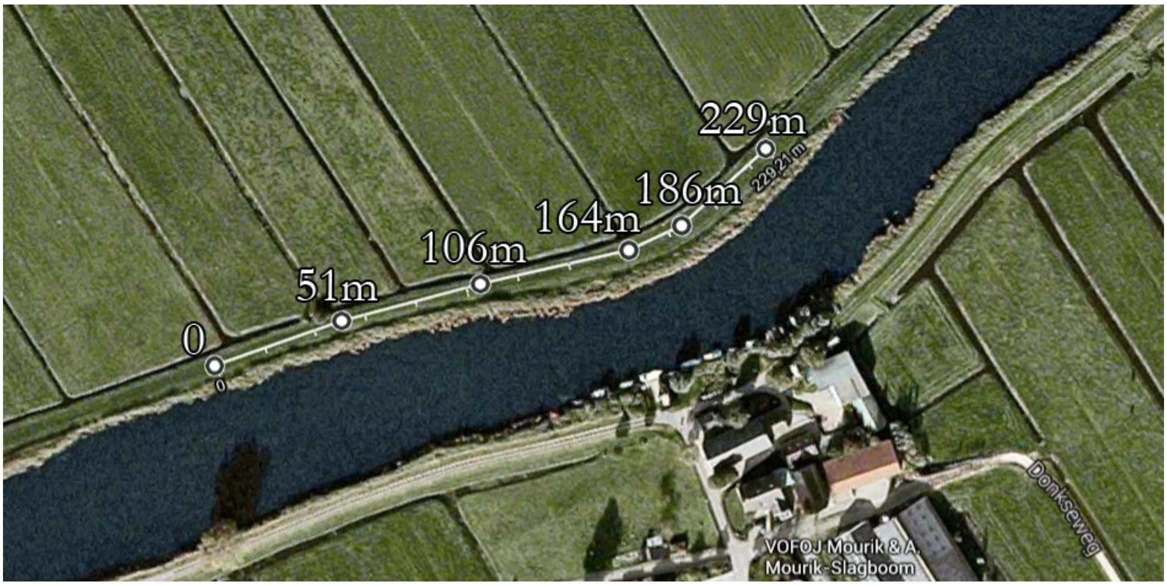
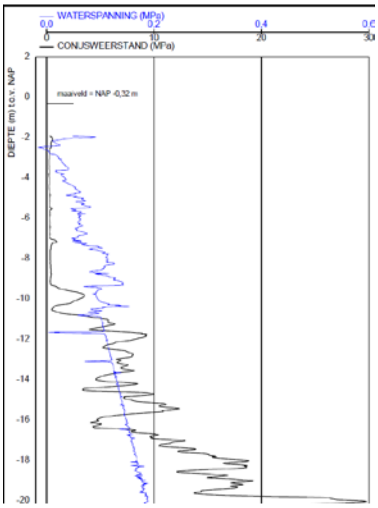
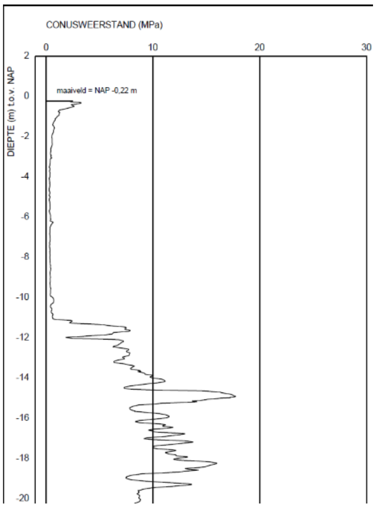


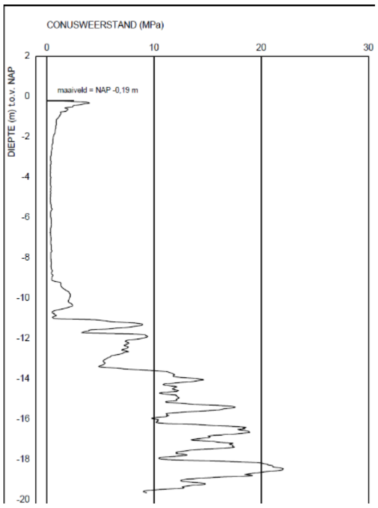
Figure F-3: The soundings and their locations near The Donk in the Netherlands



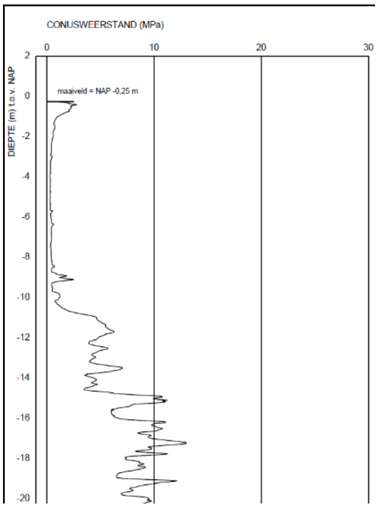
S38D01613 at $x = 0$



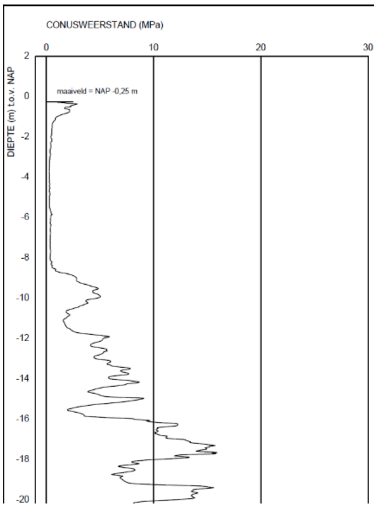
S38D01642 at $x = 51m$



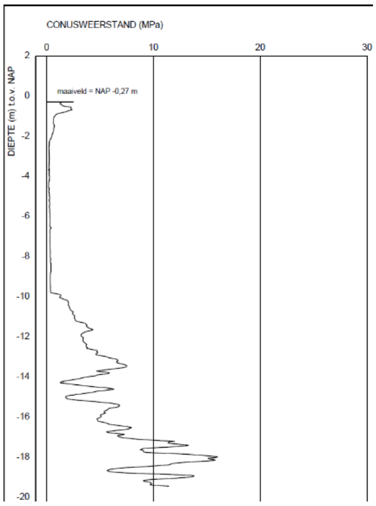
S38D01614 at $x = 106m$



S38D01643 at $x = 164m$



S38D01622 at $x = 186m$

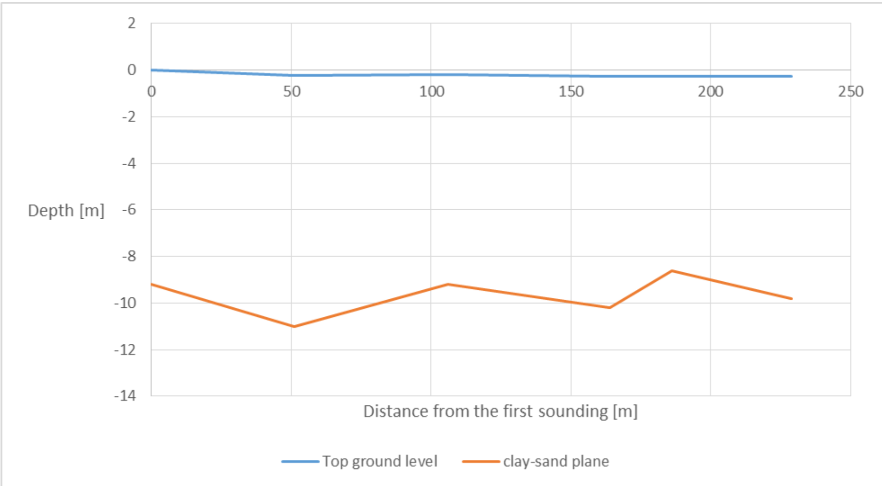


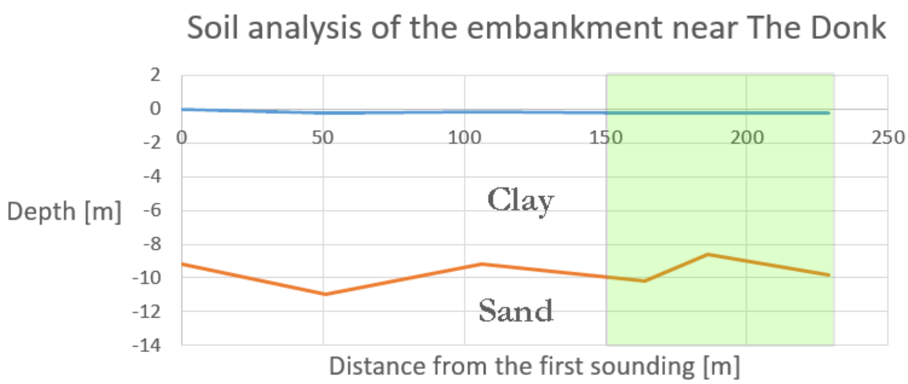
S38D01623 at $x = 229m$

All soundings show a clear separation between an upper clay layer and a lower sand layer. The level of this clay-sand plane is given in the table below for all six soundings.

Sounding ID		S38D01613	S38D01642	S38D01614	S38D01643	S38D01622	S38D01623
Distance (x) from the first sounding	[m]	0	51	106	164	186	229
Top ground level	[m]	0	-0.22	-0.19	-0.25	-0.25	-0.27
Clay-Sand plane	[m]	-9.2	-11	-9.2	-10.2	-8.6	-9.8

This consequentially results in the following graph with the top ground level and the clay-sand plane:





GENERAL INFORMATION PLAXIS2D

Plaxis 2D is a two-dimensional finite element program used to perform deformation and stability analysis for geotechnical applications. There is a clear separation between the cross section calculations and the longitudinal calculations. In the cross section calculations is the GEOWALL assumed to be uniform over the length, along the waterfront. In addition, displacements and strains are predominantly investigated in the longitudinal direction. Modelling the GEOWALL is therefore simplified to two 2D model planes, in the cross section and in the longitudinal section, instead of one 3D model.

There are several reasons for choosing Plaxis 2D as modelling program in this thesis. The main reason is the possibility to model the stresses in the construction as well as in the soil. This information can contribute to potential design improvements. It also enables it to find the critical failure mechanism, internally and globally, with one modelling program. Where LEMs only provide information about the global critical failure modes, Plaxis is able to evaluate both. The program can also be used to investigate more detailed designs, thus making it possible to investigate the impact of different design improvements. A final advantage is the option of performing a sensitivity analysis.

SAFETY CALCULATIONS

(Brinkgreve & Post, 2013) states that a SLS and ULS analysis using the same model is efficient and beneficial in the design process. The paper also explains how to include ULS and SLS in Plaxis. A typical work flow without any partial factors is a typical SLS calculation. After a successful SLS calculation, partial factors may be applied on loads and materials in the model. Partial factors on the soil are harder to define, since the soil is an acting load against the wall as well as a resistance force under the structure. Eurocode 7 (NEN-EN1997-1, 2004) & (NEN-EN1997-2, 2006) allows for partial factors on 'Action effects', which can be interpreted as the resulting structural forces (Brinkgreve & Post, 2013). In this way, it is possible to use partial factors according to the different design approaches using FEM.

Plaxis 2D is also able to provides the factor of safety for a given design. The factor of safety can be found by means of the phi-c reduction procedure. The shear strength parameters tan phi and c of the soil as well as the tensile strength are successively reduced until a failure mode has developed. The strength of the interfaces is reduced in the same way.

$$\sum Msf = \frac{tan_{\varphi input}}{tan_{\varphi reduced}} = \frac{c_{input}}{c_{reduced}} = \frac{S_{u,input}}{S_{u,reduced}} = \frac{Tensile\ strength_{input}}{Tensile\ strength_{reduced}}$$

The strength parameters are successfully reduced by a selected amount of steps. The development of $\sum Msf$ is then used to determine whether a failure mechanism has fully developed. A failure mechanism is apparent when $\sum Msf$ reaches a constant value. The factor of safety is then given by:

$$SF = \frac{available\ strength}{strenth\ at\ failure} = value\ of\ \sum Msf\ at\ failure$$

The safety calculations stops when a failure mechanism has occurred, therefore only the first occurring failure mechanism will be found.

Schweiger (2005)explains two methods to arrive at the factor of safety in Plaxis:

Method 1: An analysis is performed with unfactored parameters modelling all construction stages required. The results represent the behaviour for working load conditions at the defined construction steps. This analysis is followed by an automatic reduction of strength parameters of the soil until equilibrium can be no longer achieved in the calculation. The procedure can be invoked in any construction step. This approach is commonly referred to as phi/c-reduction technique.

Method 2: The analysis is performed with factored parameters from the outset. The factor of safety is obtained from the calculation where equilibrium could not be achieved. It is worth noting that in this approach the calculation for the SLS has to be performed in an additional analysis.

STRESSES

The displacement and stresses are found in the SLS calculation. There are four components for the stresses: Sxx, Syy, Sxy, Syx. It is possible to use a scalar field representation in which you represent all four components. It is also possible to compute the principle stresses. The principal stresses are rotated into a coordinate system where the shear stresses are zero and only the main 'principal' stress components remain. The two stress components give values for the occurring stresses and provide insight in the flow of the forces through the structure.

LOADS AND INTERFACES

Interfaces are joint elements to be added for a proper modelling of the soil-structure interaction. Interfaces are be created between the GEOWALL and the surrounding soil and are used to model the interaction between the two. The roughness of the interaction is modelled by choosing a suitable value for the strength reduction factor, R_{inter} (PLAXIS, 2015). The same literature explains that in the absence of detailed information it may be assumed that R_{inter} is of the order of 0.67. This is supported by Vrijling et al. (2011), stating that the angle of wall friction is assumed to be 2/3 of the angle of internal friction. The soil-GEOWALL interface is therefore assumed to have a strength reduction factor of 0.67.

STAGED CONSTUCTION

Plaxis2D considers several construction stages. These stages can be compared with real time construction stages. The first step is a K0 procedure, which is a special calculation method available in PLAXIS to define the initial stresses in the model, taking into account the loading history of the soil (PLAXIS, 2015). For the Mohr-Coulomb model, the default K0-value is based on Jaky's formula:

$$K_0 = 1 - \sin(\varphi)$$

This initial phase requires horizontal uniform soil layers. The GEOWALL model defines two layers: Soil layer 1 and Soil layer 2. The soil layers can have one of the three specified soil types, namely sand, clay or peat. The second phase represents a standard situations with a soil slope between water and land. In the third phase is the GEOWALL introduced at the toe of the slope. The fourth phase is used to fill the gap between the wall and the upper soil layer. The working load is applied in phase five, thus this phase is used for the SLS displacements and stresses. The SLS phase if followed by the ULS phase with partial factors over the load and the materials.

REFERENCE CASE

Rabie (2014)has modelled a hybrid Mechanically Stabilised Earth (MSE)/Soil Nail wall using a finite element model and a limit equilibrium model. A hybrid MSE/Soil Nail wall is a vertical wall of stabilised soil. The soil is stabilised by placing tensile reinforcing elements (inclusions) in the soil. The paper of Rabie concluded that traditional limit equilibrium approaches cannot be used alone for the design of such walls, it shall be supported by numerical methods for estimation of the global factor of safety and failure surface. The potential failure surfaces of the model can be identified by the density of the total displacement contour lines.

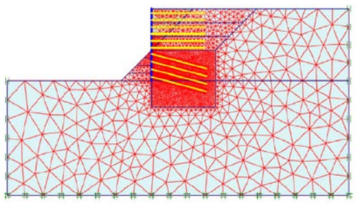


Figure G-1: Plaxis2D® set up of a MSE wall (Rabie, 2014)

In the initial phase, the initial stresses and pore water pressures of the GEOWALL under normal working conditions are calculated using Gravity loading. For this situation the water pressure distribution is calculated using a steady-state groundwater flow calculation.

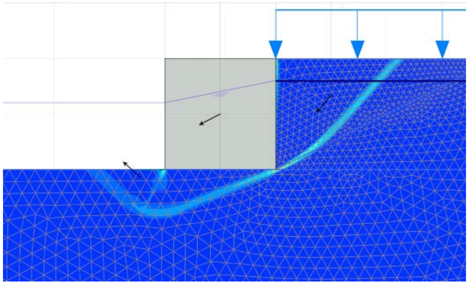
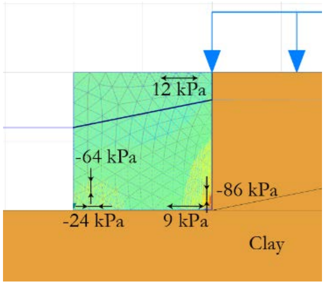
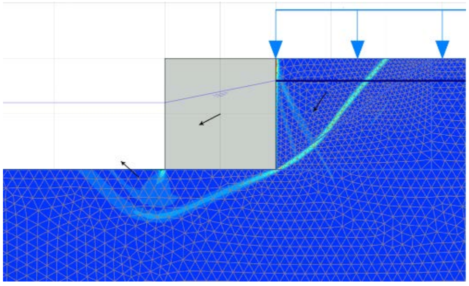
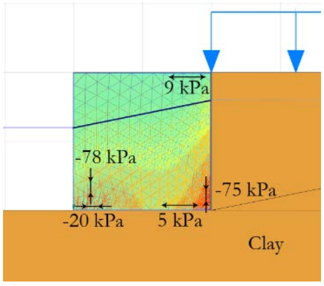
PARTIAL FACTORS

Partial load factors				
		EQU	STR/GEO*	STR/GEO**
Permanent	Unfavourable	1.1	1.35	1.0
	Favourable	0.9	1.0	1.0
Variable	Unfavourable	1.5	1.50	1.30
	Favourable	0	0	0

*Applied to structural actions **Applied to geotechnical actions

Partial material factors			
Parameter		Symbol	Value
Shearing resistance	tan(φ)'	γ_{ϕ}	1.25
Effective cohesion	c'	$\gamma_{c'}$	1.25
Undrained shear strength	Cu	γ_{cu}	1.4
Unconfined compressive strength	Qu	γ_{qu}	1.4
Weight density	y	γ_{γ}	1.0

Results from analysis II for a four and five metre retaining wall on clay in the cross-section.

Retaining Height	Overall stability [slip circle at failure]	Horizontal and vertical internal effective stresses [kPa]	Max / Min principal effective stresses σ'_3 (tensile)	σ'_1 (compr.)
			70	**-3000**
4m	 SF = 1.4		58.54	-158.2
5m	 SF = 1.2		52.36	-110.1

BACKGROUND INFORMATION SAFETY ANALYSIS

1. What is the relation between a lighter CSEB block and the safety against sliding and overturning for larger GEOWALLs® (h=1-5) on a sandy and soil in the cross-section?
2. What is the relation between the tensile and compressive strength of a CSEB block and the safety against structural failure for larger GEOWALLs® (h=1-5) on a sandy soil in the cross-section and on a transition between a sandy and clay soil in longitudinal direction?
3. What is the relation between a lighter CSEB block and the safety against geotechnical failure for larger GEOWALLs® (h=1-5) on a clay soil in the cross-section?

EQUILIBRIUM FAILURE

The retained soil determines the lateral force pushing against the structure, while the soil beneath the structure is determining the resisting force by its friction with the structure. A larger GEOWALL will coincide with a larger horizontal thrust and at the same time a larger resisting force as the total weight of the GEOWALL increases. The forces that are considered in the calculations are the following:

Variable	Equilibrium	Parameter
Material Properties	Weight of the GEOWALL Lateral stresses Friction factor	$\sum V$ Q f
Geometrical data	Height of the GEOWALL	h
Loads/Resistances	Surcharge Head	Q

STRUCTURAL FAILURE

The unique advantage of the GEOWALL is the reuse of locally available soil. The soil which is taken out is compressed with additives to add the necessary strength. The compressed soil in the shape of blocks are subsequently returned to their original location, resulting in a minimal surplus of weight between the original situation without the GEOWALL and the new situation with a GEOWALL. It is expected that this small increase in weight and the coinciding small increase in vertical effective stress ($\Delta\sigma'_v$) will results in small values for the settlement (Δh) and with that small tensile stresses in the structure in the longitude direction.

In the cross section is the surface (width x height) dominant in this failure mechanism. A very small structure is more likely to fail than a wide structure.

Variable	Structural (cross-section)	Parameter	Structural (longitudinal)	Parameter
Material Properties	Tensile and compressive strength of the GEOWALL Lateral stresses	$\sigma_{t,GEOWALL}$ $\sigma_{c,GEOWALL}$ Q	Tensile and compressive strength of the GEOWALL Lateral stresses	$\sigma_{t,GEOWALL}$ $\sigma_{c,GEOWALL}$ Q
Geometrical data	Height of the GEOWALL	h	Surface of the GEOWALL	h x b
Loads/Resistances	Differential settlement	u	Surcharge Head	Q

GEOTECHNICAL FAILURE

Variable	Geotechnical	Parameter
Material Properties	Weight of the GEOWALL Strength of the soil	$\sum V$ φ/c
Geometrical data	Height of the GEOWALL	h
Loads/Resistances	Surcharge	Q

CONCLUSION

Variable	Equilibrium		Structural (cross-section)		Structural (longitudinal)		Geotechnical	
Material Properties	Weight of the GEOWALL Lateral stresses	$\sum V$ Qs	Tensile and compressive strength of the GEOWALL Lateral stresses	$\sigma_{t,GEOWA}$ $\sigma_{c,GEOWA}$ Q	Tensile and compressive strength of the GEOWALL Lateral stresses	$\sigma_{t,GEOWA}$ $\sigma_{c,GEOWA}$ Q	Weight of the GEOWALL Strength of the soil	$\sum V$ φ/c
Geometrical data	Height of the GEOWALL	h	Height of the GEOWALL	h	Surface of the GEOWALL	h x b	Height of the GEOWALL	h
Loads/Resistances	Surcharge Head	Q	Differential settlement	u	Surcharge Head	Q	Surcharge	Q

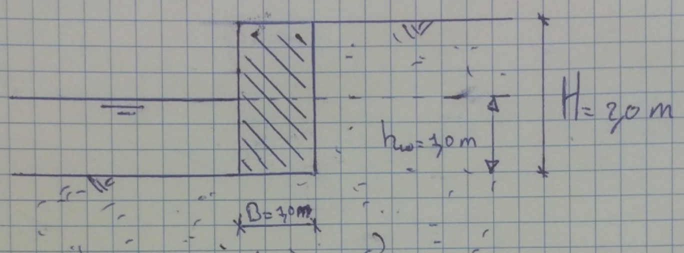
	Equilibrium	Structural	Geotechnical
Cross-section	Q & W	B & h	c/phi subsoil
Longitudinal section		u	

INFLUENCE OF DESIGN IMPROVEMENTS

Failure Mechanism Geotechnical improvement	Sliding	Overturning	Strength wall	Large Deformations	Vert. Bearing capacity	Overall stability	Piping
Wider	+	+	+/-	+/-	-	+	+
Embedded	++	+	-	+/-	+/-	+	+
Triangular	+/-	+	+/-	+/-	+	+/-	+/-
Stepped	+/-	+	-	-	+	+/-	+

Input

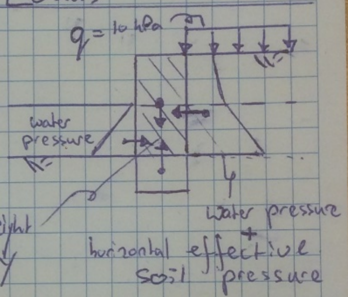
Geometrical Data



Material Properties

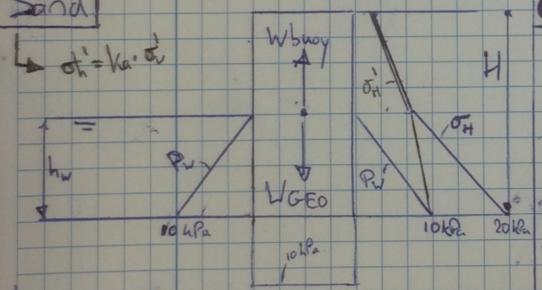
	γ_{sat}	γ_{dry}	c	φ
Sand	20	17	0	35°
Clay	14	14	8	22°

Loads



Sand

$\hookrightarrow \sigma'_h = K_a \cdot \sigma'_v$



Horizontal Force, Right =

$\frac{1}{2} \cdot P_{w,Right} \cdot h_w + q \cdot K_a \cdot H + \frac{1}{2} \cdot \sigma'_{h,GWL} \cdot (H - h_w) + \sigma'_{h,GWL} \cdot h_w + 0{,}5\,(\sigma'_{h,bottom} - \sigma'_{h,GWL}) = 18{,}68$ kN/m

Horizontal Force, Left =

$\frac{1}{2} \cdot P_{w,left} \cdot h_w = 5{,}0$ kN/m

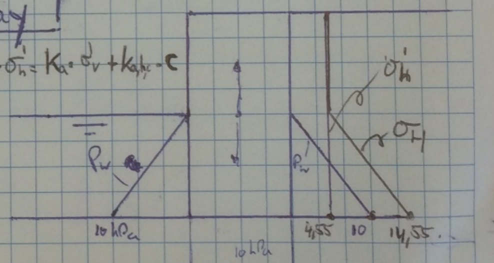
$\Sigma H = 13{,}68$ kN/m

in which

$K_a = \frac{1 + \sin\varphi}{1 - \sin\varphi} = \frac{1 + \sin(35)}{1 - \sin(35)} = 0{,}271$

Clay

$\hookrightarrow \sigma'_h = K_a \cdot \sigma'_v + K_{a,h,c} \cdot c$



Horizontal Force, Right =

$\frac{1}{2} \cdot P_{w,Right} \cdot h_w + q \cdot K_a \cdot H + \frac{1}{2} \cdot \sigma'_{h,GWL} \cdot (H - h_w) + \sigma'_{h,GWL} \cdot h_w + 0{,}5\,(\sigma'_{h,bottom} - \sigma'_{h,GWL}) = 14{,}10$ kN/m

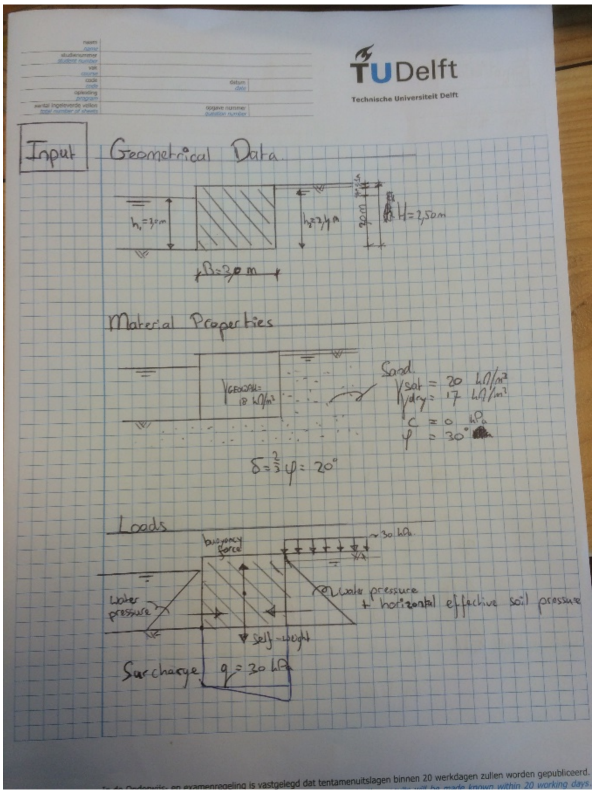
Horizontal Force, Left =

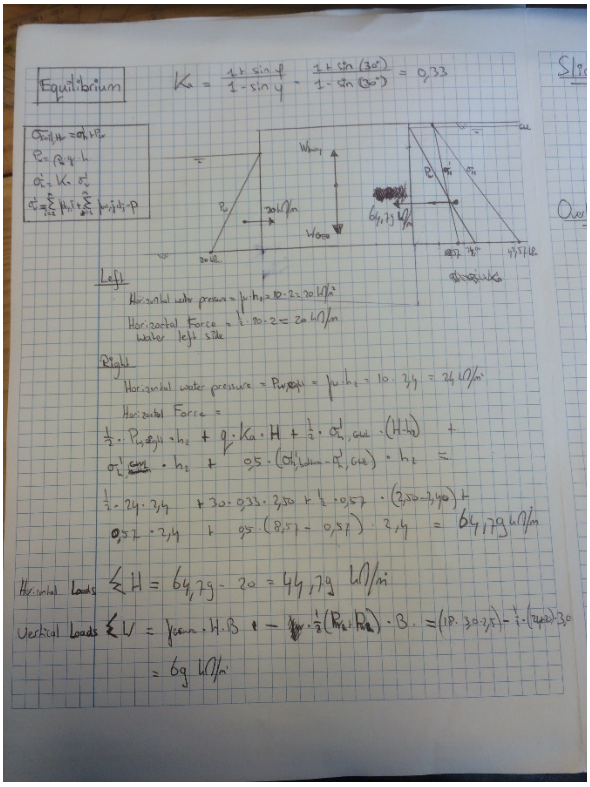
$\frac{1}{2} \cdot P_{w,left} \cdot h_w = 5{,}0$ kN/m

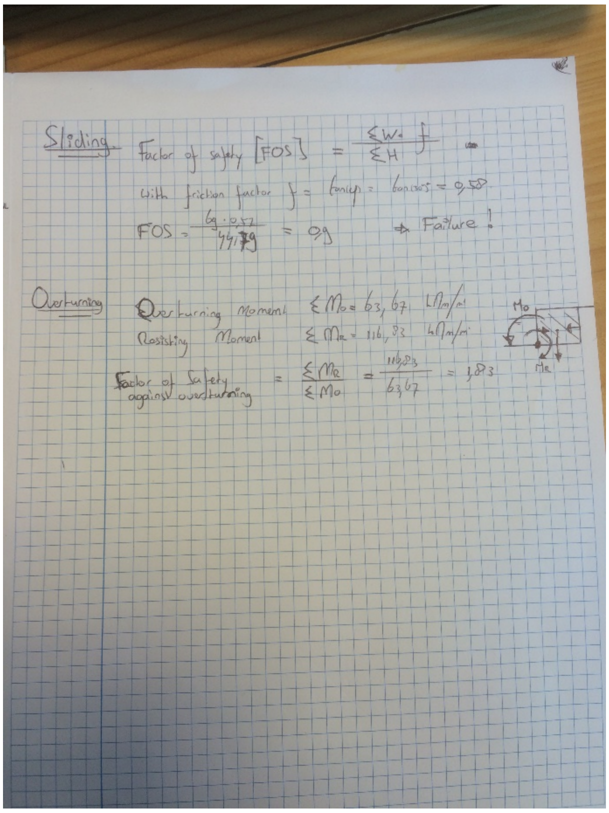
$\Sigma H = 9{,}10$ kN/m

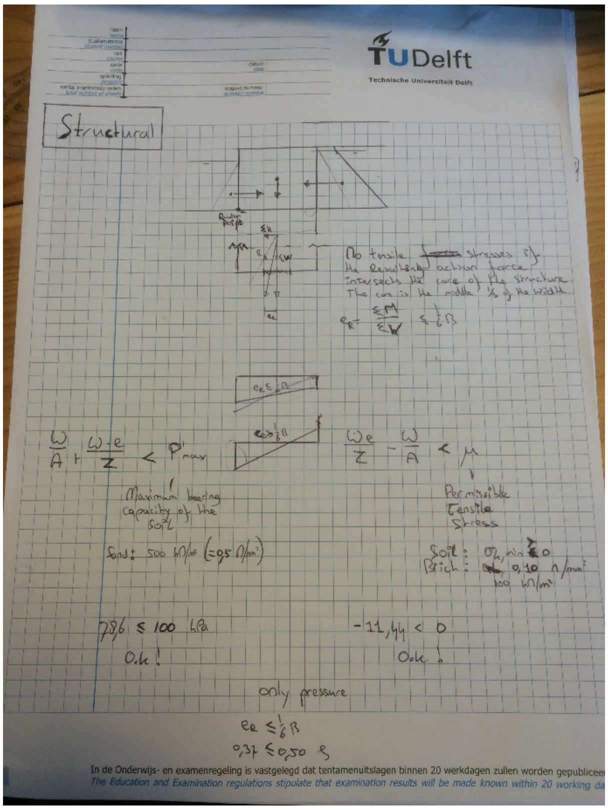
$K_a = \frac{1 + \sin\varphi}{1 - \sin\varphi} = 0{,}455$

$K_{a,h,c} = -\frac{2\cos(\varphi)\cos(\beta)(1 - \tan(\alpha)\tan(\beta))(\cos(\alpha - \delta))}{1 + \sin(\varphi + \delta - \alpha - \beta)} = -1{,}12$


Input
Geometrical Data
Material Properties
Sand
C = 0
φ = 30°
Loads
Water pressure
Self-weight
Surcharge q = 30 kPa


Equilibrium
= 0,33
Left
Right
Horizontal Loads
Vertical Loads
= 69 kN/m


Sliding
Factor of safety [FOS]
FOS
= 0,9
Failure !
Overturning
Overturning Moment
Resisting Moment
Factor of Safety against overturning
= 1,83


TU Delft
Technische Universiteit Delft
Structural
No tensile stresses if the resulting action force intersects the core of the structure. The core is the middle 1/3 of the width
Maximum bearing capacity of the soil
Permissible tensile stress
78,6 ≤ 100 kPa
O.k !
-11,44 < 0
O.k !
only pressure
0,37 ≤ 0,50
In de Onderwijs- en examenregeling is vastgelegd dat tentamenuitslagen binnen 20 werkdagen zullen worden gepubliceerd

CASE DELFT

INPUT

Soil properties

The following sounding test is found at dinoloket.nl for the designated location:

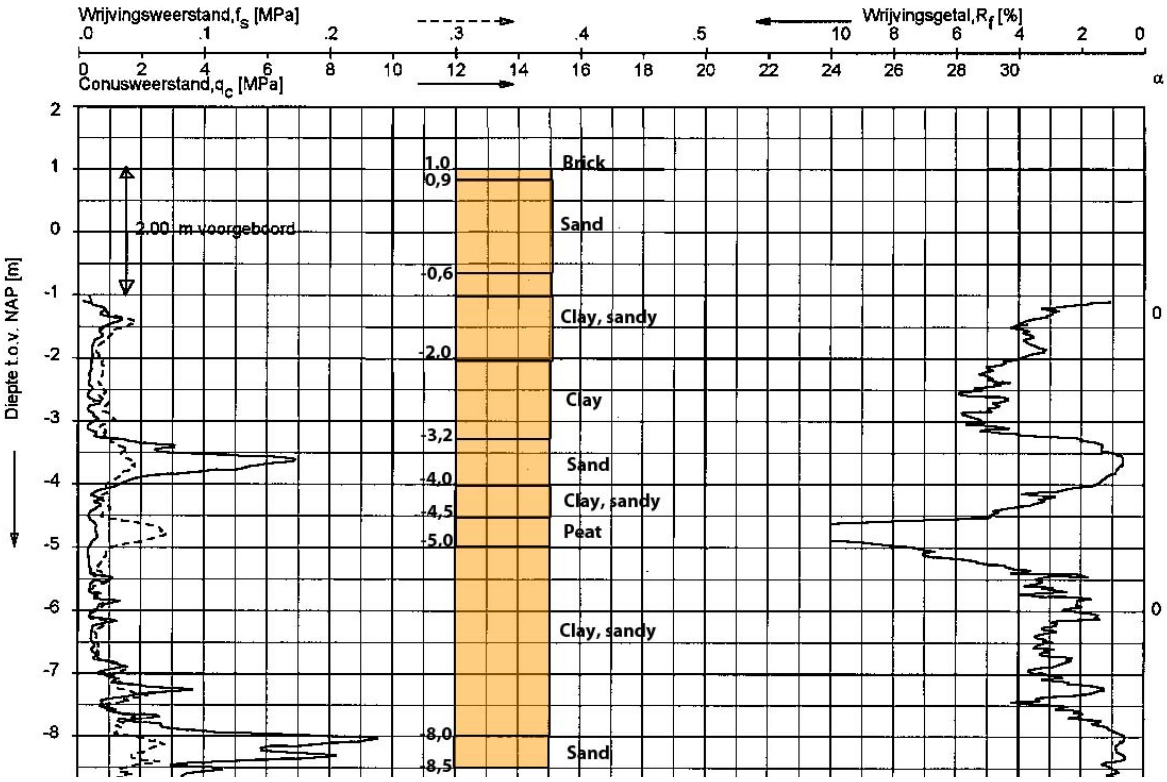


Figure J-1: Sounding test from dinoloket.nl

The following Geotechnical parameters are found:

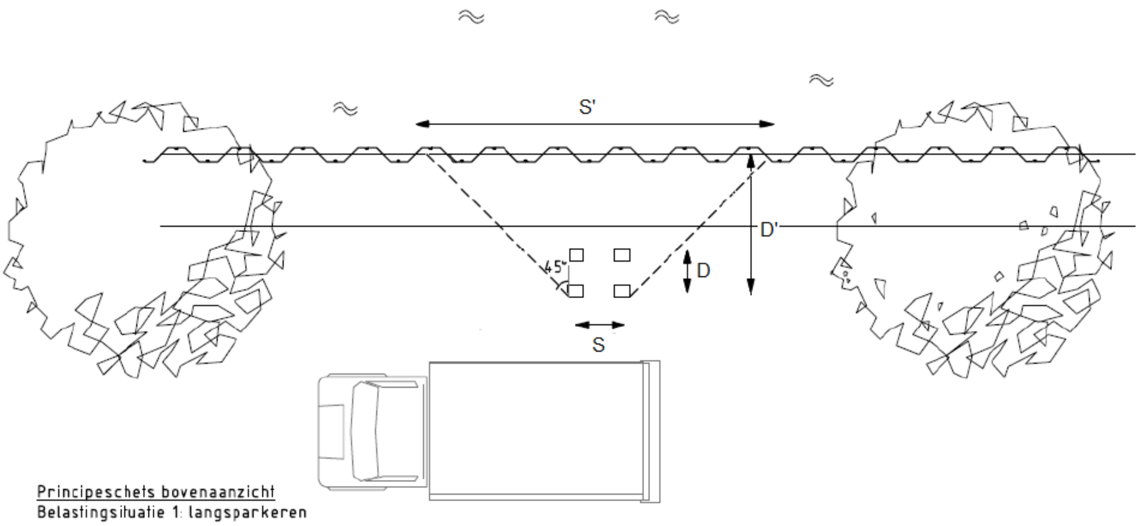
Ground	Top	Bottom	Yd	Yn	C’	Phi’	Delta’
Sand	+0.9	-0.6	17	20	0	30	20
Clay, Sandy	-0.6	-2.0	18	18	0	27.5	18.3
Clay	-2.0	-3.2	17	17	10	17.5	11.7
Sand	-3.2	-4.0	17	20	0	30	20
Clay, Sandy	-4.0	-4.5	18	18	0	27.5	18.3
Peat	-4.5	-5.0	13	13	5	15	10
Clay, Sandy	-5.0	-8.0	18	18	0	27.5	18.3
Sand	-8.0	-8.5	17	20	0	30	20

Surcharge due to traffic

The surcharge due to traffic can be calculated, according to de Gijt (2013), with the following formula:

$$q = \frac{4 \cdot F}{D} \cdot \left(\frac{S}{S + 2D'} \right)$$

This formula is used to determine the vertical uniform distributed load in a 2D calculation. A vehicle with a maximum weight of 4000kg will result in a vertical downward force of 10kN per wheel. The total vertical load in reality acts on a surface of D x S = 1.5m x 2.5m. This load is spread out over a larger area: D' x S', in which D' is the distance between the centre point of gravity and the wall of the canal. This distance is in this case 1.5 metres, resulting in an average surcharge of 15 kN/m2.

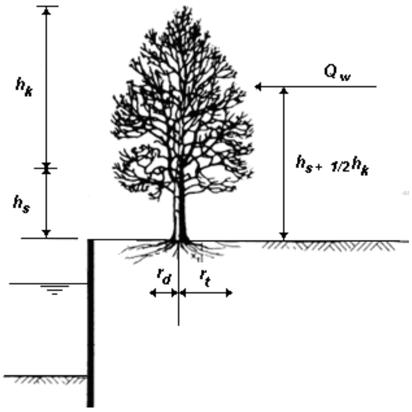


Water level difference

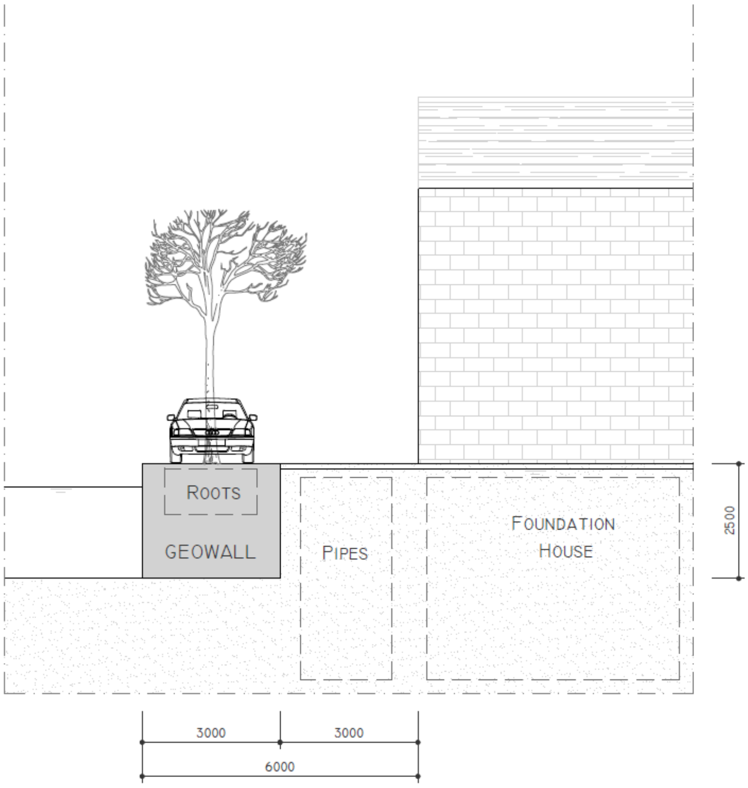
The maximum difference in water level is the lowest possible water level in the canal and the highest groundwater level. The highest groundwater level is the same as the top of the soil. In case of insufficient data, it is recommended by de Gijt (2013) to work with a water level difference of 0.5 metres.

Trees

Trees are commonly found near the side of a canal. The roots of these trees can have serious effects on the GEOWALL. de Gijt (2013) distinguishes four loads. The weight of the tree is seen as a permanent load and can be around 20 kN. The root system with a diameter of 3 metres transfers the force to an area of 7 m2. In a 2D cross-section this results in a surcharge of 2.9 kN/m2 over a width of 2.7 metres. The expansion of the roots towards the GEOWALL may cause cracks in the construction, however it is yet not possible to quantify this load. A fallen tree is seen as an accidental load. An excavation pit due to a falling tree results in a maximum depth of 1 to 1.5 metres. The wind load is variable is transferred through the trunk and the roots of the tree to the construction. This load is calculated in NEN-EN1997-1 (2004). The uniform distributed compression load for the area near the roots of the tree can be around 85 kN/m2 and the uniform distributed tensile load is around 18 kN/m2 near the roots of the tree.



CONCEPT DESIGN



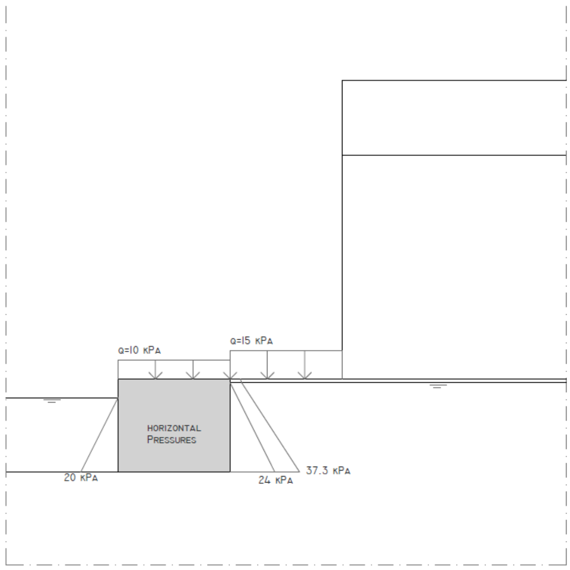
EQUILIBRIUM CALCULATIONS

Input analytical method:

Description	Symbol	Value	Unit
Backfill slope angle above wall	β	0.00	°
Angle of internal friction	Φ	27.50	°
Angle of wall friction	δ	18.33	°
Back of wall angle to horizontal	α	0.00	°
Cohesion	c	0.00	kN/m2
Surcharge	q	18.00	kN/m2
Wet soil density	γsat	18.00	kN/m3
Dry soil density	γdry	18.00	kN/m3
GEOWALL density	γg	18.00	kN/m3
Water density	ϱ	10.00	kN/m3
Actual height of wall	H	3.00	m
Water level outside	h	2.40	m
Ground water level	GWL	2.90	m
Permissible soil bearing capacity	qa	500.00	kN/m2

Process

The starting point is the standard design. Failure against sliding with a safety factor of 1.5 is governing and determines the width of the structure.



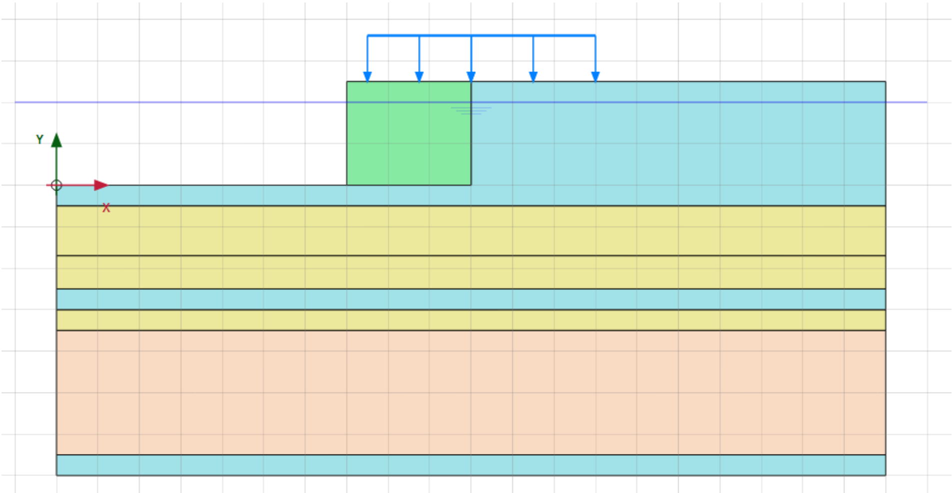
Surcharge and Horizontal soil stresses

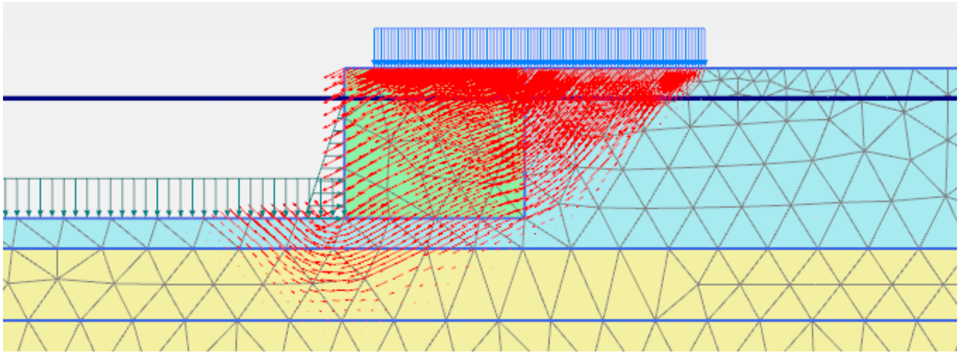
$$\sum H = 32.7\ kN/m$$
$$\sum W = 99.0\ kN/m$$

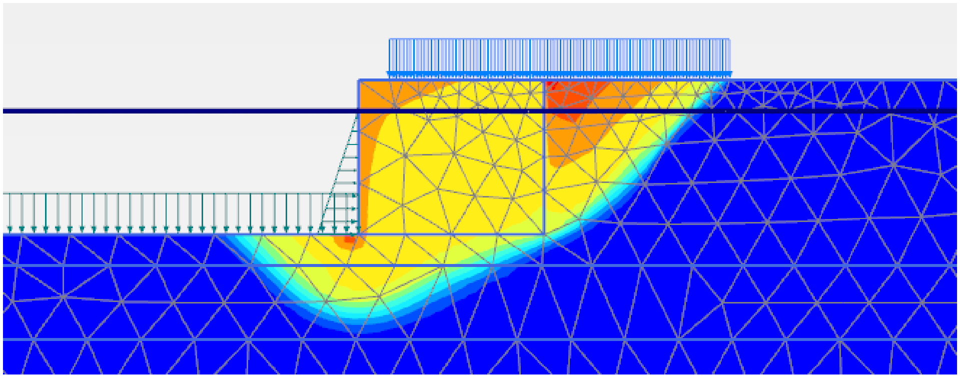
Horizontal bearing capacity (sliding)

$$\sum H < f \cdot \sum W$$

SF = 1.5

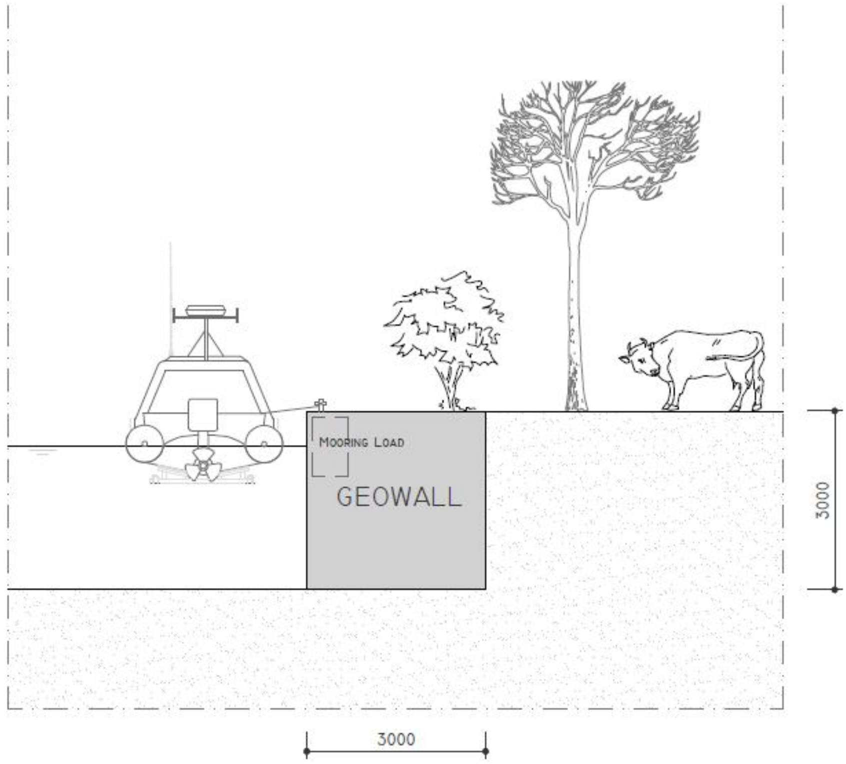






SF = 1.35

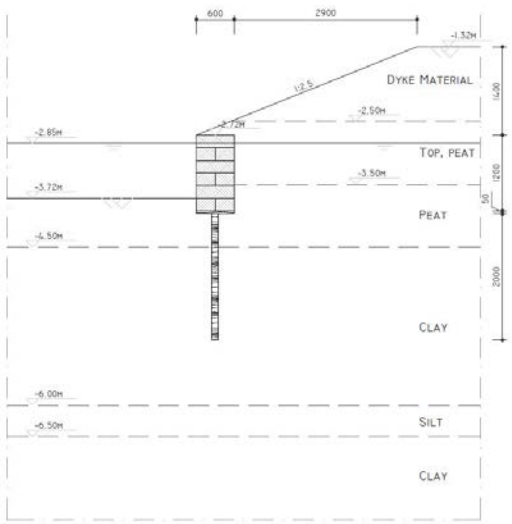
CASE - RURAL AREA



Safety against sliding = 1.5

Safety against overturning = 4.9

CASE BAM



	Top level	Unit weight	Friction angle	Cohesion
	[m, NAP]	[Knm-3]	[ϕ]	[kNm-2]
Dyke material	-0,3	11	17,5	2.0
Top, peat	-2,5	10,3	27,5	3.0
Peat	-3,5	10,3	27,5	1.7
Clay	-4,5	14	17,5	5.0
Silt	-6	16	30	0.0
Clay	-6,5	14	17,5	5.0