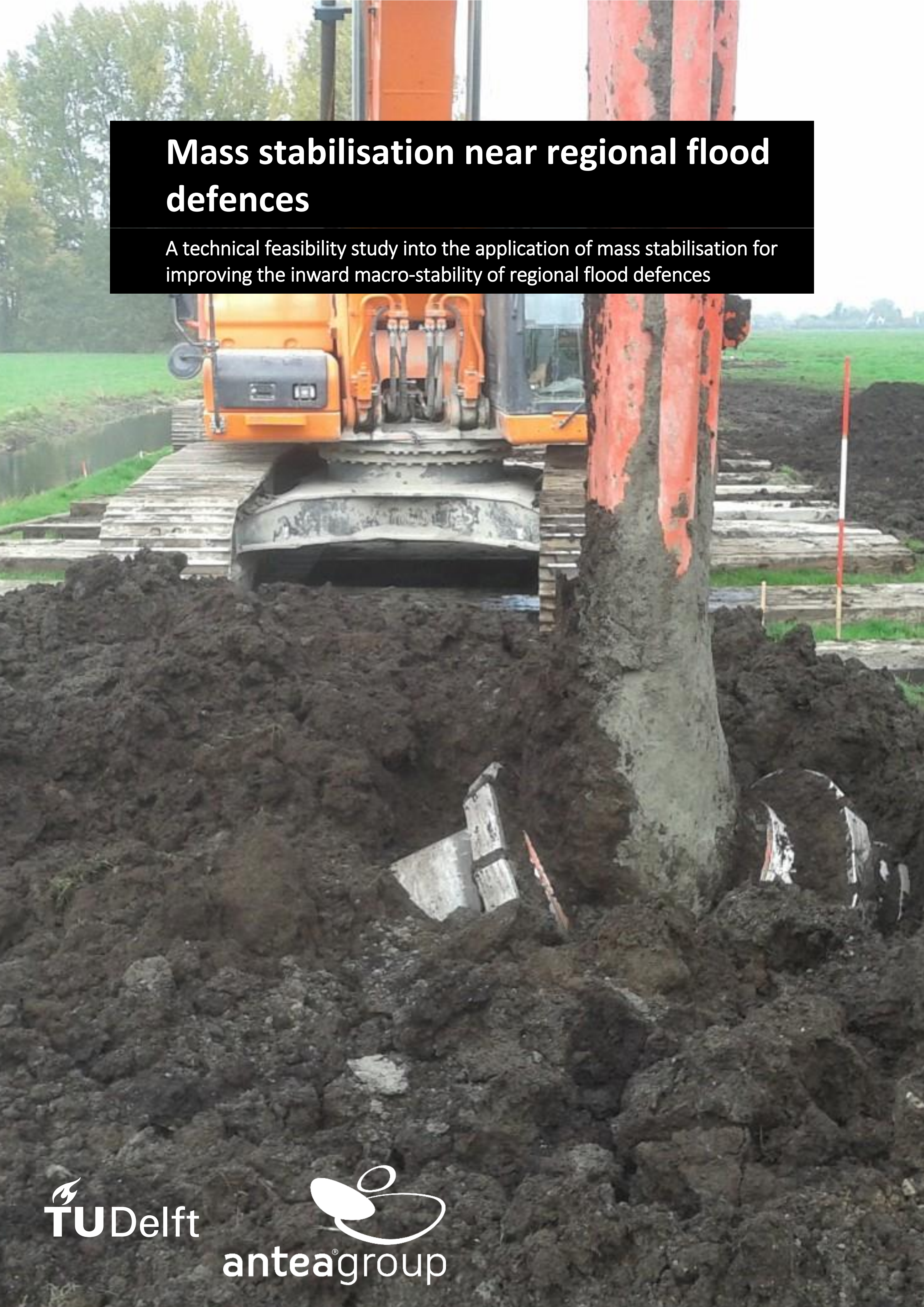
Mass stabilisation near regional flood defences

A technical feasibility study into the application of mass stabilisation for improving the inward macro-stability of regional flood defences



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Mass stabilisation near regional flood defences

A technical feasibility study into the application of mass stabilisation for improving the inward macro-stability of regional flood defences

By

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in partial fulfilment of the requirements for the degree of

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Preface

In front of you is my master's thesis for completing my master Geo-Engineering at Delft University of Technology. Over the course of last year I have conducted in-depth research into the technical feasibility of applying mass stabilisation for reinforcing levees. During this research, I had the honour of experiencing the many facets of the engineering profession, from consultant to laboratory technician and project manager. This was a very educational process, in which I gained a lot of experience. However, all of this was not possible without help.

First of all I would like to express my sincerest gratitude to all thesis committee members. All committee members were always there to give me support, feedback and encouragement throughout my research. I would especially like to thank Professor Cristina Jommi and Jan-Willem Bardoel for their tremendous support during my research, helping me and guiding me throughout my research.

I would also like to thank all colleagues from Antea Group who helped me with the tools, equipment, funding and assistance I needed to conduct this research. In particular I would like to express my gratitude to Pieter Jeroen Bart for always taking the time to help me come up with solutions to problems I ran into during my research. I also want to thank Corrie Bos for allowing me to put her photograph on the cover of my thesis.

This research would not have been possible without the help of the Municipality of Montfoort. I wish to express my gratitude to the Municipality of Montfoort, in particular to Johan Nijzink, for allowing me to sample the soil I needed for my laboratory research in the Ecopark in Linschoten.

Without the geotechnical laboratory of Fugro NL Land B.V. this research would not have been possible as well. I would like to thank Egon Koenders and Ibrahim (Ibro) Bajramovic for their assistance and help during most of the laboratory research, teaching me how to make stabilised soil samples and how to use the equipment in the laboratory. I would also like to thank Salvador Paz Noriega for our brain-storm sessions during which we discussed and explained my findings, as well as how to solve some of the problems I ran into during the laboratory research. Lastly, I wish to express my gratitude to Wouter van der Weijst for allowing me to carry out my laboratory research at the geotechnical laboratory of Fugro NL Land B.V.!

Many thanks also go out to Angelo Sarabèr from Vliegasunie B.V. and Nico Vonk from ORCEM B.V. for making the flue gas desulphurisation gypsum and the ground-granulated blast-furnace slag available for my laboratory research.

Last but not least I would like to thank my family, my mother and my sisters for their great support during this research. There were some difficult times and their support helped me pull through, especially in the final stages of the research.

N. Overbosch

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Abstract

In the Netherlands, alternative methods for reinforcing levees are still sought after as reinforcing levees is becoming more difficult. Limited space or the desire to maintain the landscape make it more difficult to reinforce levees by traditional elevation and extension of the levees (CUR-commissie C141, 2007). Along with the transportation of materials that is needed to reinforce the levee and the associated impact of that transportation (e.g. hindrance and CO_2-emissions), a desire for alternative reinforcement methods has been created. A possible reinforcement method that could still be applied under these conditions is mass stabilisation.

Mass stabilisation is a soil improvement technique recently introduced in the Netherlands (Dekker, 2015b), with which soft soil is mixed with a binder on site and in-place to create a homogeneous and stronger soil layer (Forsman, Jyrävä, Lahtinen, Niemelin, & Hyvönen, 2015). The technique has been developed in Finland in the early 1990s and has mostly been applied, both abroad and in the Netherlands, for road constructions (Forsman et al., 2015). However, mass stabilisation has never been applied for reinforcing levees before despite the advantages of the technique. It is presumed that the technique requires no extra space at the levee, does not affect the landscape, can be implemented quickly and requires little transportation of materials. Still, little research into this particular application of mass stabilisation has been carried out to date.

Given the benefits, the possible application of mass stabilisation at levees is explored in this study. The objective of this research is to determine the technical feasibility of applying mass stabilisation for improving the inward macro-stability of levees by stabilising strips of soil. In this study, only the application at Dutch regional flood defences is considered.

To demonstrate the technical feasibility, mass stabilisation must meet the following two criteria: mass stabilisation must be able to solve a stability deficit at levees and mass stabilisation must be practicable at levees. To help assess the technical feasibility, a literature study is carried out initially to collect background information on mass stabilisation and properties of stabilised soils. The gathered information has shown that laboratory research is essential for two reasons. First, similar soils at different sites have different physical and chemical properties and may require different binders or dosages to stabilise the soils, resulting in stabilised soils with vastly different properties (Building Research Establishment (BRE), 2002). Because of this, measurements of the properties of soils stabilised at a specific location cannot just be applied in projects elsewhere, which means that site-specific research is always required (Building Research Establishment (BRE), 2002). Secondly, little is known about the mobilisation of the effective strength parameters of stabilised soil. This information is required for assessing the stability of levees in compliance with Dutch safety standards (Stichting Toegepast Onderzoek Waterbeheer, 2015a). Because of both reasons, laboratory research has also been carried out in this study. In the laboratory, an additional criterion for the technical feasibility has been examined: the achievability of the desired effective strength parameters at specified strains in compliance with Dutch safety standards.

The first criterion, the ability of mass stabilisation to solve a stability deficit, is assessed by modelling theoretical reinforcements at two real Dutch levees (‘boezemkaden’) with stabilisations at three spots: toe, slope and crest. With these stabilisations increases in the Factor of Safety between 7% and 47% have been achieved, solving the stability deficit at both levees. However, the most effective spot for stabilisation to yield the biggest increases in the Factor of Safety has been shown to be case-dependent.

The second criterion, the achievability of the desired effective strength parameters, has been examined in the geotechnical laboratory of Fugro NL Land B.V. in Arnhem. First, a suitable binder recipe has been selected for the stabilisation of a peat and an organic clay sampled near one of the examined levees. For both examined stabilisations the required unconfined compressive strength has been achieved, whereas the required combinations of the effective strength parameters at specified strains in compliance with Dutch safety standards has not. This is the result of either a too low binder dosage or an improper selection of the consolidation stresses. Regardless, trial stability calculations have shown that the measured combinations of the effective strength parameters are still sufficiently high to reinforce the examined levee.

The third criterion, the practicability of mass stabilisation at levees, has been assessed by modelling the time-dependent execution for a continuous stabilisation at the toe of one of the examined levees with different assumptions of the unit weight and the initial strength of the stabilised soil. With help of the laboratory measurements and two-dimensional stability analyses with weighted averages of the soil properties and the preload, the execution of mass stabilisation, solely on the basis of strength, is found to be feasible.

Although on the basis of this research applying mass stabilisation at regional flood defences seems technically feasible, further studies will be required before a definitive conclusion on the technical feasibility can be drawn. It is therefore recommended to carry out additional analyses for the practicability, with particular attention to aspects like settlement and 3D-effects that have not been included in the analyses due to limitations in this study. In addition to this, it is advisable to do research into the relationship in the properties and the variability thereof between soil stabilised in the laboratory and in the field, because it is suspected that these can differ considerably.

Table of contents

List of figures

List of tables

List of acronyms

<u>Acronym</u>		<u>Description</u>
CDIT	-	abbreviation of *Coastal Development Institute of Technology*. This Institute is located in Tokyo, Japan.
CEM I (52,5 R)	-	CEM I is a Portland cement. The Portland cement used in this research was CEM I 52,5 R, which is a Portland cement with rapid initial hardening and an average 28-day unconfined compressive strength of 52,5 MPa.
CEM III/B (42,5 N-LH/HS)	-	CEM III/B is a blast-furnace slag cement. The blast-furnace slag cement used in this research was CEM III/B 42,5 N-LH/HS, which is a cement with normal initial hardening and an average 28-day unconfined compressive strength of 42,5 MPa.
CIUc triaxial tests	-	abbreviation of *isotropically consolidated undrained triaxial compression test*.
COV	-	abbreviation of *coefficient of variation*, which is the ratio of the standard deviation of a soil parameter to the mean of the same soil parameter.
DSS tests	-	abbreviation of *direct simple shear tests*.
FGD-gypsum	-	abbreviation of *Flue Gas Desulphurisation gypsum*. This gypsum is won from flue gases (fumes) from coal-fired power plants (Albarius, 2019).
FHADM	-	abbreviation of the *Federal Highway Administration Design Manual* that contains global information on deep mixing, an alternative stabilisation technique to mass stabilisation.
FoS	-	abbreviation of *Factor of Safety*, which is a factor describing how much stronger a system is compared to the strength it needs to withstand the expected loading of the system.
GGBS	-	abbreviation of *Ground-Granulated Blast-furnace Slag*. Ground-granulated blast-furnace slag is a residual product that is produced during the production of iron in a blast-furnace (Beton Lexicon, 2018).
HDSR	-	abbreviation of the Water Board *Hoogheemraadschap De Stichtse Rijnlanden*, which is responsible for most of the water management in the Province of Utrecht (the Netherlands).
HWC	-	abbreviation of *high water conditions*, which are the soil mechanical and geohydrological conditions at high water, such as the normative high water level and the height of the phreatic surface (Rijksoverheid, n.d. a).
NWC	-	abbreviation of *normal water conditions*, which are the soil mechanical and geohydrological conditions under everyday conditions.
UCS	-	abbreviation of *unconfined compressive strength*, which is the uniaxial strength one measures when the sample has no lateral support.

List of definitions

Concept		**Description**
Binder	-	A stabilising substance that chemically reacts with the soil and the pore water to form a hardened material with a dense structure (CUR-commissie C121 "Eind evaluatie No-Recess testbanen Hoeksche Waard", 2001).
Binder dosage	-	The amount of binder in kilograms (kg) that is added to 1,0 cubic metre (m^3) of undisturbed soil.
Boezemkade(n)	-	A type of Dutch regional flood defence. A ‘boezemkade’ is a relatively small levee holding back water that is kept at a more or less constant level outside the levee.
D-GeoStability	-	Software programme used for the assessment of the stability of cross-sections of levees.
Gyttja	-	“Soil containing a high degree of organic matter originating from remains of plants and animals rich in fats and proteins” (Building Research Establishment (BRE), 2002, p. 4).
Inward macrostability	-	The resistance against sliding of a mass of soil from the inner slope of the levee (polder side).
Legger	-	A legger is a collection of maps in which the dimensions and debtors of every waterway and every levee and dike are precisely recorded (Hoogheemraadschap De Stichtse Rijnlanden, n.d.).
Mass stabilisation	-	The stabilisation of soft (organic) soils, typically with high water contents, on site and in-place with a binder or a blend of binders (Forsman et al., 2015).
Regional flood defence (levee)	-	Regional flood defences are flood defences that protect the land against flooding from inland water, such as from small lakes, small rivers and canals (Rijkswaterstaat, n.d. b).
Remoulded soil	-	A soil whose structure was changed or modified as a result of disturbance or manipulation.
(Soil-binder) mixture	-	A mixture in which soil is mixed with binder, possibly also with an amount of water added to the mixture on top of the naturally present water in the soil.
Stabilised soil sample	-	A (partially) cured sample of stabilised soil produced from a (soil-binder) mixture.

List of symbols

Symbol		Description	Unit
B	-	Skempton's coefficient	[-]
c'	-	Effective cohesion	[kN/m^2] (kPa)
C_c	-	Compression index	[-]
C_s	-	Secular compression coefficient below pre-consolidation pressure	[-]
C_s'	-	Secular compression coefficient above pre-consolidation pressure	[-]
C_{sw}	-	Swelling index	[-]
C_p	-	Primary compression coefficient below pre-consolidation pressure	[-]
C_p'	-	Primary compression coefficient above pre-consolidation pressure	[-]
CR	-	Compression ratio	[-]
C_r	-	Reloading index	[-]
$c_{v;10}$	-	Coefficient of vertical consolidation at 10°C	[m^2/s]
C_α	-	Coefficient of secondary compression	[-]
COV_X	-	Coefficient of variation of a soil parameter	[-]
d	-	Diameter of the sample	[mm]
$D_{extruded}$	-	Diameter of the extruded stabilised soil sample	[mm]
D_{mould}	-	Inner diameter of the mould	[mm]
e_0	-	Initial void ratio	[-]
E_{oed}	-	Oedometer stiffness modulus	[kN/m^2] (kPa)
$E_{u;50}$	-	Undrained secant Young's Modulus at 50% strength	[kN/m^2] (kPa)
$FoS_{required}$	-	Required Factor of Safety against instability of the levee	[-]
h	-	Height of the sample	[mm]
h/d	-	Height to diameter ratio of a sample	[-]
$h_{extruded}$	-	Height of the extruded stabilised soil sample	[mm]
$k_{v;10}$	-	Vertical hydraulic conductivity at 10°C	[m/s]
m_b	-	Mass of binder in a stabilised soil sample	[g]
$m_{component}$	-	Required mass of the component for the mixture	[g]
$m_{min.\ layer}$	-	Mass of mixture required for creating 1 layer of compacted mixture	[g]
$m_{mould+compacted\ mixture}$	-	Mass of the mould when filled with the mixture	[g]
m_{mould}	-	Mass of the (empty) mould	[g]
m_s	-	Mass of soil solids in a soil sample	[g]
$m_{soil\ min.}$	-	Minimum mass of material that is required to fill one mould	[g]
m_v	-	Coefficient of volume compressibility	[m^2/kN] (kPa^{-1})
m_w	-	Mass of water in a moist (wet) soil sample	[g]
p'	-	Mean effective stress	[kN/m^2] (kPa)
q	-	Deviator stress	[kN/m^2] (kPa)
RR	-	Reloading ratio	[-]
s'	-	Effective normal stress ($(\sigma_1' + \sigma_3')/2$)	[kN/m^2] (kPa)
SR	-	Swelling ratio	[-]
S_r	-	Degree of saturation	[%]
S_u	-	Undrained shear strength	[kN/m^2] (kPa)
t	-	Shear stress ($(\sigma_1' - \sigma_3')/2$ and $(\sigma_1 - \sigma_3)/2$)	[kN/m^2] (kPa)
UCS	-	Unconfined compressive strength	[kN/m^2] (kPa)
u_f	-	Displacement at failure of the sample during a laboratory test	[mm]
$u_{horizontal}$	-	Horizontal displacement of the upper half of the soil specimen with respect to the lower half of the soil specimen (shearbox test)	[mm]
V_b	-	Volume of binder in a stabilised soil sample	[m^3]
V_{mould}	-	Volume of the mould	[L]
V_s	-	Volume of soil solids in a soil sample	[m^3]
V_w	-	Volume of water in a moist (wet) soil sample	[m^3]
w/b	-	Water-to-binder factor ($= m_w/m_b$)	[-]
w_{des}	-	Desired or target water content of the soil	[%]

w_{nat}	-	Natural water content of the soil ($= m_w/m_s$)	[%]
w_{sat}	-	Water content of the soil at saturation	[%]
w_{stab}	-	Water content of the stabilised soil ($= m_w/(m_s + m_b)$)	[%]
X_d	-	Design value of soil parameter	[any]
X_k	-	Characteristic value of soil parameter	[any]
X_m	-	Mean value of soil parameter	[any]

Symbol		**Description**	**Unit**
α	-	Scaling factor to account for percentage increase in strength	[-]
α_{binder}	-	Dosage of a binder (component)	[kg binder/m^3 undisturbed soil]
α_{water}	-	Dosage of water required for mixture	[kg water/m^3 soil]
γ	-	Shear strain	[%]
γ_{bulk}	-	Bulk unit weight of the soil	[kN/m^3]
γ_d	-	Model factor	[-]
γ_{dry}	-	Dry unit weight of the soil	[kN/m^3]
γ_f	-	Shear strain at failure	[%]
$\gamma_{mat(X)}$	-	Partial material factor on soil parameter X	[-]
γ_n	-	Damage factor	[-]
γ_s	-	Schematisation factor	[-]
γ_{sat}	-	Saturated unit weight of the soil	[kN/m^3]
$\gamma_{stab.;bulk}$	-	Bulk unit weight of the stabilised soil	[kN/m^3]
$\gamma_{stab.;dry}$	-	Dry unit weight of the stabilised soil	[kN/m^3]
Δu	-	Change in pore pressure as measured during shearing (triaxial test)	[kN/m^2] (kPa)
ε_a	-	Axial strain	[%]
ε_f	-	Vertical/axial strain at failure of the sample during a laboratory test	[%]
ε_v	-	Vertical strain	[%]
ε_{vol}	-	Volumetric strain	[%]
$\varepsilon_{vol;C}$	-	Volumetric strain at the end of consolidation	[%]
ε_{1C}	-	Vertical strain at the end of consolidation	[%]
μ	-	Mean value of any soil parameter	[any]
ρ_{bulk}	-	Bulk density of the soil (density of the soil at natural water content)	[kg/m^3]
ρ_{des}	-	Density of the soil at the desired water content	[kg/m^3]
ρ_{dry}	-	Dry density of the soil	[kg/m^3]
ρ_{guide}	-	Expected density of the mixture directly after filling the mould	[kg/m^3]
ρ_{req}	-	Required mass of drier soil to be mixed with a certain amount of water to get a mass of wet soil with the desired water content representative of a cubic metre of undisturbed soil	[kg dry soil/m^3 undisturbed soil]
ρ_s	-	Particle density (density of the soil solids)	[kg/m^3]
ρ_{sat}	-	Saturated density of the soil	[kg/m^3]
ρ_{stab}	-	Density of the soil-binder mixture	[kg/m^3]
$\rho_{theoretical}$	-	Theoretical maximum density of a mixture	[kg/m^3]
σ'_p	-	Apparent pre-consolidation pressure or yield stress	[kN/m^2] (kPa)
σ_v	-	Applied vertical stress during a laboratory test	[kN/m^2] (kPa)
σ'_1	-	Major principle effective stress	[kN/m^2] (kPa)
σ_{1C}		Major principle stress at the end of consolidation	[kN/m^2] (kPa)
σ'_3	-	Minor principle effective stress	[kN/m^2] (kPa)
σ_{3C}	-	Minor principle stress at the end of consolidation	[kN/m^2] (kPa)
σ_n	-	Normal stress	[kN/m^2] (kPa)
σ_X	-	Standard deviation of any soil parameter	[any]
τ	-	Applied shear stress during a laboratory test	[kN/m^2] (kPa)
τ_{max}	-	Maximum shear stress measured in a shearbox test	[kN/m^2] (kPa)
ϕ'	-	Effective angle of internal friction	[°]

1 Introduction

In this chapter an introduction to the research outlined in this report is given. First of all, the problem description that gave rise to this research is presented in section 1.1. The research objective that was formulated for this research is subsequently presented in section 1.2. The intended approach to reach the objective is described in section 1.3. The demarcations of this research are listed in section 1.4. The scientific relevance of this research is described in section 1.5. The thesis outline is shown in section 1.6.

1.1 Problem description

In the Netherlands, we face a challenge. 59% of the country is vulnerable to flooding, either due to vulnerability to river flooding or because the area lies below sea level (Planbureau voor de Leefomgeving, n.d.). This 59% of the Netherlands just so happens to be the area where half of the population of the country lives and where most of the gross national product is earned. As a result, protection against flooding is a necessity (Deltacommissaris, n.d.).

In order to create protection against flooding, the Dutch built levees over the past centuries (Rijksdienst voor het Cultureel Erfgoed, n.d.). However, these constructed levees are unable to hold back the water indefinitely. Natural and anthropogenic soil subsidence (TNO, n.d.) and higher water levels as a result of sea level rise and increased river discharges (Dutch national government and Water Boards, 2019) are examples of factors that reduce the stability of the levee. Since these factors cause a slow decrease of the stability over time, flood defences need to be examined regularly to determine whether they still meet present-day safety standards (Rijkswaterstaat, n.d. a).

In the event a levee is found to fail the safety standard on inward macro-stability, measures are taken to reinforce the levee. Reinforcement of levees traditionally occurs by either decreasing the slope or by constructing a berm (CUR-commissie C141, 2007). These solutions are usually preferred as they are the most effective and the simplest to apply. However, these traditional solutions are not always applicable or desirable. Traditional solutions have an impact on the landscape and ecology and require a lot of space directly next to the levee which is not always available (CUR-commissie C141, 2007). Thus, if a levee is to be reinforced when a traditional solution is either not possible or undesirable, an alternative reinforcement method will need to be applied.

One possible alternative reinforcement method could be mass stabilisation. Mass stabilisation is an originally Finnish soil improvement technique in which soft soil, typically with large water contents, is mixed in-situ with a binder (Forsman et al., 2015). After mixing, the binder will cure, resulting in a stronger and stiffer material in time. Presumed advantages of applying this method at levees include:

- Mass stabilisation could be applied at the crest and the slope of the levee, thus not necessarily requiring additional space next to the levee;
- The landscape will not change, as the shape of the levee after reinforcement will not have changed compared to the shape of the levee prior to reinforcement;
- It is expected that less material will have to be supplied to the site for reinforcement with mass stabilisation compared to other reinforcement methods like decreasing the slope or constructing a berm, which may save on CO_2-emissions as less transport is needed.

Despite the possible advantages of mass stabilisation compared to traditional solutions for reinforcing levees, hardly any research into reinforcing levees with mass stabilisation has been carried out to date. As a result, it is unknown whether applying mass stabilisation in this manner is even possible in terms of design and execution. This gave rise to this research, in which the technical feasibility of applying mass stabilisation for reinforcing levees is examined.

1.2 Research objective

The main objective of this thesis is to determine whether the application of mass stabilisation for improving the inward macro-stability of Dutch regional flood defences is technically feasible. The main research question for this study is formulated as follows:

'Is the application of mass stabilisation for improving the inward macro-stability of regional flood defences by stabilising strips of soil technically feasible?'

Mass stabilisation is considered a technically feasible reinforcement method for regional flood defences, ***from now on referred to as levees***, if it is proven that the technique meets the following three criteria:

- Mass stabilisation is able to solve a stability deficit at levees;
- The desired strength in compliance with Dutch safety standards can be achieved by stabilising the soil;
- Mass stabilisation can be carried out at levees without causing the levee to fail during execution.

To examine whether mass stabilisation meets these three criteria, a number of sub-questions were drawn up. For the criterion of the ability of mass stabilisation for solving a stability deficit at levees, the following two sub-questions were drawn up:

1. ***'Which increases in the Factor of Safety can be realised by stabilising strips of soil at the levee?'***
2. ***'Where should the stabilisation of the soil at the levee preferably be carried out from an empirical point of view?'***

For the criterion of the achievability of the desired strength a broader sub-question was drawn up. Besides determining whether the desired strength can be achieved with soil stabilisation, it is also relevant to know the development of the strength in time in order to examine the practicability of mass stabilisation at levees. For this reason, the following sub-question was drawn up:

3. ***'How do the strength properties of the soil(s) to be stabilised from the selected case change as a result of stabilisation with a preselected binder and dosage?'***

Lastly, the following sub-question was drawn up for the criterion on the practicability of mass stabilisation at levees:

4. ***'Is the application of mass stabilisation at the levee of the selected case practicable?'***

In this research, these research questions were answered by following the methodology as outlined in the next section.

1.3 Methodology

In order to reach the objective of this research, a number of steps were taken. These steps are shown in the flow chart as presented in figure 1.1. A detailed description of the approach taken for each step is presented in the next subsections.

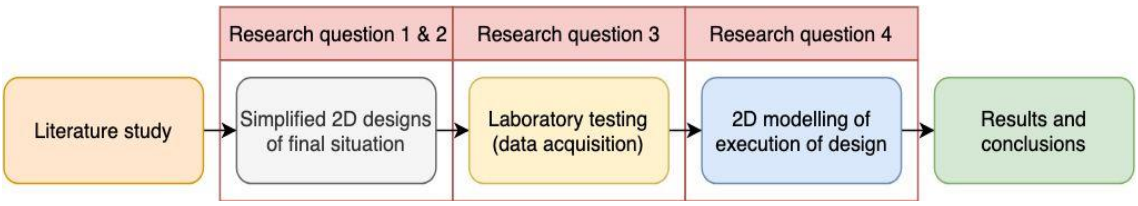

Figure 1.1; Flow chart showing the steps taken during this master thesis to reach the stated objective.

1.3.1 Literature study

The research was started with a literature study in which a broad search for information on the mass stabilisation soil improvement technique was carried out. The purpose of the literature study was to determine what was already known about mass stabilisation and could directly be used in this study, as well as determining which matters required additional research. During the literature study, information on the following subjects was collected:

- General information on the mass stabilisation technique;
- Possible and current applications of mass stabilisation;
- Strength properties and unit weights of stabilised soils.

The findings from literature were reported and used to identify additional knowledge gaps that required further study in order to examine the technical feasibility of reinforcing levees using mass stabilisation.

1.3.2 Design analyses

A major part of determining the technical feasibility of applying mass stabilisation for reinforcing regional flood defences is to determine whether mass stabilisation can be used to solve a stability deficit at levees. In order to determine this, the influence of soil stabilisation on the inward macro-stability of two real Dutch levees was examined by means of two-dimensional stability analyses.

For both Dutch levees, a cross-section was taken and subsequently modelled in D-GeoStability (version 17.1). In the model, an infinite strip of stabilised soil with remoulded soil around it was modelled at three different spots at the levee: at the toe, at the slope and at the crest. An improved strength was assigned to the stabilised soil and the influence of the presence of this stronger soil on the inward macro-stability of the levee was determined. In addition to this, the strips of stabilised soil were also separately modelled with an increased unit weight to evaluate the change in the Factor of Safety. The results of the two-dimensional stability analyses were subsequently used to determine:

- The increases in the Factor of Safety that could be realised for each levee;
- The preferred spot for stabilisation at each levee by comparing the obtained Factors of Safety

With these results, sub-questions 1 and 2 as presented in section 1.2 were answered.

After the analyses, a single strip of stabilised soil from a single case was selected for which the achievability of the desired strength by means of soil stabilisation and the practicability of mass stabilisation at levees was examined. In order to examine the achievability of the desired strength, the strength needed to achieve the required increase in the Factor of Safety had to be determined. For the selected strip of stabilised soil, the required strength was determined by modelling:

- A variety of different strengths;
- Either an increase or no change in the unit weight depending on the position at the levee;
- A layer of remoulded soil around the strip of stabilised soil.

1.3.3 Laboratory research

The achievability of the strength needed for the strip of stabilised soil from the selected case is an important factor in determining the technical feasibility of applying mass stabilisation for reinforcing regional flood defences. In addition, it is also important to know the strength development in time for determining the practicability of applying mass stabilisation for reinforcing regional flood defences. In order to examine both aspects, soil samples were stabilised in the geotechnical laboratory of Fugro NL Land B.V. and tested for strength.

First, soil samples were taken in the field close to the examined cross-section of the selected levee. Remoulded and undisturbed samples were taken from all layers that should be stabilised based on the strip of stabilised soil from the selected case. In the laboratory, the undisturbed soil samples were subsequently subjected to a variety of laboratory tests to measure the index, strength and stiffness properties of these soils. These measurements were mostly used as reference measurements, although the index properties were also required for making soil-binder mixtures representative of the field.

Next, many different soil-binder mixtures were made to determine the most suitable binder and an optimal dosage for the most suitable binder for the stabilisation of all soil types sampled from the field. Once a binder type and binder dosage were selected, multiple samples of the same soil-binder mixtures were made after which each sample was left to cure a different amount of time. After curing, the strength of all samples was determined and subsequently combined to determine the strength development of the examined mixtures in time.

Lastly, the same soil-binder mixtures were produced again and left to cure until curing was complete. After complete curing, the samples were subjected to the same laboratory tests as the undisturbed soil samples were to allow for comparison of the properties, the most important being the strength.

The results of the laboratory research were used to answer sub-question 3 as presented in section 1.2.

1.3.4 Implementation analyses

The last criterion for determining the technical feasibility of applying mass stabilisation for reinforcing regional flood defences is the practicability of mass stabilisation at levees. To determine whether mass stabilisation is practicable at the selected levee without causing it to fail during execution, the stability of a fixed segment of the levee was assessed during execution using two-dimensional stability analyses.

At the examined levee, a continuous stabilisation of 1,0 metre long blocks of soil to a strip of stabilised soil at the selected spot at the levee was modelled. After stabilisation of a block, a preload was applied and left on top of the stabilised soil until the stabilised soil had developed sufficient strength as a result of curing. This method of execution was examined in four scenarios, which differed in the assumptions on two important variables:

- The initial strength of the stabilised soil directly after mixing;
- The unit weight of the stabilised soil.

In each of the four scenarios, the stability of the levee during execution was assessed using two-dimensional stability analyses with weighted averages of the strength, unit weight and preload over the examined segment. The strength development of the stabilised soil applied in all scenarios and the increased unit weight applied in some scenarios were based on the laboratory measurements.

With the results from all four scenarios, sub-question 4 as presented in section 1.2 was answered.

1.3.5 Conclusions

In this final step, the results obtained for each of the three criteria were combined to determine whether the application of mass stabilisation for reinforcing regional flood defences is technically feasible, thereby answering the main research question presented in section 1.2.

1.4 Demarcations of the research

For the purposes of this research, some matters that are related to mass stabilisation were not examined nor considered. These matters included the following:

- Environmental aspects of mass stabilisation;
- In-situ variation of the strength of mass stabilised soils;
- Influence of mass stabilisation on other failure mechanisms of levees besides the inward macro-instability;
- Influence of stiffness and permeability of mass stabilised soils on the stability of levees.

First of all, in the Netherlands there will be environmental requirements on the in-situ stabilisation of soil as a result of adding material not naturally found in soils to the subsurface. These environmental requirements include among others the allowable amount of leaching of substances from the stabilised soils to the environment. Although the determination of the leaching characteristics of stabilised soils remains a topic of debate in the Netherlands as a result of current environmental legislation (Bos, 2018), especially considering that it is currently the biggest obstacle in getting mass stabilisation widely applied in the Netherlands, this issue will not be dealt with in this thesis. Neither will the examined stabilised soil samples produced during the laboratory research be tested for leaching. Both matters will not be included in this thesis as this is not the field of expertise of the author nor the purpose of this research.

Secondly, it is not known how the variability of the strength of the soil changes as a result of mass stabilisation in both the laboratory and in the field. To quantify this, both field stabilisations and a lot of laboratory tests would have to be carried out. This was not considered feasible in this research and was therefore considered out of scope. Besides this, it is known that it is possible that in properly stabilised soils pockets of weaker stabilised soil could be present (Forsman et al., 2015). This may heavily influence the stability of levees, as the slip surface defining the stability of the levee would prefer to pass through these pockets of weaker soil. Although it is known that this could occur, this effect was not considered in this research and thus considered out of scope.

Thirdly, only the effect of mass stabilisation on the inward macro-stability of levees (i.e. regional flood defences) was examined in this research. The influence of mass stabilisation on all other failure mechanisms of levees, such as piping, micro-stability and outward macro-stability, was not examined in this research.

Lastly, mass stabilisation of soil will not only change the strength, but also the stiffness and the permeability of the soils. However, in this research a focus was applied on strength and how this influenced the inward macro-stability of levees. As a result, the effects of changes in stiffness or permeability of the soils on the inward macro-stability of levees were not examined and thus considered out of scope.

1.5 Contribution to science and practice

In this thesis, the existing knowledge on applying mass stabilisation at flood defences will be expanded. Since little research into this particular application of mass stabilisation has been carried out to date, little is known about designing a reinforcement of a levee with mass stabilisation. Additionally, since no actual application of mass stabilisation for reinforcing levees has been carried out to date, little is also known about the practicability of the technique at levees. By examining these aspects, new insights will be developed and new knowledge will be obtained. This helps to better understand the possibilities and practicability of the technique, possibly resulting in an additional application field for mass stabilisation. This is turn could also provide engineers with an alternative reinforcement method for levees which may be fast to apply, durable and customisable.

Besides this, the existing knowledge on strength of stabilised soils will be expanded by the laboratory research, in particular on the effective strength parameters (i.e. effective cohesion and effective angle of internal friction) and the mobilisation of these parameters. This knowledge is important for designing reinforcements of Dutch regional flood defences in compliance with Dutch safety standards, as current reinforcements of regional flood defences are designed using effective strength parameters evaluated at small strains (Mohr-Coulomb model). Although in this research only the small strain strength approach was examined as this was the prevailing method at the time of writing this thesis, the results of the laboratory research could also be used to evaluate the applicability of the technique at large strain strengths (Critical State Soil Mechanics).

1.6 Thesis outline

The steps that are taken in this study as shown in figure 1.1 (see section 1.3) make up the structure of this report. In chapter two an overview of the collected literature on mass stabilisation is presented. In chapter three various designs for improving the inward macro-stability of two real Dutch regional flood defences using mass stabilisation are presented and discussed. In chapter four the laboratory research programme and the results from the laboratory tests are presented. In chapter five the results of the implementation analyses for realising the selected design are presented. This thesis ends with chapters on the conclusion, discussion and recommendations.

2 Literature study

2.1 Introduction

Before this research was shaped into a technical feasibility study for the application of mass stabilisation for reinforcing regional flood defences, literature on mass stabilisation was briefly searched for. The purpose of the literature search was to quickly establish what was currently known about mass stabilisation and its applications. After a short search, it became clear that mass stabilisation had never been applied at levees before. Since alternative, possibly more sustainable methods for the reinforcement of Dutch levees compared to traditional methods or reinforcement are highly desired (CUR-commissie C141, 2007), innovations like mass stabilisation become an interesting alternative. As a result, this master thesis research into the technical feasibility of reinforcing levees with mass stabilisation was started.

In order to determine whether applying mass stabilisation for reinforcing levees was technically feasible, two major aspects had to be examined:

- The ability of mass stabilisation to solve a stability deficit of a levee;
- The practicability of the stabilisation of the soil at levees.

In order to examine these two aspects, information from literature was required. As a result, a broad literature study was carried out to establish what was already known and could be used to examine the aforementioned two aspects. In addition, the literature could be used to determine the gaps in existing knowledge. For this, information on three main categories was collected during the literature study:

- The mass stabilisation technique itself: what is it precisely and what is already known about the technique?
- Possible applications of mass stabilisation: for which applications has mass stabilisation been used before?
- Properties of stabilised soils: what is currently known about the properties of stabilised soils?

The collected information on each of these three main categories is presented in respectively section 2.2, 2.3 and 2.4. The collected information was subsequently analysed and used to identify the knowledge gaps in existing literature. These matters had to be examined on top of the two aspects listed earlier in order to be able to examine the technical feasibility of applying mass stabilisation for reinforcing levees. Subsequently, fitting research questions were drawn up for these matters. This process of information gathering, interpretation and formulation of research questions is presented in figure 2.1.

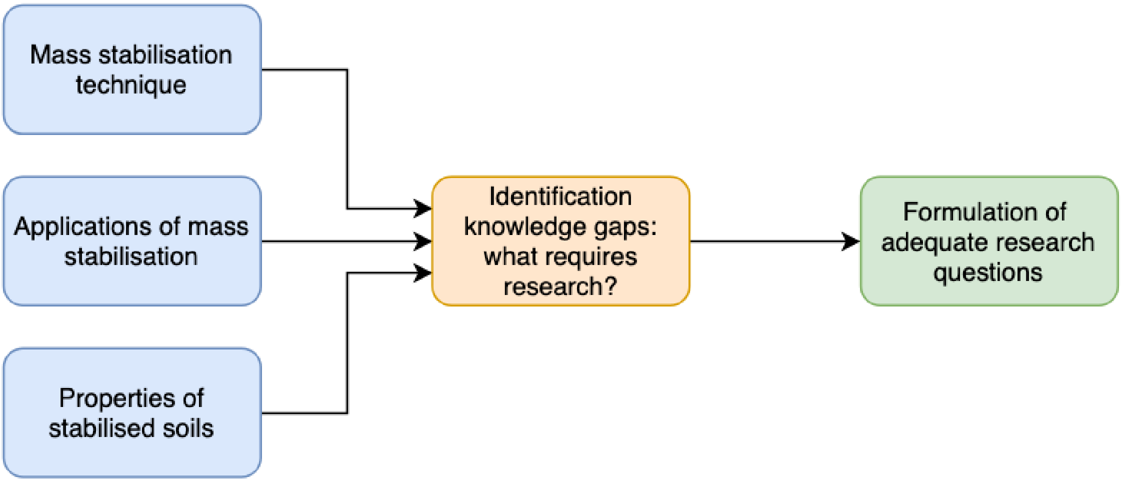

Figure 2.1; Flow chart of the information gathering for the literature study and how this lead to the research questions.

This chapter ends with section 2.5 in which a conclusion is drawn on the literature study. In this conclusion, a brief summary of the findings related to the three main categories is presented. In addition to this, the identified knowledge gaps that should be examined in this research are presented as well.

2.2 Mass stabilisation

This section presents the collected information on the soil improvement technique called mass stabilisation. It covers many aspects of the technique, such as the advantages of the technique, the typical binders used and execution of the technique.

2.2.1 Soil improvement technique

Mass stabilisation is the stabilisation of soft soils, typically with high water contents, on site and in-place with a binder, or a blend of binders, in order to create a homogeneous and stronger soil layer (Forsman et al., 2015) with the goal of reducing settlements and/or improving the stability of a structure (Building Research Establishment (BRE), 2002).

The technique was developed in Finland in the early nineties and has been successfully applied in various projects since 1993. Initially applied for the stabilisation of peat in road and railway constructions, it quickly became clear that mass stabilisation could also be used in other geotechnical constructions and in environmental engineering applications. As time passed, the technique was developed further and started being applied in other countries (Forsman et al., 2015), with the first Dutch application of the technique occurring in Stolwijk in 2015 (Dekker, 2015b).

Mass stabilisation is carried out with the ALLU stabilisation system, which consists of an excavator with a special mixing attachment and a pressure feeder, as shown in figure 2.2. The pressure feeder feeds dry material, also known as a binder, from the tank to the mixing attachment, which subsequently injects the binder using compressed air and mixes it with the soil (Allu Finland Oy, 2007). The binder subsequently reacts with the moisture present in the soil to form a stabilised material, thereby decreasing the water content (Building Research Establishment (BRE), 2002). However, this reaction requires sufficient moisture to be present in the soil. As a result, mass stabilisation is only applicable in soils with moisture contents larger than 40% and is therefore best suited for saturated soils with high moisture contents, such as clays and (strongly) organic soils (Mullins & Gunaratne, 2015).

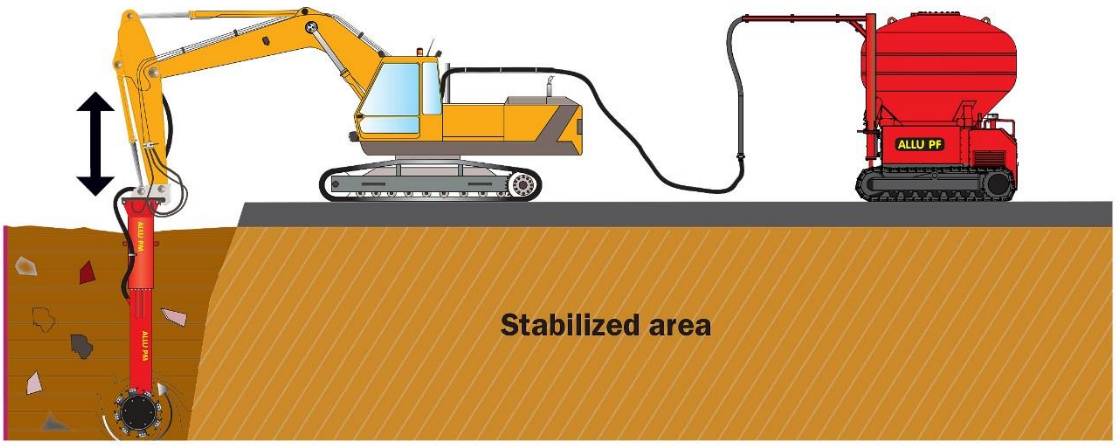

Figure 2.2; Image of the ALLU stabilisation system stabilising a soil. It comprises of an excavator with an ALLU PMX Power Mixing attachment and an ALLU PF 7 (single tank) or ALLU PF 7+7 (double tank) pressure feeder. This image shows a single tank pressure feeder (Wordpress, n.d.).

After stabilisation has been completed, a layer of strengthened soil has been created. Depending on the case, either a fully stabilised or a partially stabilised soil layer is created as shown in figure 2.3. A fully stabilised soil layer is the complete stabilisation of one or multiple soft soil layers up to a firm and stronger soil layer at depth. Such a stabilised soil layer has the advantage that it hardly settles when loaded by a structure on top. However, a full stabilisation can only be realised if the firm soil layer lies within 7,0 to 8,0 metres from the ground surface, which is the maximum depth the ALLU stabilisation system can reach (Forsman et al., 2015).

If the total soft soil layer thickness exceeds 7,0 – 8,0 metres, only a partial soil stabilisation will be possible. A partially stabilised soil layer is the partial stabilisation of one or soil layers up to a certain depth. A partially stabilised soil layer will settle more than a fully stabilised soil if loaded, but the loads exerted will be distributed to the underlying soil layers. This could result in more uniform settlement behaviour and could prevent harmful differential settlements, although this depends on the untreated soil layers below the stabilised soils (Forsman et al., 2015).

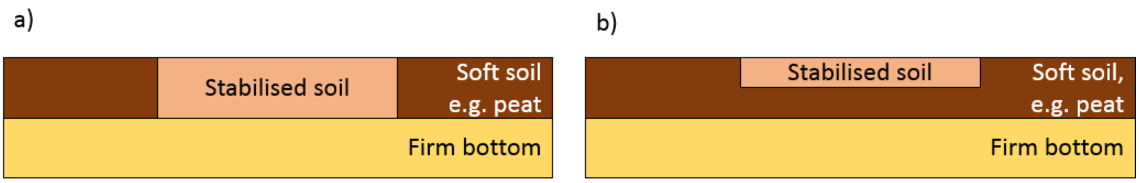

Figure 2.3; The two different implementations of mass stabilisation (Forsman, Jyrävä, Lahtinen, Niemelin, & Hyvönen, 2015). Image a) shows a stabilisation of the entire soft soil thickness (full stabilisation). Image b) shows a stabilisation of a part of the soft soil thickness (partial stabilisation).

2.2.2 Advantages and disadvantages

Mass stabilisation is a versatile soil improvement technique and therefore has a wide array of advantages. However, the technique also has its drawbacks. An overview of the general advantages and disadvantages of mass stabilisation is presented below.

The general advantages of mass stabilisation include the following:

- **Flexible improvement of engineering properties of the soil:** the engineering properties of soft soils can be changed to meet the demands (Building Research Establishment (BRE), 2002).
- **Rapid:** mass stabilisation is a relatively fast soil improvement method compared to other soil improvement techniques, such as traditional preloading in combination with vertical drainage (CUR onderzoekscommissie D34 "Kalk-cementkolommen", 2001).
- **Little transport is needed:** the immediate subsurface is used, so little soil needs to be excavated and carried away from the site or supplied to the site for construction. The result is that little transport will be necessary, limiting CO_2-emissions and costs of the transport (KWS Infra Rotterdam, 2016).
- **Little environmental nuisance:** since little transport is required, there will be little traffic burden for public streets and road networks caused by the transport vehicles. Also, vibrations and noise levels are low during the stabilisation works (Forsman et al., 2015).
- **Low carbon footprint:** as a result of less material supply and transport there will be less CO_2-emissions. Research into a specific case has shown that up to 20% and 50% of CO_2-emissions were saved by applying mass stabilisation compared to applying respectively expanded polystyrene (EPS) and traditional preloading using sand (Bos, Mijs, & Roelofs, 2018).
- **Industrial by-products can be used:** industrial by-products, such as flue gas desulphurisation gypsum (FGD-gypsum), fly ash and blast furnace slag, could be used as binders for the stabilisation of the soil. This saves on overall binder costs and allows for obtaining stabilised soils with better chemical and/or technical properties (Forsman et al., 2015).
- **Contaminated soil can be treated:** soils containing harmful substances could be stabilised on site to prevent the spread of the contamination to the surroundings. This saves the need for excavating, transporting and subsequent landfilling of the contaminated soil (Forsman et al., 2015).
- **Limited (differential) settlements:** settlement of constructions built on top of stabilised soil are limited, with the magnitude of the occurring settlement depending on the load of the construction and whether a full or partial stabilisation of the soil layer(s) was carried out. Differential settlements of the structure are also reduced due to the load distribution of the stabilised soil (Forsman et al., 2015).
- **Low maintenance costs:** if a full stabilisation was carried out, then the construction on top will settle little, significantly reducing the maintenance costs (Building Research Establishment (BRE), 2002).
- **Stabilised material remains workable:** upon a limited addition of binder to the soil, the stabilised soil material will remain workable for future projects. This allows the digging of trenches in the stabilised soil as well as pile driving through the stabilised soil (Bos, 2018).

The disadvantages and limitations of mass stabilisation include the following:

- **Risk of leaching of harmful components:** the introduction of a material not normally present in the soil by mass stabilisation could lead to leaching of harmful substances. The leaching of these substances could occur from either the soil or the binder. This may cause contamination of the groundwater (CUR onderzoekscommissie D34 "Kalk-cementkolommen", 2001).
- **Underground utilities may pose problems:** underground utilities, such as cables, pipes and old foundations, impede the execution of mass stabilisation (U.S. Department of Transportation, 2013).
- **Curing time:** the hardening process, and therefore the strength development, of the stabilised soil takes time. This means that it takes time for the strength of the stabilised soil to develop to sufficient levels before construction near or on top of the stabilised soil may occur (Building Research Establishment (BRE), 2002).
- **Limited installation depth:** the maximum depth to which soils can be stabilised with mass stabilisation is 7,0 to 8,0 metres (Forsman et al., 2015).
- **Not applicable in all soils:** mass stabilisation cannot be applied in soils that are very stiff or dense, like sands and gravel, or in soils rich in boulders (U.S. Department of Transportation, 2013).

2.2.3 Binders

Improving the properties of soils requires a stabilising substance that chemically reacts with the soil to form a hardened material with a dense structure (CUR-commissie C121 "Eind evaluatie No-Recess testbanen Hoeksche Waard", 2001). These required substances are called binders (Building Research Establishment (BRE), 2002). This section deals with binders by discussing the binders that could be used in soil stabilisation, the experience with binders in soil stabilisation and the typical binder quantities.

2.2.3.1 Possible binders

There are two main types of binders: hydraulic and non-hydraulic binders. Hydraulic binders are binders that harden out in the presence of water, whereas non-hydraulic binders do not harden out in the presence of water. Non-hydraulic binders either remain inert in the presence of only water or they dissolve in the water. Although non-hydraulic binders do not seem to be useful for stabilisation, they are sometimes applied to activate latent hydraulic binders (Building Research Establishment (BRE), 2002). Latent hydraulic binders are binders that do not possess hydraulic properties by themselves, but start behaving as a hydraulic binder when exposed to calcium-rich solutions. An example of latent hydraulic binders are pozzolans, which are materials that react hydraulically in the presence of lime (Livesey, 2018). Examples of (latent) hydraulic and non-hydraulic binders that could be used in soil stabilisation are presented in table 2.1.

Most commonly, a hydraulic binder, such as cement, is used as a basis constituent for the binder used in soil stabilisation. The reason cement is often used is because it allows for fast initial curing of stabilised soils. Besides cement, lime can also be used as a basis constituent (Forsman et al., 2015). Lime is a non-hydraulic binder and only reacts with clay minerals, a pozzolanic process that takes a lot of time. A consequence of this is that the short-term strength of the stabilised soil will be (significantly) lower than when cement is applied (CUR onderzoekscommissie D34 "Kalk-cementkolommen", 2001). Therefore when lime is required to be added, it is typically applied in combination with other binder components to allow for faster strength development of the stabilised soil (Forsman et al., 2015).

Alternatively, should the stabilisation of the soil with the basis constituent not lead to the desired result, additives could be added. Additives are added to reduce the harmful effects of phenomena that may occur that interfere with or negatively affect the strength development due to the presence of certain components or ions in the soil. These phenomena include the following (CUR-commissie C121 "Eind evaluatie No-Recess testbanen Hoeksche Waard", 2001):

- Delay of the hardening process;
- Increase in porosity;
- Reduction of the adsorption of organic components;
- Decrease in the unconfined compressive strength;
- Reduction of the binding capacity.

Other reasons one might have to add additives to the basis constituent is to reduce the emission of CO_2. An example is the addition of fly ash to cement. A research into the environmental life cycle inventory of Portland cement concrete has shown that, based on the measurements carried out during the research, every percent of cement replaced with fly ash could save about 0,7% in energy consumption for the production of concrete, saving a lot of CO_2 being emitted (Nisbet, Marceau, & VanGeem, 2002).

Table 2.1; Overview of possible binders for the stabilisation of soils (Building Research Establishment (BRE), 2002) (CUR onderzoekscommissie D34 "Kalk-cementkolommen", 2001) (CUR-commissie C121 "Eind evaluatie No-Recess testbanen Hoeksche Waard", 2001) (joostdevree.nl, n.d.) (Mortar Industry Association, 2013) (Geos N.V., 2018) (Van Mannekus & Co.) (Claycrete Global, n.d.) (Carmeuse, 2015) (Forsman et al., 2015).
Legend:
H – hydraulic binder (hardens in the presence of water)
LH – latent hydraulic binder (hardens only in the presence of both water and calcium)
NH – non-hydraulic binder (does not harden in the presence of water or dissolves in water)

<table>
<tr><th>Basis for binder blend</th><th>Additives</th><th>Special binders / additives</th></tr>
<tr><td>Cement (H):
• Portland cement (CEM I)
• Portland-composite cement (CEM II)
• Blast-furnace slag cement (CEM III)
• Pozzolanic cement (CEM IV)
• Composite cement (CEM V)
• Masonry cement</td><td>Non-hydraulic additives (NH):
• Anhydrite
• Clay
• Gypsum
• Limestone flour
• (Super) plasticisers
• Sand
• Water glass</td><td>Binder materials (unknown whether hydraulic or non-hydraulic):
• Claycrete (II)™
• ViaCalco</td></tr>
<tr><td>Lime (NH):
• Quicklime
• Slaked lime</td><td rowspan="2">Latent-hydraulic additives (LH):
• Ground-granulated blast-furnace slag
• Lignite fly ash
• Silica fume (i.e. microsilica)</td><td rowspan="2">Additives (unknown whether hydraulic or non-hydraulic):
• Geocrete®</td></tr>
<tr><td>Combination of lime and cement</td></tr>
</table>

2.2.3.2 Experience

In 2001, the EuroSoilStab project, in which the stabilisation of soft (organic) soils was researched, published its results. During the project, a variety of Nordic and Dutch soils were stabilised in the laboratory with different binders to determine the relative strength increase of the stabilised soil. The results of these laboratory tests were used to determine the suitability of the binders for the stabilisation of various Nordic soil types, as shown in table 2.2, and of various Dutch soil types, as shown in table 2.3 (Building Research Establishment (BRE), 2002).

Later, in 2005, a European Standard on soil stabilisation was developed. This European Standard is EN 14679 (*'Execution of special geotechnical works – deep mixing')*. In this European Standard, commonly used binders are mentioned for both dry and wet mixing of soils. Dry mixing is the injection of dry binder into the soil and the subsequent mixing of the dry binder with the soil (this is what mass stabilisation does). An overview of the commonly used binders in dry mixing according to the European standard is presented in table 2.4. Wet mixing on the other hand is the mixing of the dry binder with water (making slurry) before mixing it with the soil (Building Research Establishment (BRE), 2002). For wet mixing, it was only reported that cement is the most frequently used binder (Technical Committee CEN/TC 288, 2005).

Comparing the recommended binders of table 2.2, table 2.3 and table 2.4 shows that most recommended binders to be used in various soils are more or less the same in all three tables. However, European Standard EN 14679 mentions that lime is a suitable binder for clays, that a blend of lime and cement is a suitable binder for organic clays and that a blend of lime, gypsum and cement is a suitable binder for peat. Test results from the EuroSoilStab project show that these binders are not to be preferred in these soils. Therefore it may be wise to consult the recommendation on the suitability of binders based on the country of origin of the soil to be stabilised.

Table 2.2; Indication of the functioning of various binders in different Nordic soil types (Building Research Establishment (BRE), 2002).
Legend:
xxx very good binder in many cases
xx good binder in many cases
x good binder in some cases
- not suitable
OC organic content

Binder	Silt [OC: 0–2 %]	Clay [OC: 0-2 %]	Organic soils (e.g. organic clay) [OC: 2-30 %]	Peat [OC: 50-100 %]
Cement	xx	x	x	xx
Cement + gypsum	x	x	xx	xx
Cement + furnace slag	xx	xx	xx	xxx
Lime + cement	xx	xx	x	-
Lime + gypsum	xx	xx	xx	-
Lime + slag	x	x	x	-
Lime + gypsum + slag	xx	xx	xx	-
Lime + gypsum + cement	xx	xx	xx	-
Lime	-	xx	-	-

Table 2.3; Indication of the functioning of various binders in different Dutch soil types (CUR onderzoekscommissie D34 "Kalk-cementkolommen", 2001).
Legend:
xxx very good binder in many cases
xx good binder in many cases
x good binder in some cases
- not suitable or not known

Binder	Soil type			
	Silt	Clay	Organic clay	Peat
Portland cement	x	xxx	xxx	xx
Portland cement + gypsum	x	x	xx	xx
Blast furnace slag cement	xx	xx	xx	xxx
Blast furnace slag cement + gypsum/anhydrite	xx	xx	xxx	xxx
Lime + Portland cement	xx	xx	x	-
Lime + gypsum	xx	xxx	xx	-
Lime + furnace slag	x	x	x	-
Lime + gypsum + furnace slag	x	x	x	-
Lime + gypsum + Portland cement	xx	xxx	xxx	-
Lime	-	x	-	-

Table 2.4; Commonly used binders in dry mixing according to European Standard EN 14679 ('Execution of special geotechnical works – deep mixing') (Technical Committee CEN/TC 288, 2005).

Soil type	Suitable binder
Clay	Lime or lime + cement
Quick clay	Lime or lime + cement
Organic clay and gyttja (strong organic clay)	Lime + cement or cement + granulated blast furnace slag or lime + gypsum + cement
Peat	Cement or cement + granulated blast furnace slag or lime + gypsum + cement
Sulphate soil	Cement or cement + granulated blast furnace slag
Silt	Cement or lime + cement

2.2.3.3 Binder quantities

It cannot be predicted on beforehand which binder and dosage will give optimal results for the stabilisation of soils due to the complex chemical and physical interaction between the soil and the binder. Very similar soils stabilised with the same binder, or similar soils stabilised with binders with slightly different properties, may yield stabilised soils with very different soil properties. These results can sometimes even contradict previous experiences. Therefore, laboratory tests and field tests should always be carried out in every case to determine the optimal binder mixture and dosage for achieving a certain strength of the stabilised soil (Building Research Establishment (BRE), 2002).

However, in order to obtain an indication on the required binder dosage, one could make use of previous experiences. For any given mass stabilisation project, typically between 100 and 400 kilogram of binder per cubic metre of soil to be stabilised will be required (Building Research Establishment (BRE), 2002). However, the actual amount of binder required depends on the type of soil, the water content of the soil, the selected binder and the required soil strength (CUR-commissie C121 "Eind evaluatie No-Recess testbanen Hoeksche Waard", 2001). For some of these combinations experience has shown what the required binder type and quantity is to stabilise a soil to the desired strength. These experiences are summarised in table 2.5.

Table 2.5; Typical required quantities for the stabilisation of soils from various locations. These required quantities are based on (local) experience (Building Research Establishment (BRE), 2002) (CUR onderzoekscommissie D34 "Kalk-cementkolommen", 2001) (CUR-commissie C121 "Eind evaluatie No-Recess testbanen Hoeksche Waard", 2001).

Soil	Location	Binder quantity [kg binder/m³ soil]	Strength
Marine clays	Not mentioned	80 – 120 (dry)	Field strength (S_u) of 40 – 60 kPa
Organic soils	Not mentioned	250 – 350 (dry) 300 – 400 (wet)	Field strength (S_u) of 100 – 150 kPa
Clay soils	Sweden/Finland	70 – 200 (dry) CEM I, possibly with quicklime	Not mentioned
Organic soils	Sweden/Finland	200 – 300 (dry) CEM I or CEM III	Not mentioned
Gyttja (strong organic clay)	Sweden/Finland	120 – 200 (dry) CEM I or CEM III (40% slag)	Not mentioned
Peat	Sweden/Finland	150 – 250 (dry) CEM I or CEM III (60% slag), possibly with anhydrite	Not mentioned

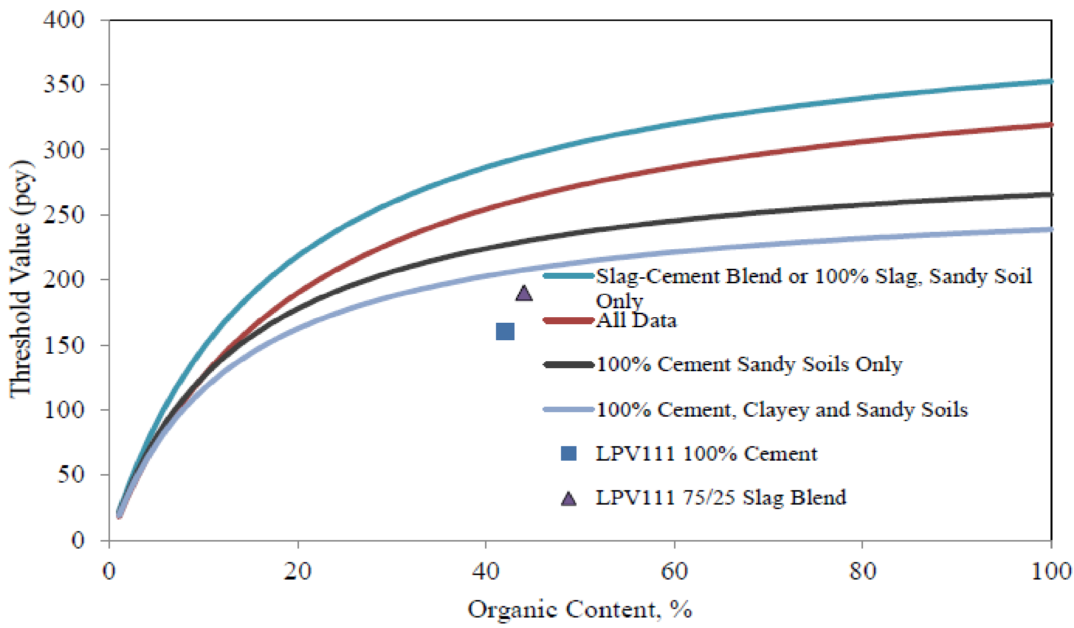

Figure 2.4; Amount of binder that is at least required to stabilise soils for various organic contents of the original soil (Costello, 2016). This threshold must be exceeded to gain strength improvement. Note: 1 pcy ≈ 0,60 kg/m³.

Organic soils tend to require a larger amount of binder than inorganic soils (see table 2.5), with increasing amounts of binder required with increasing organic content of the soil as shown by figure 2.4 (U.S. Department of Transportation, 2013). The main reason for organic soils requiring such large quantities of binder for the stabilisation is due to a lack of mineral particles in the organic soil. In inorganic soils, the binder is used to 'glue' the mineral particles together, resulting in a hardened material with a dense structure (Building Research Establishment (BRE), 2002). However, this is hardly possible in organic soils. As a result, the binder is required to form the matrix of the stabilised material which will give the stabilised organic soil its strength (Huiden, 1999).

Another reason for requiring large quantities of binder is that the binder is required to raise the pH of an organic soil. Organic soils tend to have a low pH (Huiden, 1999), which disturbs the hardening reaction. As a result, additives raising the pH, such as lime or gypsum, will sometimes be needed when stabilising organic soils (CUR onderzoekscommissie D34 "Kalk-cementkolommen", 2001).

2.2.4 Execution of mass stabilisation

During the execution of a mass stabilisation in the field, the following general steps are taken in sequence (Forsman et al., 2015):

1. Site preparation for mass stabilisation;
2. Division of the area to be stabilised in blocks;
3. Execution of the stabilisation works;
4. Construction of a loading embankment/working platform;
5. Quality control and subsequent follow-up of stabilisation soils.

First of all, the site has to be prepared before a field stabilisation can be carried out. The topsoil is harrowed and trees, shrubs, stumps and roots are removed. Additional fillings and objects (e.g. culverts, pipelines and cables) in the area to be stabilised that could also impede the stabilisation are located and subsequently removed (Forsman et al., 2015).

Following this, the area of the soil to be stabilised is divided into blocks of equal size by placing wooden sticks. This is shown in figure 2.5. The blocks are typically 3,0 x 3,0 metres (9,0 m^2) to a maximum of 5,0 x 5,0 metres (25,0 m^2) on the surface (Allu Finland Oy, 2007), with a maximum possible depth of 7,0 – 8,0 metres. Once the area is divided, an initial working platform is constructed (Forsman et al., 2015).

Figure 2.5; The division of the area in blocks prior to stabilisation (Allu Finland Oy, 2007).

Subsequently, the stabilisations works are started using the ALLU stabilisation system shown in figure 2.2 (Allu Finland Oy, 2007). The driver moves the excavator over the initial working platform to the first soil block. Once there, the mixing attachment is turned on and put in the ground. In the ground the mixing attachment is moved up and down in the soil block while the binder is simultaneously injected (Allu Finland Oy, 2007). This results in a stabilised soil within the block, but also 0,5 metre of remoulded soil below the block (CUR onderzoekscommissie D34 "Kalk-cementkolommen", 2001). During the stabilisation of the block, the driver can determine by means of a GPS-based in-situ mixing guidance and recording system whether sufficient binder has been injected and whether the soil block has been sufficiently mixed (ALLU Finland Oy, 2016).

After successful mixing of the block has been achieved, a strainer cloth is spread over the stabilised soil. Subsequently, a preload is applied on top of the strainer cloth, typically in the form of a compaction embankment of 0,5 to 1 metre thick (about 9 to 18 kPa). This compaction embankment is required to ensure consolidation of the stabilised soil layers, such that the cementation reactions start faster and surplus water from the stabilised soils is removed. Additionally, the compaction embankment usually also serves as a working platform for the stabilisation equipment during the remainder of the stabilisation works (Forsman et al., 2015). With the compaction embankment completed, the process of stabilisation and subsequent loading is repeated for the next block.

Lastly, during and after the stabilisation works, a thorough quality control is carried out. The quality control is carried out to ensure that the stabilised soil develops the desired properties. The quality control consists of field test and laboratory tests on specimens sampled from the stabilised soil. Field tests include sounding methods (i.e. CPT, column penetrometer tests and shear vane tests) and monitoring (e.g. settlement and pore water pressure), whereas laboratory tests mostly include strength tests (e.g. triaxial tests and unconfined compression tests), index tests (e.g. water content and density) and chemical tests (e.g. determination of pH, binder content or leaching).

2.3 Applications of mass stabilisation

Mass stabilisation is originally a Finnish soil improvement technique and has been applied for a few decades in Scandinavia now. Over the years, mass stabilisation has been applied for many purposes in many projects and has seen its first application in the Netherlands only a few years back. This section deals with the possible applications of mass stabilisation and the known projects in which mass stabilisation has been applied up to now.

2.3.1 Type of applications

Mass stabilisation of soils knows various applications, both geotechnically and environmentally. Mass stabilisation knows many applications because it allows the user to change the properties of the soil as desired. Some of the possible geotechnical and environmental applications of mass stabilisation include (Building Research Establishment (BRE), 2002):

- Increasing the strength of the soil for purposes of increasing the bearing capacity of the soil, essentially creating foundations for constructions such as embankments, buildings and bridges;
- Isolation and/or immobilisation of contaminations present in the soil;
- Stabilisation of the soil to protect adjacent structures from transmitted vibrations (caused by trains for example);
- Reduction of the horizontal earth pressure on sheet pile walls by stabilising the soil on the active side of the sheet pile wall;
- Stabilisation of very soft soils for tunnel boring applications;
- Increase in the stability of trenches;
- Increase in the stability of embankments yet to be built.

2.3.2 Mass stabilisation projects

Mass stabilisation has been applied in various projects across Europe, most of which were carried out in Scandinavia. Global information on some of these projects in Europe were collected, of which a summary of the project specifications is presented in table 2.6. The project specifications of mass stabilisation projects carried out in the Netherlands up to the time of writing of this thesis are presented separately in table 2.7. It should be noted that relatively little is known about the Dutch mass stabilisation projects, as the companies involved shared little information on these projects with the media.

Table 2.6; Overview of some mass stabilisation projects that have been carried out (Forsman, Marjamäki, Jyrävä, Lindroos, & Autiola, 2016) (Forsman & Dettenborn, 2016) (Koivisto, Forsman, & Leppänen, 2004) (Forsman, Jyrävä, Lahtinen, Niemelin, & Hyvönen, 2018).

Project location	Purpose of stabilisation	Quantity of soil stabilised	Stabilisation depth	Applied binder and dosage	Target strength
Tallinn, Estonia (2009)	In-situ mass stabilisation of peat to create a bearing layer for the construction of a motorway.	200.000 – 300.000 m^3	Variable between 1,8 and 3,4 m, due to the length of the motorway section.	200 kg/m^3 Portland cement with 70 kg/m^3 of a blend of two fly ashes.	Shear strength of 50 kPa.
Vantaa, Finland (2002 - 2003)	In-situ mass stabilisation of peat and column stabilisation (i.e. stabilisation of discrete or overlapping columns of soil) of clay to create a bearing layer for the construction of the yards of an IKEA establishment.	*Mass stabilisation*: 65.000 m^3 of peat; *Column stabilisation*: 110.000 m^3 of clay.	*Mass stabilisation*: up to 2,5 - 4,5 m depth; *Column stabilisation*: up to 9,0 m deep underneath mass stabilised soil.	*Mass stabilisation*: 100 kg/m^3 Portland cement; *Column stabilisation*: 90 kg/m^3 Nordkalk FTC (blend of gypsum, lime and cement)	*Mass stabilisation*: Undrained shear strength of 40 kPa after 30 days of curing; *Column stabilisation*: Undrained shear strength of 90 kPa after 30 days of curing.
Poole, United Kingdom (UK) (2013)	In-situ mass stabilisation of dredged material put between the sheet-pile and the quay wall for the extension of the quay.	1.000 – 2.000 m^3 (estimated on the basis of the original plan)	Unknown; not mentioned.	Unknown.	UCS of 100 kPa.
Luhdanoja, Mäntsälä, Finland (2002 - 2006)	In-situ mass stabilisation of the soil to construct a strong bearing layer for high speed railway tracks.	50.000 m^3	*Area 1*: soft peat up to 5,0 m deep; *Area 2*: clayey soil up to 1,0 m and soft peat up to 5,0 m deep.	200 kg/m^3 Portland cement.	Stabilised layer had to be able to carry up to a 40-50 tons pile driving rig.
Stockholm, Sweden (2014)	In-situ mass stabilisation of soft soil for the modernisation and extended construction of the Roslagsbanan railroad.	18.000 m^3	Up to 3,0 - 4,0 m depth. The soft soil layers were 10,0 m thick in total, so not all soft soil was stabilised.	200 kg/m^3 cement-slag mixture (70/30 mass ratio).	Undrained shear strength of 75 kPa.
Salmenkylä, Hamina, Finland (2014)	In-situ mass stabilisation of mud and clay to create a bearing layer for a petrol station to be built on top of.	7.400 m^3	Up to 7,0 m deep.	75 kg/m^3 Nordkalk GTC (gypsum, lime and cement).	Undrained shear strength of 40 kPa.

Table 2.7; Overview of some mass stabilisation projects that have been carried out in the Netherlands (Dekker, 2015a) (Dekker, 2015b) (Pellikaan & Hagenaar, 2016) (van Gils, 2017) (Tissink, 2016) (de Jong & Morel, 2018).

Project location	Purpose of stabilisation	Quantity of soil stabilised	Stabilisation depth	Applied binder and dosage	Target strength
Stolwijk, Province of South-Holland, Netherlands (2015)	In-situ mass stabilisation of peat for the construction of a bike lane.	Approximately 16.000 m^3.	Up to 3,5 m deep.	115 kg/m^3 of blast furnace slag cement (CEM III/B 42,5 N) with 35 kg/m^3 of gypsum.	UCS of 50 kPa after 28 days of curing.
Jirnsum, Province of Friesland, Netherlands (2016)	In-situ mass stabilisation of peat and clay under an abutment of a bridge.	Approximately 5.550 m^3.	Up to 5,0 m deep.	About 90 kg/m^3 of blast furnace slag cement (CEM III/A 32,5 N).	Unknown.
Vlaardingen, Province of South-Holland, Netherlands (2016)	In-situ mass stabilisation of among others peat for the construction of a road.	Unknown.	Up to 5,0 m deep.	Unknown dosage of cement (type of cement unknown).	Unknown.
's Gravenzande, Province of South-Holland, Netherlands (2018)	In-situ mass stabilisation of soft soils for the construction of infrastructure.	1.250 m^3.	Unknown.	Unknown.	Unknown.

2.4 Properties of stabilised soil

By mass stabilising soils, the physical and chemical properties of the soils are altered to create new stabilised materials with the desired physical and/or chemical properties (Building Research Establishment (BRE), 2002). These changes are variable, since the changes depend on the type of binder applied, the applied dosage of binder and the physical and chemical properties of the soil to be stabilised (Building Research Establishment (BRE), 2002). Because of this, literature recommends to carry out laboratory tests and field tests in every project involving the stabilisation of the soil (Building Research Establishment (BRE), 2002). However, some indications on the properties of stabilised soil are also found in literature, which could be used for initial analyses. This section deals with some of the physical properties of soils and how they can be changed by stabilisation.

2.4.1 Strength

Increasing the strength of the soft soil is one of the main purposes of mass stabilisation (see section 2.3.1). The increase in the soil strength may be desired for increasing the stability of constructions or for increasing the bearing capacity of soft soils (Building Research Establishment (BRE), 2002). However, having an indication on the stabilised soil strength and its development are necessary for initial analyses and designs with stabilised soils. The changes in strength and the development of the strength of soils due to stabilisation are therefore discussed in this section.

2.4.1.1 Factors influencing the strength

The strength of stabilised soils depends on the properties of the original soil and the properties of the binder that is used in the stabilisation. Besides these two important factors, the applied mixing procedure, the conditions under which curing of the samples takes place and the manner in which the samples are loaded after curing also affect the strength. Some of these factors are site and project dependent, whereas others are controlled by the contractor in the field (U.S. Department of Transportation, 2013). A detailed list summarising the factors that influence the strength of the stabilised soils is presented in table 2.8. Because of all these factors influencing the strength, it is difficult to predict the strength of stabilised soils.

Table 2.8; Factors that influence the change in soil strength after stabilisation (U.S. Department of Transportation, 2013).

Category	Factors
Characteristics of binder	• Type of binder(s) • Quality of the binder(s) • Mixing water and/or additives or not
Characteristics and conditions of soil (especially important for clays)	• Physical, chemical, and mineralogical properties of the soil • Organic content of the soil • pH of the pore water • Water content of the soil
Mixing conditions	• Amount of binder used or required • Mixing efficiency • Amount of time for mixing and/or remixing
Curing conditions	• Temperature • Curing time • Humidity • Wetting and drying, freezing and thawing, etc.
Loading conditions	• Loading rate • Confining pressure • Stress path (e.g. compression, tension, and simple shear)

2.4.1.2 Achievable strength

The achievable undrained shear strength of the soil samples stabilised in the laboratory is normally 10 to 50 times the undrained shear strength of the undisturbed (natural) soil (Building Research Establishment (BRE), 2002) as shown in equation (2-1). Strengths of soils produced in the laboratory that can be achieved are undrained shear strengths of up to several hundreds of kilopascals (Allu Finland Oy, 2007) or possibly even unconfined compressive strengths of several megapascals (U.S. Department of Transportation, 2013). Although relatively large strengths can be achieved for soil samples stabilised in the laboratory, this may not necessarily be achievable in the field as well. For mass stabilisation, the undrained shear strength reached in the field could be similar or as low as 20 percent of the undrained shear strength as measured in the laboratory (Forsman et al., 2015). This is shown in equation (2-2).

$$S_{u(laboratory)} = (10\ to\ 50) \cdot S_{u(natural\ soil)} \quad \textbf{(2-1)}$$
$$S_{u(stabilised\ in-situ)} = (0{,}2\ to\ 1{,}0) \cdot S_{u(laboratory)} \quad \textbf{(2-2)}$$

where:

$S_{u(laboratory)}$	*- undrained shear strength of soil samples stabilised in the laboratory*	*[kPa]*
$S_{u(natural\ soil)}$	*- undrained shear strength of the natural soil*	*[kPa]*
$S_{u(stabilised\ in-situ)}$	*- undrained shear strength of in-situ stabilised soil*	*[kPa]*

These possible differences in strength between soils stabilised in the laboratory and in the field are caused by differences in the factors listed in table 2.8. The most important factor explaining these differences is the mixing conditions. In the laboratory, a better mixing of the soil with the binder can be achieved, because the mixing process is well controlled and the homogeneity of the mixture can be visually assessed (CUR onderzoekscommissie D34 "Kalk-cementkolommen", 2001). Other important factors influencing the field strength are the natural variation in soils, the possible unequal spread of the binder in the mixed soil and the limited mixing accuracy of the equipment. Therefore the undrained shear strength of soils stabilised in the field is seldom larger than a few hundreds of kilopascal, despite the fact that much larger strengths can be achieved in the laboratory. Typically, undrained shear strengths between 50 and 150 kPa are obtained in the field instead (Allu Finland Oy, 2007).

Furthermore, the drained shear strength parameters of stabilised soil samples can also be rather large. Åhnberg carried out a number of anisotropically consolidated drained (CAD) triaxial tests on different stabilised soils and determined the effective strength parameters of the stabilised soils (Åhnberg, 2006). For most of the triaxial tests, a K_0-value of 0,8 was applied. The derivation of the effective strength parameters was done using data from both 28 days and 32 days after mixing due to a shortage of triaxial setups available to Åhnberg. An overview of Åhnberg's results is presented in table 2.9. It was not mentioned whether the effective strength parameters were determined at peak stress or at a strain level.

Table 2.9; Drained shear strength parameters determined for a number of different stabilised soil samples using drained triaxial tests between 28 and 32 days of curing (Åhnberg, 2006).

Soil type	Applied binder dosage	c' [kPa]	ϕ' [°]
Peat	100 kg/m^3 Portland cement (CEM I 42,5 R) + 100 kg/m^3 blast-furnace slag (GGBS)	137	32
Gyttja (strongly organic clay)	56 kg/m^3 Portland-limestone cement (CEM II/A-LL 42,5 R) + 14 kg/m^3 quicklime (CL90-Q)	137	28
Clay	100 kg/m^3 Portland-limestone cement (CEM II/A-LL 42,5 R)	115	32
Clay	100 kg/m^3 quicklime (CL90-Q)	30	32

Lastly, European Standard EN 14679 (*'Execution of special geotechnical works – deep mixing*)*'* reports that the tensile strength of the stabilised can be assumed equal to 5 to 15 percent of the unconfined compressive strength of the stabilised soil (Technical Committee CEN/TC 288, 2005). However, design manuals from other countries disagree with this. Japanese and American design manuals presuppose that the tensile strength of stabilised soil cannot be relied upon and therefore assume that the stabilised soil has no tensile strength (U.S. Department of Transportation, 2013).

2.4.1.3 Strength development

The strength of the stabilised soil increases with time until the hardening reactions are finished. However, the manner in which the strength increases is dependent on the soil being stabilised and the applied binder (CUR onderzoekscommissie D34 "Kalk-cementkolommen", 2001). In order gain some more insight in the strength development of stabilised soils, tests were done on a variety of stabilised soil samples during the EuroSoilStab project. Different soil types were stabilised with different binders and subsequently tested in unconfined compression tests at different curing times to determine the strength development of these stabilised soil samples (Building Research Establishment (BRE), 2002). Some examples of those measured strength developments are presented in figure 2.6.

The results of the unconfined compression tests from figure 2.6 showed that the strength of most soil-binder mixtures increased logarithmically in time. A major exception to this trend are soils stabilised with a binder blend containing quicklime. Soils stabilised with a binder blend containing quicklime seemed to show either a linear strength increase in time or hardly any increase in strength at all. Furthermore, the duration of the strengthening of the stabilised samples was also found to depend on the binder material used in the stabilisation. When Portland cement was used, the strength of the stabilised soil did not change anymore after 28 days of curing. However, when other binders materials were added to the Portland cement or used instead of the Portland cement, like lime, blast-furnace slag, gypsum or fly ash, it was found that the strength of the soil continued to increase after 28 days of curing up to at least several months thereafter (Building Research Establishment (BRE), 2002).

However, if measurements of the strength development cannot be made, one could also use equation (2-3) to get an estimate of the strength development of stabilised soil. Researchers from Geotechnica SA Incorporated had collected data on measured strength developments for various stabilised soils and found that equation (2-3) was a conservative estimate of the strength development for some of them. The stabilised soils for which equation (2-3) can be used included either fine- or coarse-grained soils stabilised with Portland cement or with a blend of Portland cement and ground-granulated blast-furnace slag (U.S. Department of Transportation, 2013). The equation describing a conservative estimate of the strength development for the aforementioned stabilised soils is also presented graphically in figure 2.7. As evident from equation (2-3), the graph in figure 2.7 shows a logarithmic strength increase in time. This trend matches most of the strength developments observed in figure 2.6.

$$\mathrm{f_c} = 0{,}187 \cdot \ln(\mathrm{t}) + 0{,}375 \qquad \textbf{(2-3)}$$

where:

f_c	*- curing factor, which is the ratio of the UCS at time t to the UCS at 28 days*	*[-]*
t	*- curing time*	*[days]*

If the soil is instead stabilised with a binder which is not either Portland cement or a blend of Portland cement with ground-granulated blast-furnace slag, equation (2-3) cannot be used to make an estimate of the strength development of the stabilised soil. Instead, one can make use of the relative strength increase of various cement mixtures for the preparation of concrete (Mullins & Gunaratne, 2015). An overview of the strength development for some different cement mixtures for concrete are presented in figure 2.8. It is interesting to note that this figure, as well as figure 2.7, shows that Portland cement or a binder blend containing Portland cement will result in a lot of strength being developed during the first 7 days of curing.

Despite the indicative curves and equation (2-3), it cannot be known in advance how the strength develops exactly. This was confirmed by some of the test results from the EuroSoilStab project. The results showed that some peat and organic clay samples stabilised with only Portland cement still had a significant strength development after 28 days of curing, despite the fact that it was expected that no more additional strength would be developed after 28 days of curing (CUR onderzoekscommissie D34 "Kalk-cementkolommen", 2001). This can also be considered a reason why laboratory and field tests are recommended for examining the properties of stabilised soils in every case.

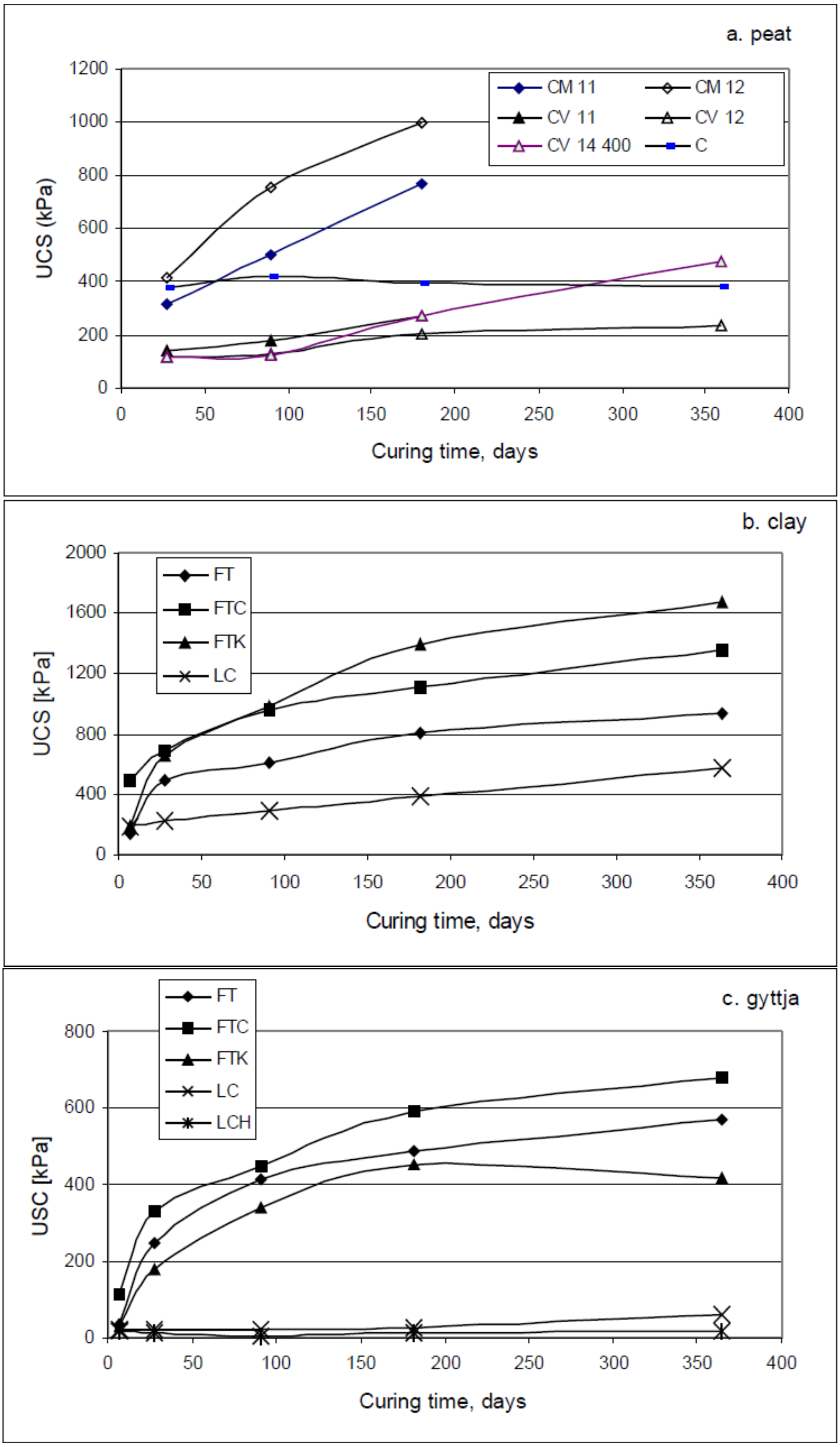

Figure 2.6; Examples of the strength development of soil-binder mixtures measured during the EuroSoilStab project. The applied binder dosage(s) for each of these soils was not mentioned. "C = cement, M = blast furnace slag from Sweden, V = a Swedish fly ash, H = a Finnish fly ash, F = Finnstabi®-gypsum, T = a secondary hydrated lime with at least 50% Ca(OH)₂, L = lime (CaO), K = blast furnace slag from Finland" (Building Research Establishment (BRE), 2002, p. 39).

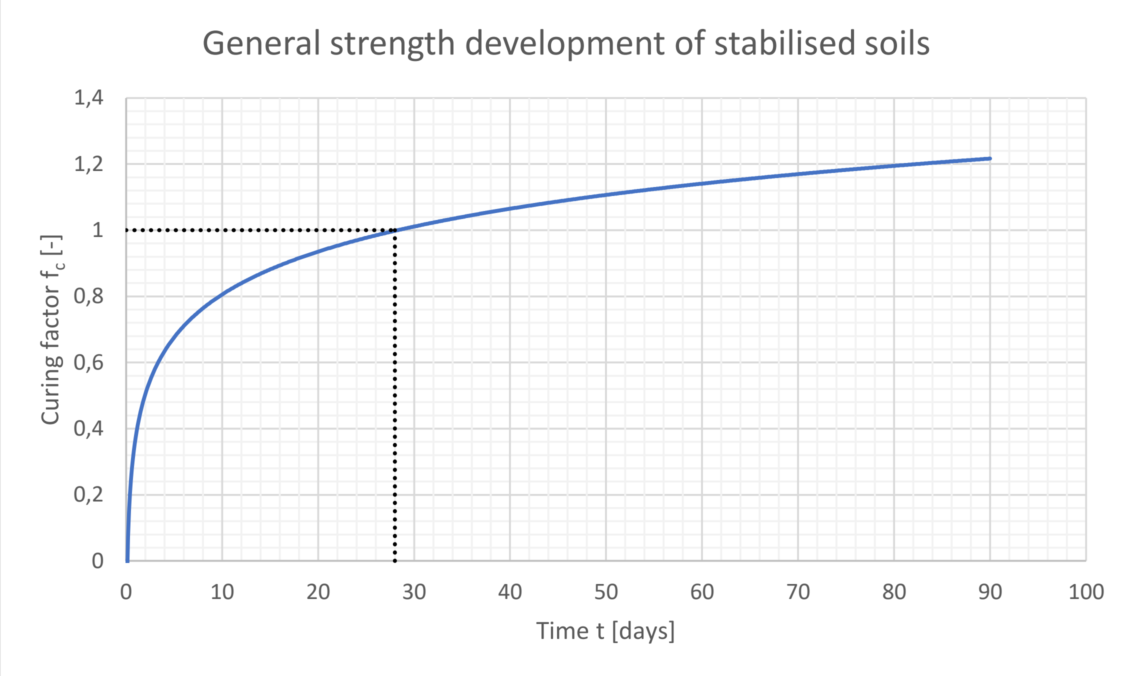

Figure 2.7; Graphical visualisation of equation (2-3), showing the influence of the curing time on the strength development of the soil mixed with either Portland cement or a blend of Portland cement and ground-granulated blast-furnace slag. The dotted line shows the strength at 28 days, when the curing factor is equal to 1,0.

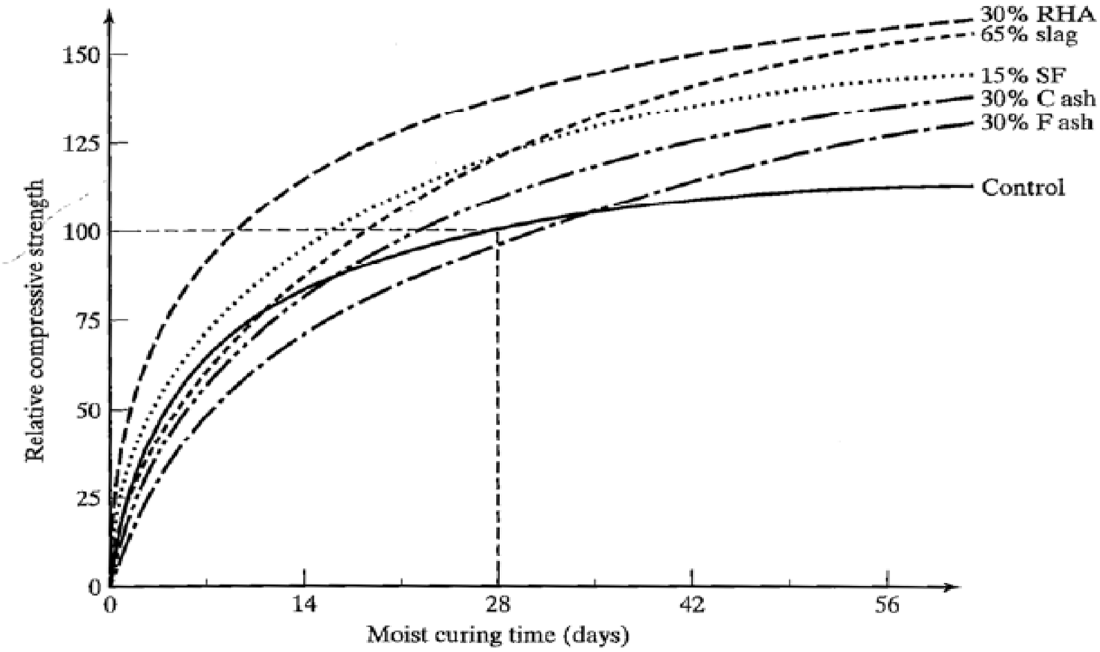

Figure 2.8; Indicative strength development curves for different cement compositions for the production of concrete. The control line is 100% Portland cement. All other lines portray a mixture of Portland cement with the indicated quantity of replaced cement in terms of mass. It was assumed that the same binder dosage and water-to-cement ratio was used for all mixtures (Mullins & Gunaratne, 2015).

Legend:

RHA:	*rice husk ash;*
slag:	*ground-granulated blast-furnace slag;*
SF:	*silica fume;*
C ash:	*class C fly ash;*
F ash:	*class F fly ash.*

2.4.1.4 Variability

Researchers of Geotechnica SA Incorporated also carried out statistical analyses on 7873 unconfined compression test results from 14 datasets for 10 projects involving the stabilisation of soil. The results from the statistical analyses showed that the coefficient of variation (COV) of the unconfined compressive strength of stabilised soils ranged from 0,34 to 0,79 with an average of 0,56. The researchers compared these COV to the COV of the undrained shear strength of naturally occurring clay deposits in the United States, which ranged from 0,13 to 0,40. The researchers concluded that these COV indicated that the strength of stabilised soils is about two times more variable than the strength of naturally occurring clay deposits (U.S. Department of Transportation, 2013).

2.4.2 Unit weight

Swedish researcher Broms reported that the unit weight of inorganic soils stabilised with lime, cement or both was often found to be less than the unit weight of the untreated inorganic soil. Additionally, Broms also reported that the unit weight of organic soils with large water contents that were stabilised with lime, cement or both were found to be larger than the unit weight of the untreated soil (U.S. Department of Transportation, 2013). On the other hand, the Coastal Development Institute of Technology (CDIT) in Japan reported that the unit weight of soils stabilised with cement had increased with about 3 to 15% compared to the unit weight of the untreated soil, whereas the change in unit weight of soil stabilised with lime was found to be negligible. CDIT also reported that the change in unit weight of soils stabilised with wet mixing was found to negligible as well (Coastal Development Institute of Technology (CDIT), Japan, 2002).

Lastly, both the EuroSoilStab design manual and Dutch CUR report 2001-10 *'Deep soil stabilisation in the Netherlands'* mentioned that the characteristic value of the unit weight of mass stabilised soils should be determined using laboratory tests. In the laboratory, samples of soil are required to be stabilised and their unit weights determined, from which a characteristic value can be derived. Additionally, both the EuroSoilStab design manual and Dutch CUR report 2001-10 reported that the characteristic and design value of the unit weight of mass stabilised soil is regarded equal, implying a partial material factor of 1,0 (CUR onderzoekscommissie D34 "Kalk-cementkolommen", 2001).

2.5 Conclusions

A literature study was carried out to collect information required for examining aspects determining the technical feasibility of applying mass stabilisation for reinforcing levees. The literature study focussed on three main categories:

- The mass stabilisation technique itself;
- Current and possible applications of mass stabilisation;
- Properties of stabilised soil.

From literature a lot of information on these three topics was found. Many aspects of mass stabilisation are broadly described in literature and a lot is known about the technique. However, less is known about properties of stabilised soils. Although there are many indications on the undrained shear strength and the unconfined compressive strength in literature, little is found on the effective strength parameters. Although some indications on the effective strength parameters of stabilised soils were found, it was unknown whether those were determined at peak stress or at any specific strain. In addition to this, no information on the mobilisation of the effective strength parameters for stabilised soils was found. Such information is necessary in this research, as the stability of levees assessed with Dutch safety standards requires the effective strength parameters to be determined at specified (small) strains (Stichting Toegepast Onderzoek Waterbeheer, 2015a).

Furthermore, it has also become clear from literature that laboratory research and field tests for examining the properties of stabilised soil are highly recommended. This recommendation is made in many of the sources consulted due to the fact that properties of stabilised soils (in time) cannot be predicted (Building Research Establishment (BRE), 2002). This is because the properties of stabilised soils are dependent on the applied binder, the applied binder dosage and the physical and chemical properties of the soil to be stabilised (CUR onderzoekscommissie D34 "Kalk-cementkolommen", 2001). Even minor differences in these parameters can have a large impact, as rather similar soils or very slight differences between binders may already result in stabilised soils with potentially very different properties (Building Research Establishment (BRE), 2002). Because of this, measurements of properties of soils stabilised at a particular site cannot just be applied in projects involving the stabilisation of soils at another site. After all, it cannot be guaranteed that similar properties of the stabilised soil can be realised at other sites.

As a result of both the lack of information on the mobilisation of the effective strength parameters of stabilised soils and the recommendations for laboratory and field tests, it has been decided to conduct laboratory research in this study. Although field tests are also recommended, as stabilisations in the field may give different results than in the laboratory (Forsman et al., 2015), it was not feasible to carry out such tests due to budget limitations.

In the laboratory, the achievability of the effective strength parameters in compliance with Dutch safety standards is examined. A research question was formulated for the achievability of the strength and the methodology was adapted to include the approach to the laboratory research (see sections 1.2 and 1.3). Using this methodology, the technical feasibility of applying mass stabilisation for reinforcing levees was determined by examining the following three elements in succession in the next chapters:

- The ability of mass stabilisation to solve a stability deficit of a levee;
- The achievability of the effective strength parameters of the stabilised soils in compliance with Dutch safety standards;
- The practicability of the stabilisation of the soil at levees.

3 Design analyses

3.1 Introduction

The technical feasibility of applying mass stabilisation for reinforcing levees is dependent on among others the ability to solve a stability deficit. In order to determine whether mass stabilisation is able to solve the stability deficit, it needs to be determined whether it is possible to sufficiently increase the Factor of Safety of the levee and if so, how this could best be achieved. To examine this, the following sub-questions were formulated:

'Which increases in the Factor of Safety can be realised by stabilising strips of soil at the levee?'

'Where should the stabilisation of the soil at the levee preferably be carried out from an empirical point of view?'

In order to answer these two sub-questions, two-dimensional stability calculations were made for two real Dutch levees reinforced with mass stabilisation at either the toe, slope or crest. The approach to assessing the influence of the separate reinforcement of the soil at the toe, slope and crest on the stability of these levees is described in section 3.2. The results obtained with regard to the achievable increases in the Factor of Safety and the preferred spot for stabilisation are presented in section 3.3 and 3.4 respectively. Conclusions were then drawn from these results in section 3.5 to answer the above sub-questions. Ultimately, one of the examined stabilisations from a single case is chosen in section 3.6 for which the feasibility of the desired strength and the practicability are examined.

3.2 Analyses approach

In this section, the approach to determining the achievable increases in the Factor of Safety with stabilised soils and the preferred spot for stabilisation for two real Dutch levees using two-dimensional stability analyses is presented.

3.2.1 Examined cases

In the stability analyses, two real Dutch 'boezemkades' (i.e. a type of regional flood defence) were theoretically reinforced using mass stabilisation. These two cases were selected for the following reasons:

- The levee in both cases had a stability deficit (i.e. the Factor of Safety against inward macro-instability was too low);
- The levee in both cases consisted of soft soil and was constructed on top of soft soil;
- No obstacles like buildings, roads or trees were near or on top of the levee in both cases.

The two selected cases are introduced in the next two subsections.

3.2.1.1 Montfoortse Vaart

The first case that was selected was the levee at the channel *Montfoortse Vaart* in the Province of Utrecht (the Netherlands) between the cities of Linschoten and Montfoort (see figure 3.1). The levee along the entire *Montfoortse Vaart* falls within the region of Water Board *Hoogheemraadschap De Stichtse Rijnlanden* (HDSR) and is therefore maintained by this Water Board.

At the levee a cross-section was made near Linschoten. A model of this cross-section was made in the software programme D-GeoStability (version 17.1). The model was constructed using the borings and CPTs and the soil parameters and boundary conditions as presented in appendices A and B respectively.

For this cross-section, the Factor of Safety against inward macro-instability was calculated using two models: the Bishop and the Uplift Van calculation model. These Factors of Safety are presented in table 3.1, along with the required Factors of Safety. The derivation of the required Factor of Safety is presented in appendix B. The critical slip surface for this cross-section is shown in figure 3.2.

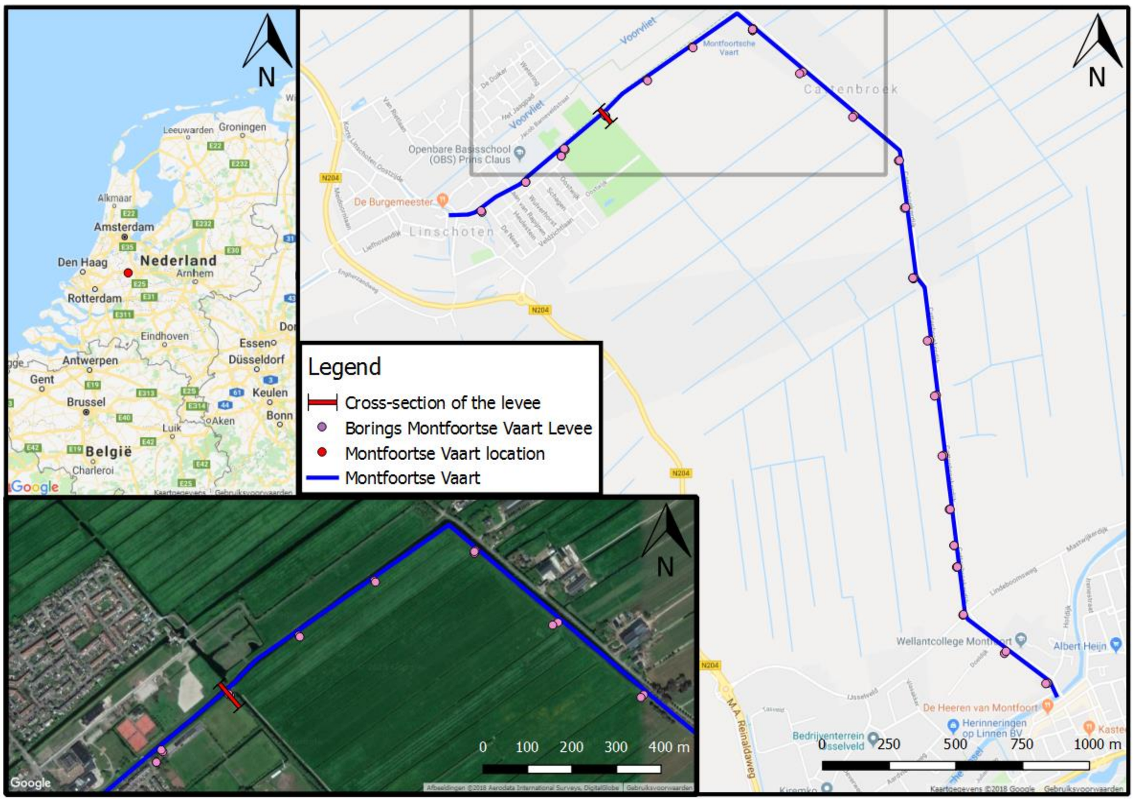

Figure 3.1; Location of the Montfoortse Vaart (Google, 2018a). The purple dots represent the borings carried out along the entire Montfoortse Vaart. The red line shows the location of the cross-section that was taken to make the 2D model in D-GeoStability. The blue line shows the course of the Montfoortse Vaart.

Table 3.1; The calculated Factors of Safety and the required Factors of Safety for the levee at the Montfoortse Vaart.

D-GeoStability model	Current Factor of Safety	Required Factor of Safety
Bishop	0,92	1,02
Uplift Van	0,91	1,07

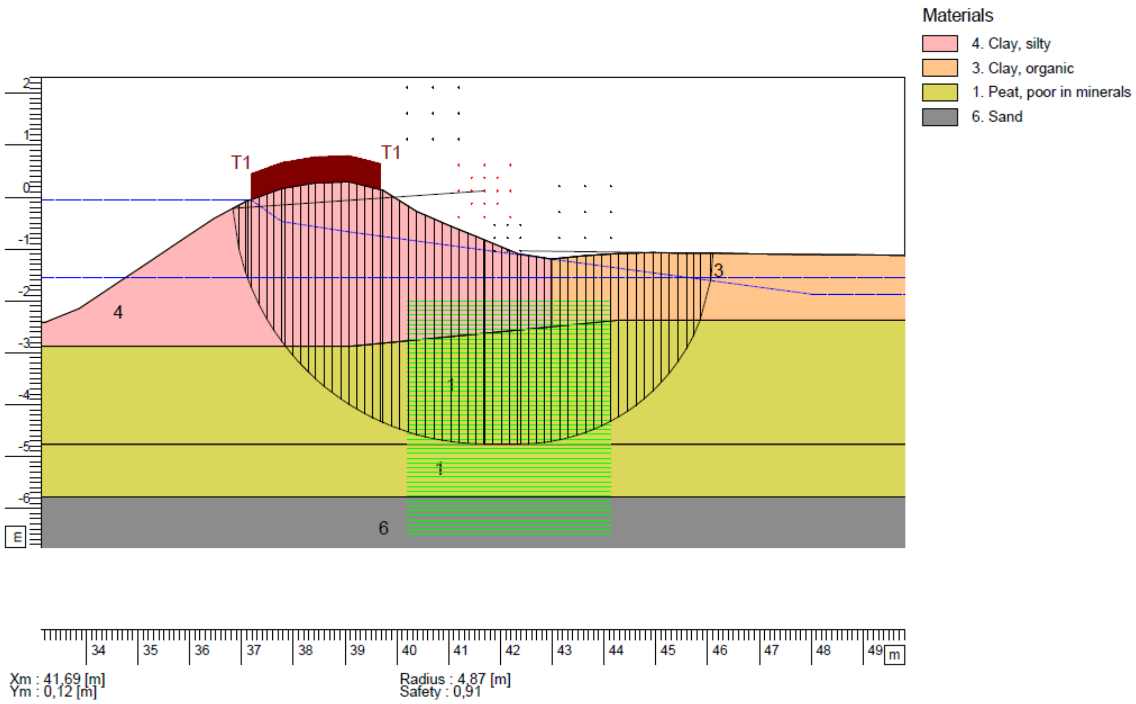

Figure 3.2; Examined cross-section of the levee at the Montfoortse Vaart as built in D-GeoStability. The shown slip surface was calculated by the Uplift Van calculation model, but the Bishop model showed a similar slip surface as well.

Although the Dutch STOWA guideline for the assessment of the safety of regional flood defences prefers the use of the Spencer calculation model, this model was not applied in this research. During initial stability calculations with stabilised soil, the Spencer model often gave unreliable results in the form of unexpectedly low values of the Factor of Safety in combination with unlikely critical slip surfaces. This was probably caused by the model using an algorithm to find the critical slip surface between two user-defined boundaries.

So instead of the Spencer model, the Bishop and the Uplift Van calculation models were applied. The Bishop calculation model always models full circular critical slip surfaces, whereas the Uplift Van calculation model can also model horizontal slip surfaces. Although the Uplift Van model is also able to model fully circular slip surfaces, the Bishop model was also applied as this model was found to be more stable than the Uplift Van model in some cases during this research.

3.2.1.2 Enkele Wiericke

The second case that was selected was the levee at the channel *Enkele Wiericke* in the Province of South-Holland (the Netherlands) and is located east of the city of Reeuwijk (see figure 3.3). The levee at the western side of the Enkele Wiericke falls within the region of Water Board *Hoogheemraadschap van Rijnland*, whereas levee at the eastern side of the Enkele Wiericke falls within the region of Water Board HDSR. Therefore the levee at the western side is maintained by Water Board *Hoogheemraadschap van Rijnland*, whereas the levee at the eastern side is maintained by Water Board HDSR.

About halfway through the levee on the western side of the Enkele Wiericke a cross-section was made. A model of this cross-section was made in the software programme D-GeoStability (version 17.1). The model was constructed using the borings and CPTs and the soil parameters and boundary conditions as presented in appendices A and B respectively.

For this cross-section, the Factor of Safety against inward macro-instability was calculated using two models: the Bishop and the Uplift Van calculation model. These Factors of Safety are presented in table 3.2, along with the required Factors of Safety. The derivation of the required Factor of Safety is presented in appendix B. The critical slip surface for this cross-section is shown in figure 3.4.

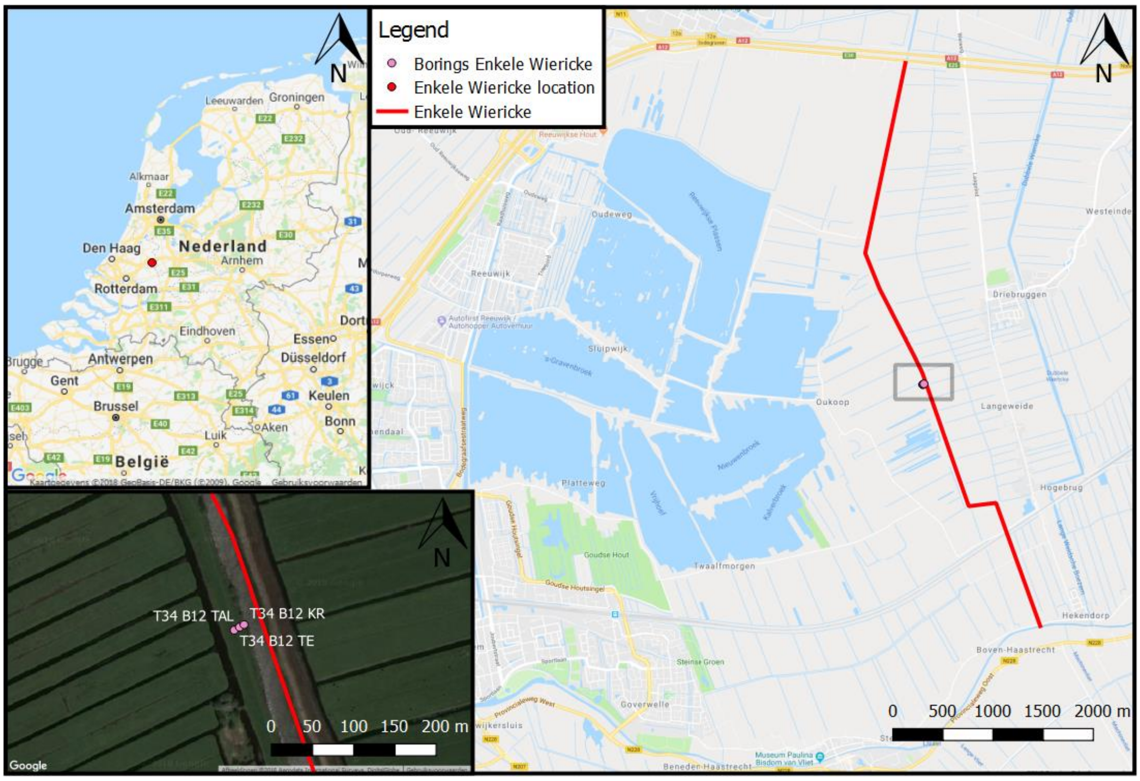

Figure 3.3; Location of the Enkele Wiericke levee (Google, 2018b). The course of the Enkele Wiericke is highlighted in red, with the purple dots highlighting the borings that were made.

Table 3.2; The calculated Factors of Safety and the required Factors of Safety for the levee at the Enkele Wiericke in the initial situation.

Model D-GeoStability	Current Factor of Safety	Required Factor of Safety
Bishop	0,96	1,08
Uplift Van	0,95	1,13

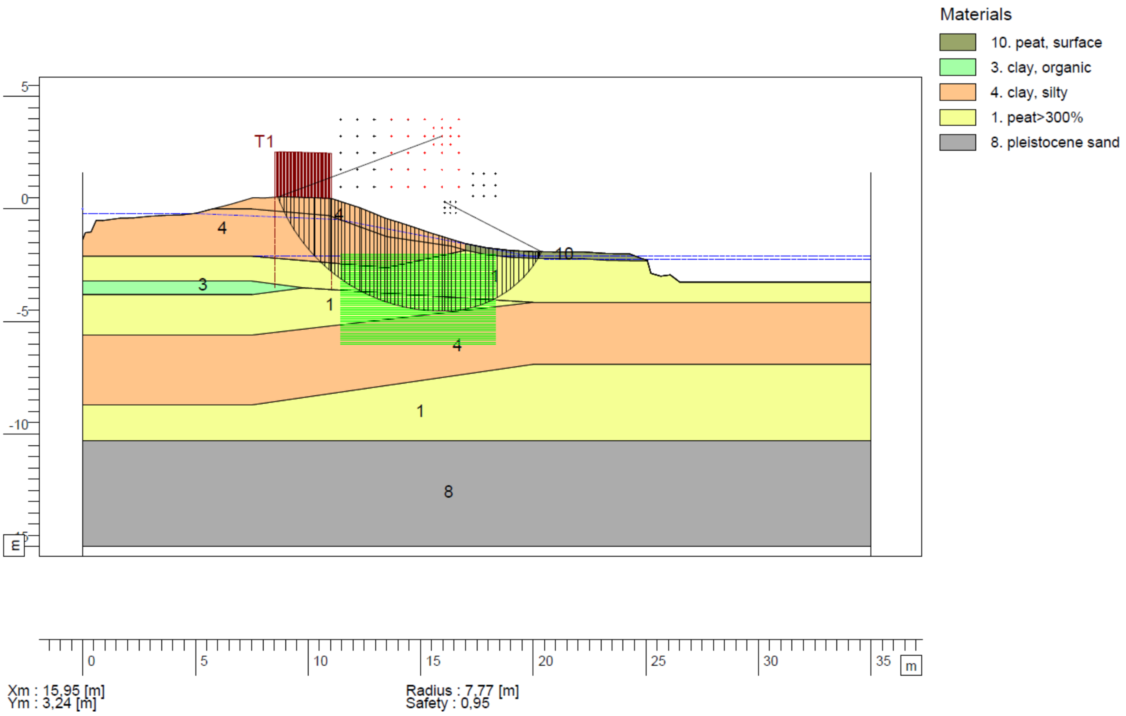

Figure 3.4; Examined cross-section of the levee at the Enkele Wiericke as built in D-GeoStability. The shown slip surface was calculated by the Uplift Van calculation model, but the Bishop model showed a similar slip surface as well.

3.2.2 Stabilised soil geometry selection

Both levees are theoretically reinforced by stabilising a strip of soil at three different spots at the levee: at the toe, at the slope and at the crest. Since a two-dimensional cross-section will be examined, the strip will look like a block and will therefore be called a block when referring to a cross-section. For the purposes of this research, the dimensions of the blocks of stabilised soil were preselected for each spot of the levee. Please note that the dimensions of the stabilised soil blocks are different per spot at the levee and per case.

At the levee of the Montfoortse Vaart, the following dimensions of the stabilised soil block were modelled:

- Toe: 5,0 metres wide and approximately 4,5 metres deep (see figure 3.5);
- Slope: 3,0 metres wide and between about 4,5 and 5,8 metres deep (see figure 3.6);
- Crest: 3,0 metres wide and approximately 5,8 metres deep (see figure 3.7).

At the levee of the Enkele Wiericke, the following dimensions of the block of stabilised soil were selected:

- Toe: 6,0 metres wide and approximately 4,6 metres deep (see figure 3.8);
- Slope: 5,0 metres wide and between about 4,5 and 6,2 metres deep (see figure 3.9);
- Crest: 5,0 metres wide and approximately 3,8 metres deep (see figure 3.10).

Please note that a small layer of remoulded soil was modelled around the block of stabilised soil. According to Dutch report CUR 2001-10 *'Deep soil stabilisation in the Netherlands'*, it is possible that upon in-situ mixing a small zone of soil underneath to the stabilisation zone gets remoulded. The report mentioned that for a similar in-situ mixing technique 0,5 metre of soil is commonly assumed to be remoulded (CUR onderzoekscommissie D34 "Kalk-cementkolommen", 2001). However, since remoulding underneath the block is commonly assumed, it isn't unlikely to assume this is the case at the sides of the block too. Hence, a layer of remoulded soil of 0,5 metre of soil was also modelled at the sides of the block.

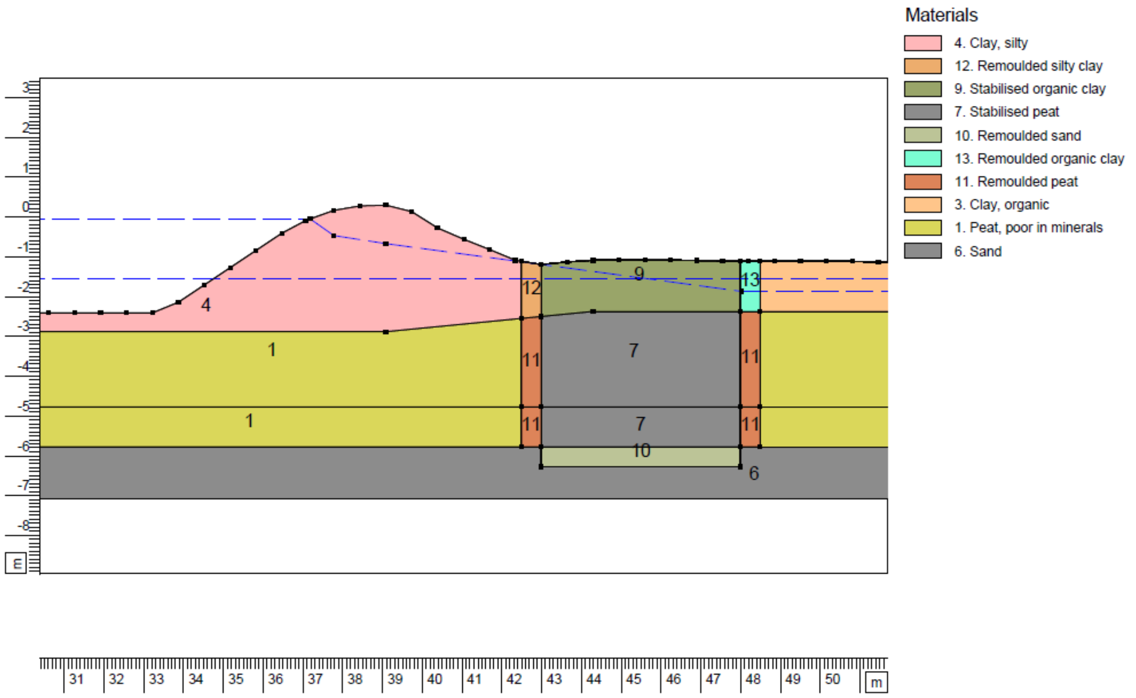

Figure 3.5; The location of the 5,0 metre wide and about 4,5 metre deep stabilised soil block at the toe of the levee of the Montfoortse Vaart.

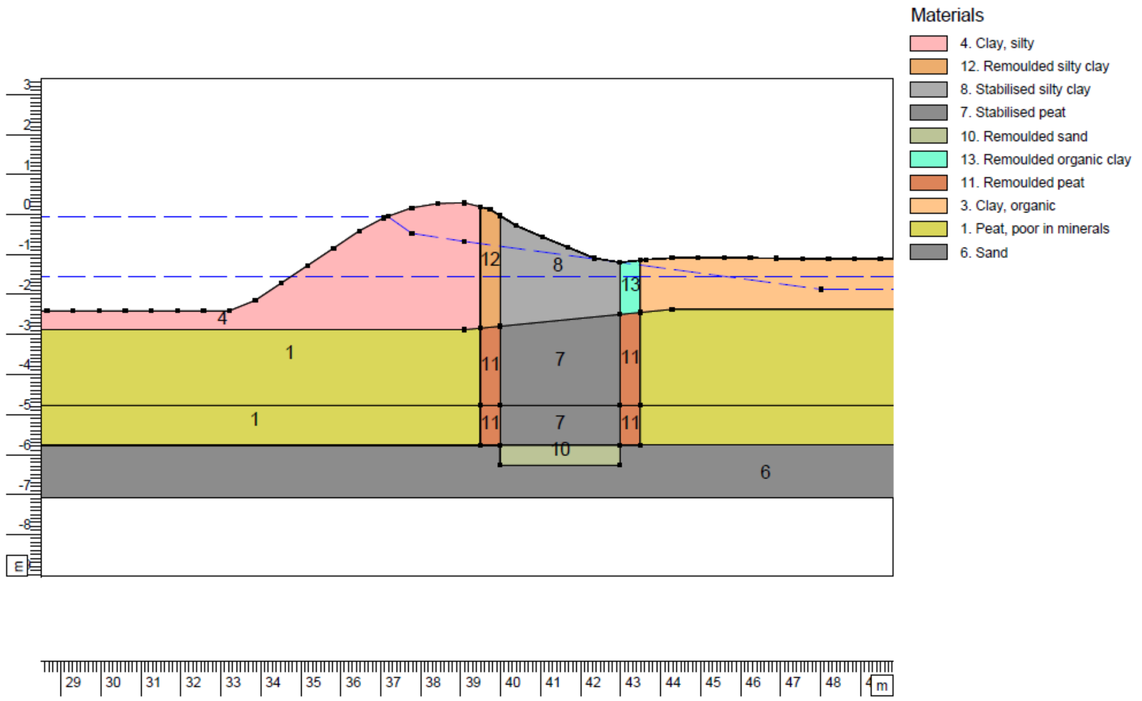

Figure 3.6; The location of the 3,0 metre wide and between about 4,5 and 5,8 metre deep stabilised soil block in the slope of the levee of the Montfoortse Vaart.

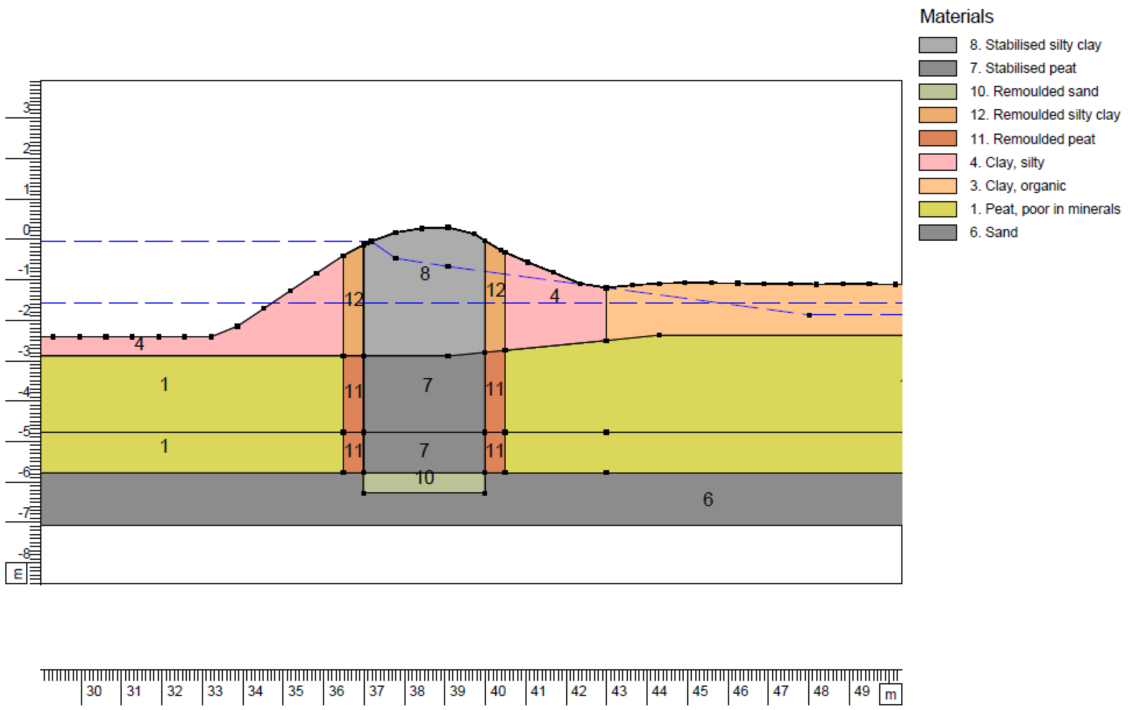

Figure 3.7; The location of the 3,0 metre wide and about 5,8 metre deep stabilised soil block at the crest of the levee of the Montfoortse Vaart.

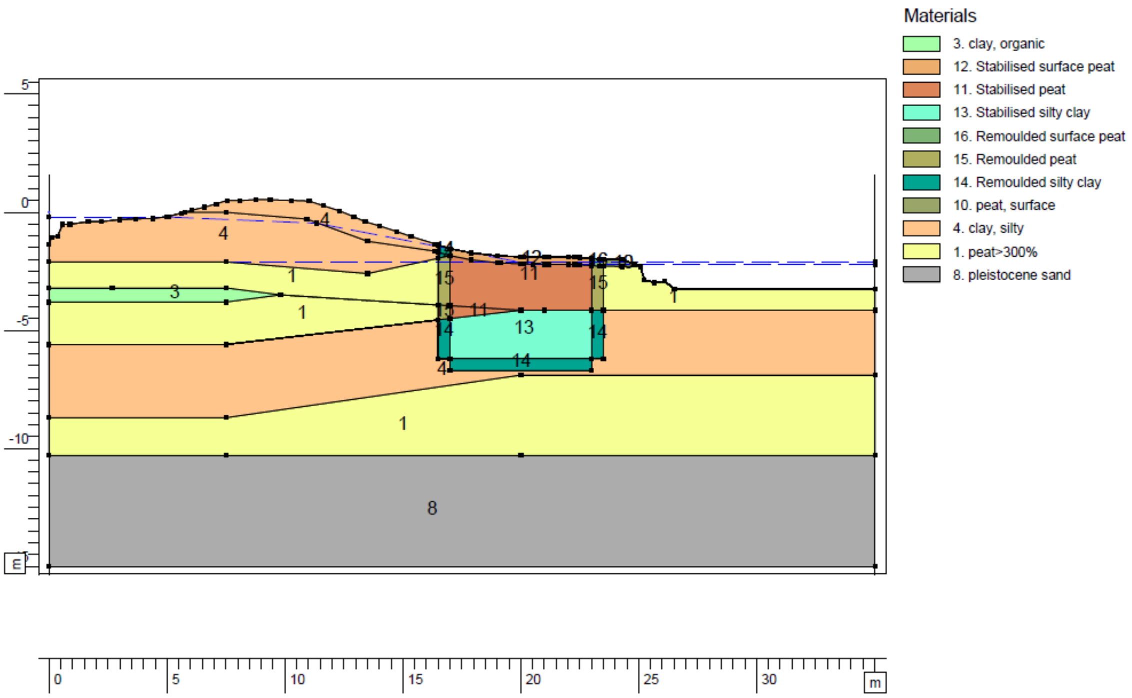

Figure 3.8; The location of the 6,0 metre wide and about 4,6 metre deep stabilised soil block at the toe of the levee of the Enkele Wiericke.

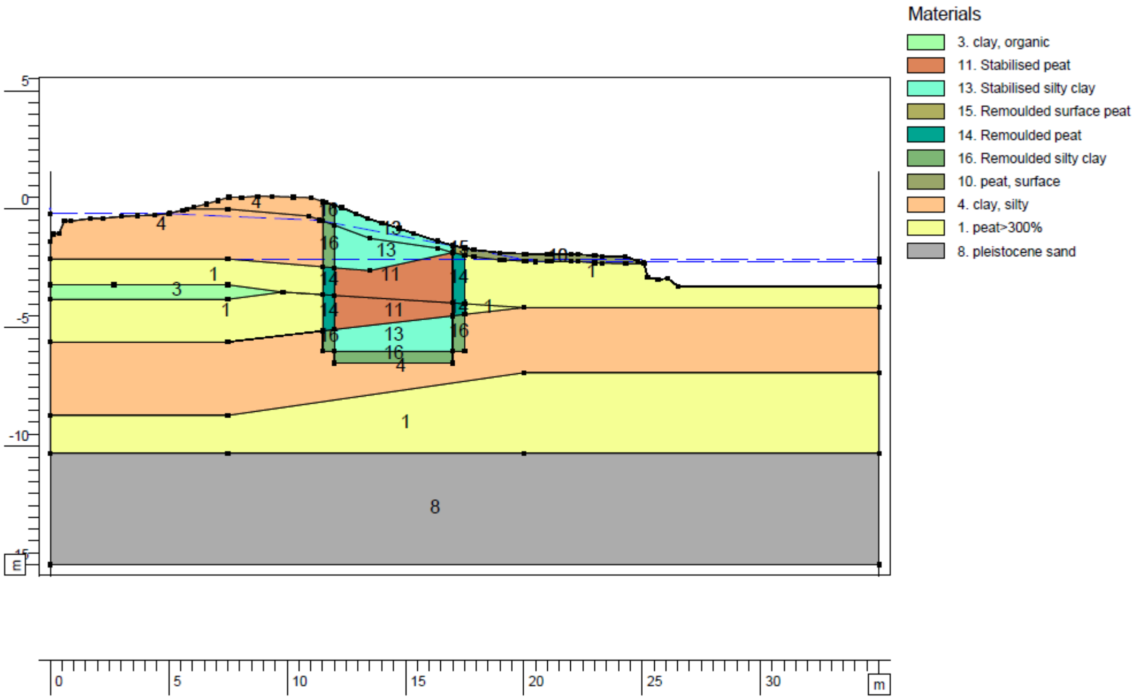

Figure 3.9; The location of the 5,0 metre wide and between about 4,5 and 6,2 metre deep stabilised soil block in the slope of the levee of the Enkele Wiericke.

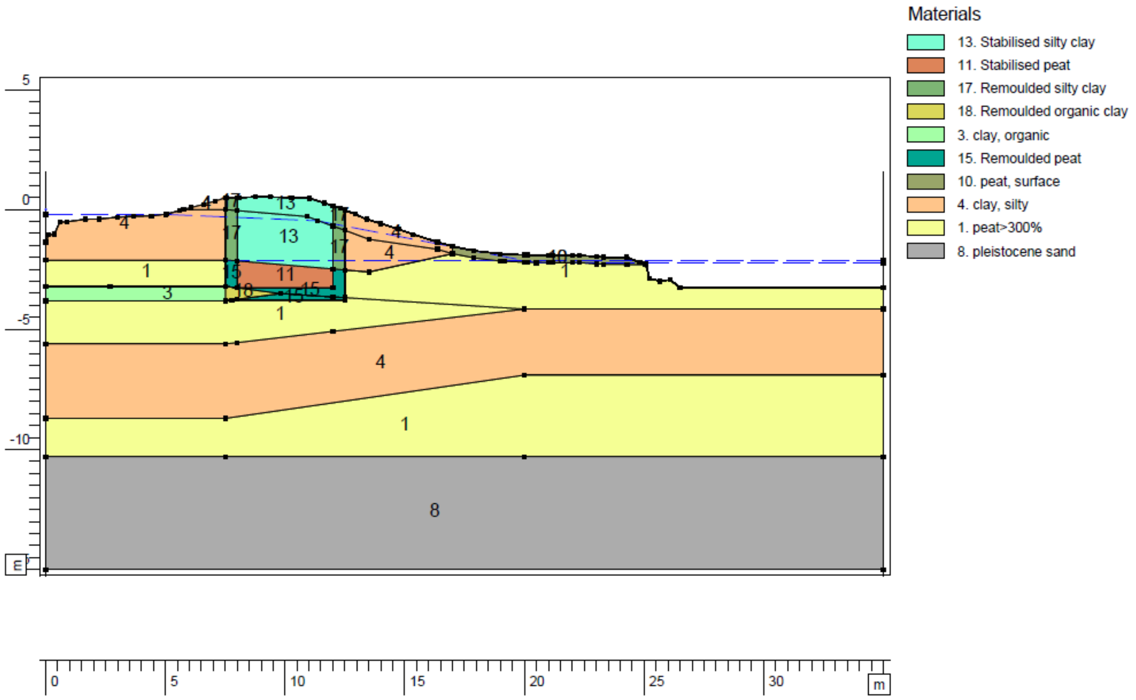

Figure 3.10; The location of the 5,0 metre wide and about 3,8 metre deep stabilised soil block in the crest of the levee.

3.2.3 Stability analyses

In order to assess the ability of mass stabilisation to solve a stability deficit, a number of two-dimensional stability analyses were carried out. In accordance with the Dutch STOWA guideline for the assessment of the safety of regional flood defences, the stability of the levees was determined by means of drained stability calculations with effective strength parameters. The stability of the reinforced levees was assessed with both the Bishop and Uplift Van calculation model. Although the required Factor of Safety for the Uplift Van calculation model is larger (see table 3.1 and table 3.2), the Bishop calculation model was also applied to check the results of the Uplift Van calculation model (in particular the slip surface).

In order to obtain an indication on the increases in the Factor of Safety against inward macro-stability that could be achieved at the two examined levees by stabilising the blocks of soil listed in section 3.2.2, two separate two-dimensional stability analyses were made per examined block of stabilised soil. In both stability analyses, the same design values of the strengths, but different design values of the unit weight were assigned to the stabilised soil layers. The soil parameters that were applied in these stability analyses for the two examined levees are presented in respectively table 3.3 and table 3.4. The soil parameters of the undisturbed soil layers for both cases were obtained from appendix B.

Table 3.3; The applied design values of the soil parameters for the stability analyses of the levee at the Mont. Vaart.

Soil type (Montfoortse Vaart)	Wet unit weight	Saturated unit weight	Effective cohesion	Effective angle of internal friction
	$\gamma_{wet;d}$	$\gamma_{sat;d}$	c'_d	ϕ'_d
	[kN/m³]	[kN/m³]	[kPa]	[°]
Peat, poor in minerals	10,00	10,00	0,67	12,7
Organic clay	12,80	12,80	0,83	31,1
Silty clay	15,95	15,95	2,01	27,3
Sand	19,00	21,00	0,00	29,0
Remoulded peat	10,00	10,00	0,27	5,2
Remoulded organic clay	12,80	12,80	0,33	13,6
Rem. silty clay	15,95	15,95	0,80	11,7
Remoulded sand	17,00	19,00	0,00	26,7
Stabilised peat	10,00 (+10%: 11,00)	10,00 (+10%: 11,00)	5,00	35,0
Stabilised organic clay	12,80 (+10%: 14,10)	12,80 (+10%: 14,10)	5,00	35,0
Stabilised silty clay	15,95 (+10%: 17,55)	15,95 (+10%: 17,55)	5,00	35,0

Table 3.4; The applied design values of the soil parameters for the stability analyses of the levee at the Enkele Wiericke.

Soil type (Enkele Wiericke)	Wet unit weight	Saturated unit weight	Effective cohesion	Effective angle of internal friction
	$\gamma_{wet;d}$	$\gamma_{sat;d}$	c'_d	ϕ'_d
	[kN/m³]	[kN/m³]	[kPa]	[°]
Peat, poor in minerals, surface	12,50	12,50	2,00	20,0
Peat, poor in minerals	10,30	10,30	2,00	20,0
Silty clay	15,40	15,40	2,80	26,8
Organic clay	13,30	13,30	1,40	25,5
Pleistocene sand	18,00	20,00	0,00	32,5
Remoulded surface peat	12,50	12,50	0,80	8,3
Remoulded peat	10,30	10,30	0,80	8,3
Remoulded silty clay	15,40	15,40	1,12	11,4
Remoulded organic clay	13,30	13,30	0,56	10,8
Stabilised surface peat	12,50 (+10%: 13,80)	12,50 (+10%: 13,80)	5,00	35,0
Stabilised peat	10,30 (+10%: 11,30)	10,30 (+10%: 11,30)	5,00	35,0
Stabilised silty clay	15,40 (+10%: 16,90)	15,40 (+10%: 16,90)	5,00	35,0

For these analyses a strength reduction of 60% was applied for all remoulded cohesive soils on both effective strength parameters (i.e. c' and $\tan(\phi')$). This strength reduction was based on the strength reduction for remoulded soils from the Dutch technical guideline for macro-stability (Zwanenburg, van Duinen, & Rozing, 2013). On the other hand, it was assumed that upon remoulding the medium packed sand at the Montfoortse Vaart would become a loosely packed sand with soil parameters based on a loose sand from table 2.b of Dutch standard NEN 9997-1 (Normcommissie 351 006 "Geotechniek", 2017).

For all stability analyses, an effective cohesion of 5,0 kPa and an effective angle of internal friction of 35° was assigned to all stabilised soil layers to model a homogenous stabilisation (see table 3.3 and table 3.4). These design values were synthetic choices. These values were selected with the purpose of increasing the strength of the soils, but without increasing the strength to such high levels that a material with properties similar to a 'block of concrete' is obtained. Also given the relatively large variations in strength recorded in literature (see section 2.4.1.4), it was decided to set the design value of the cohesion to 5,0 kPa to prevent that the mean value of the cohesion would increase to undesirably high levels.

Lastly, stability analyses were carried out with two different unit weights of the stabilised soils: equal to the undisturbed soils and equal to a 10% increase in the unit weight of the undisturbed soils. According to literature, it is not unlikely to have a 10% increase in the unit weight, but it is also possible that there is a negligible change in the unit weight due to stabilisation (see section 2.4.2). As a result, two stability analyses had to be made per examined block of stabilised soil to assess the increase in the Factor of Safety under these two conditions. The results of these stability analyses are presented in the next section. Using these results, the preferred spot for stabilisation at both levees was determined in section 3.4.

3.3 Achieved increases in the Factor of Safety

Stability analyses were carried out for each of the two examined levees which were reinforced with the two-dimensional blocks of stabilised soil listed in section 3.2.2. For each block of stabilised soil, two stability analyses were carried out: one without an increase in the unit weight and one with an increase in the unit weight. The obtained results for each of the levees is presented in the next subsections.

3.3.1 Montfoortse Vaart

The Factors of Safety that were obtained from the two-dimensional stability calculations with and without the unit weight increase for each of the examined blocks of stabilised soil at the levee at the Montfoortse Vaart are presented in table 3.5. The critical slip surface before and after reinforcement with stabilised soil without an increase in the unit weight at respectively the toe, slope and crest are presented in figure 3.11, figure 3.12 and figure 3.13. The comparison of the critical slip surfaces after reinforcement with stabilised soil with and without an increase in the unit weight at respectively the toe, slope and crest are presented in figure 3.14, figure 3.15 and figure 3.16. All presented critical slip surfaces were determined using the Uplift Van calculation model, unless major differences in the Factor of Safety or critical slip surface as determined by the Bishop and Uplift Van calculation model were obtained.

Table 3.5; Achieved increases in the Factor of Safety for each of the examined blocks of stabilised soil at the levee of the Montfoortse Vaart. The numbers in black represent the Factor of Safety of the initial situation, whereas numbers indicated in respectively green and red do and do not meet the required Factor of Safety.

Montfoortse Vaart	Toe	Slope	Crest
No change in unit weight	*Bishop:* 0,92 -> 1,24 (+35%) *Uplift Van:* 0,91 -> 1,10 (+21%)	*Bishop:* 0,92 -> 1,25 (+36%) *Uplift Van:* 0,91 -> 1,23 (+35%)	*Bishop:* 0,92 -> 1,36 (+48%) *Uplift Van:* 0,91 -> 1,33 (+46%)
10% increase in unit weight	*Bishop:* 0,92 -> 1,39 (+51%) *Uplift Van:* 0,91 -> 1,24 (+36%)	*Bishop:* 0,92 -> 1,36 (+48%) *Uplift Van:* 0,91 -> 1,34 (+47%)	*Bishop:* 0,92 -> 1,31 (+42%) *Uplift Van:* 0,91 -> 1,31 (+44%)

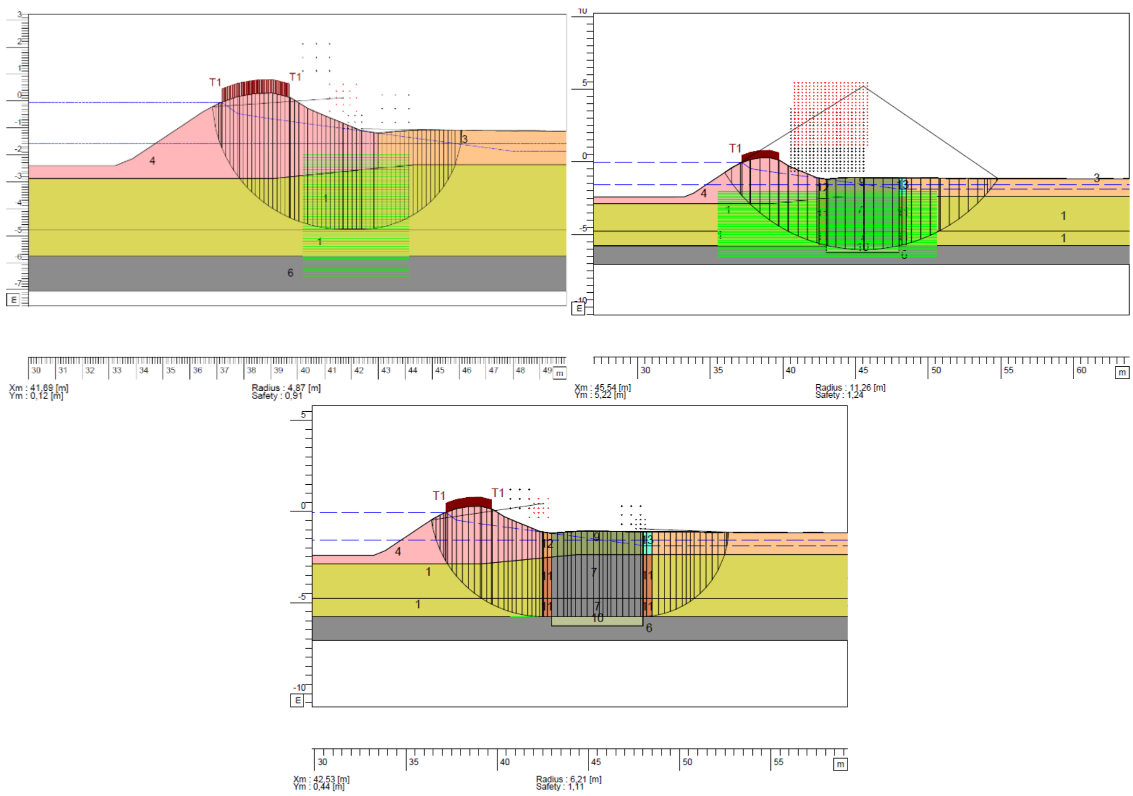

Figure 3.11; The critical slip surface of the initial situation (upper left image: Uplift Van) and of the final situation with a block of stabilised soil at the toe without a unit weight increase (upper right image: Bishop; lower image: Uplift Van).

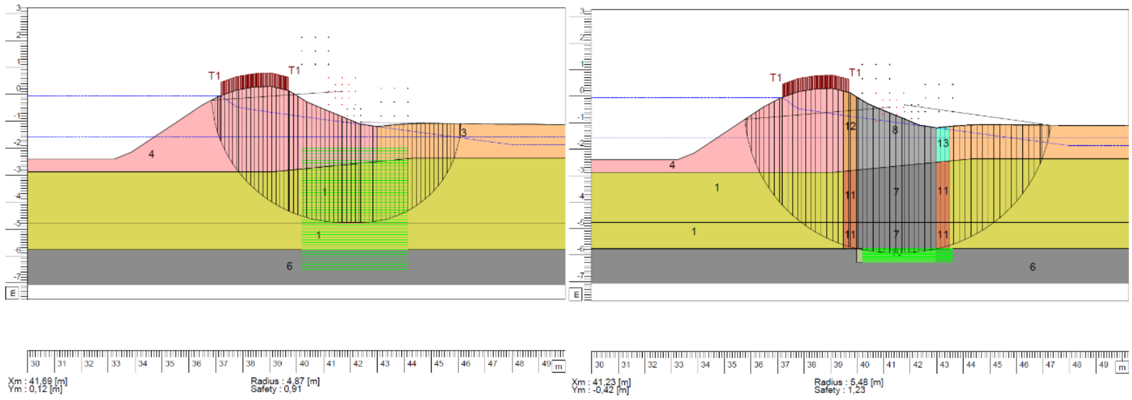

Figure 3.12; The critical slip surface of the initial situation (left image: Uplift Van) and of the final situation with a block of stabilised soil at the slope without a unit weight increase (right image: Uplift Van).

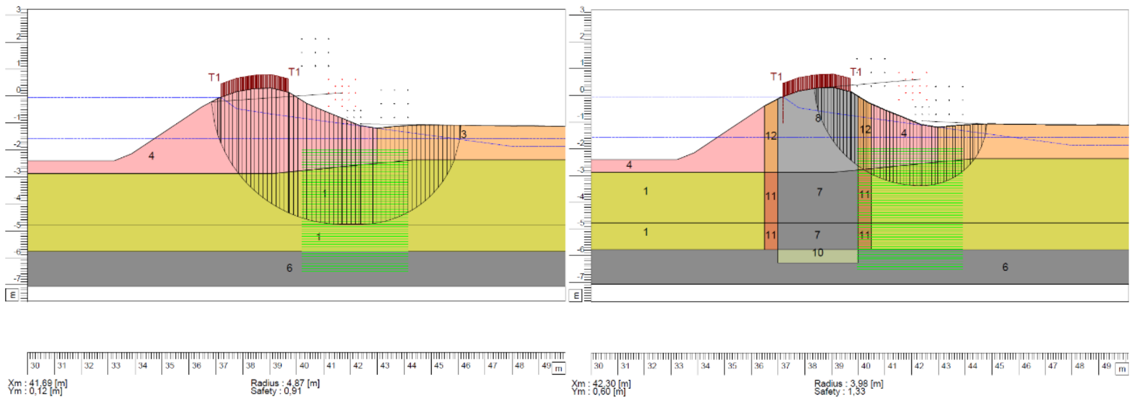

Figure 3.13; The critical slip surface of the initial situation (left image: Uplift Van) and of the final situation with a block of stabilised soil at the crest without a unit weight increase (right image: Uplift Van).

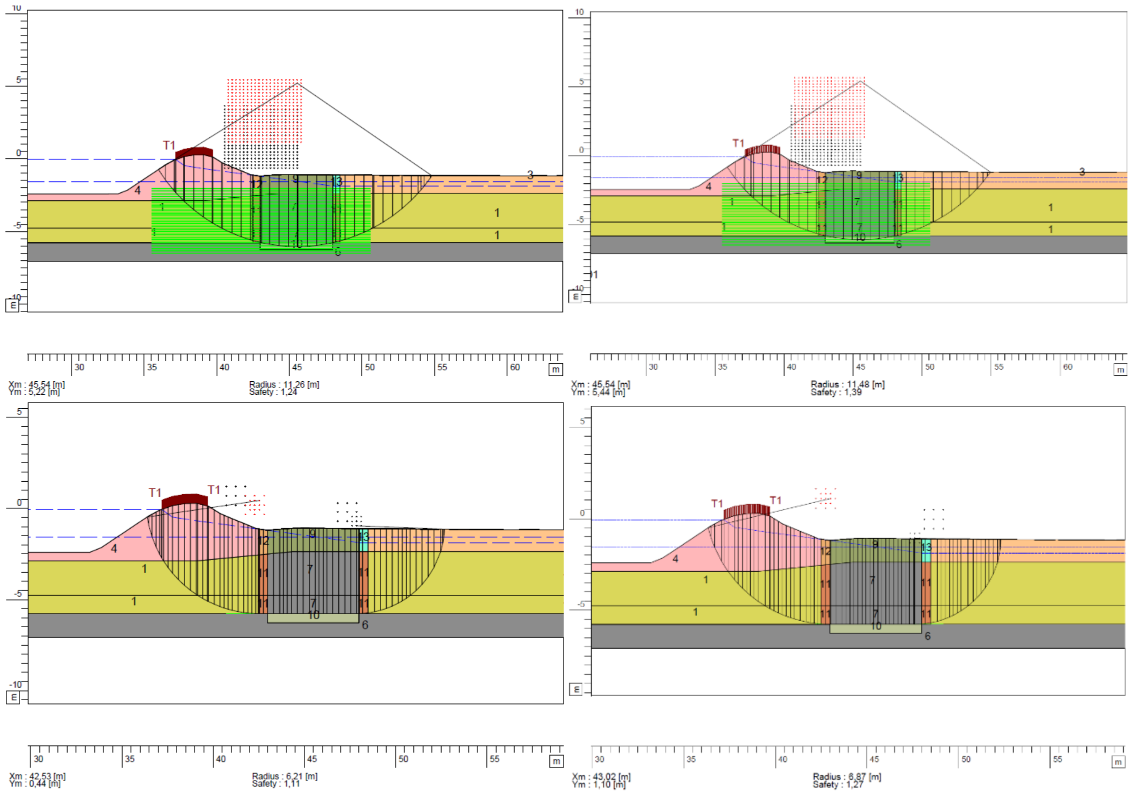

Figure 3.14; The critical slip surface of the final situation with a block of stabilised soil at the toe without a unit weight increase (upper left image: Bishop; lower left image: Uplift Van) and with a unit weight increase (upper right image: Bishop; lower right image: Uplift Van).

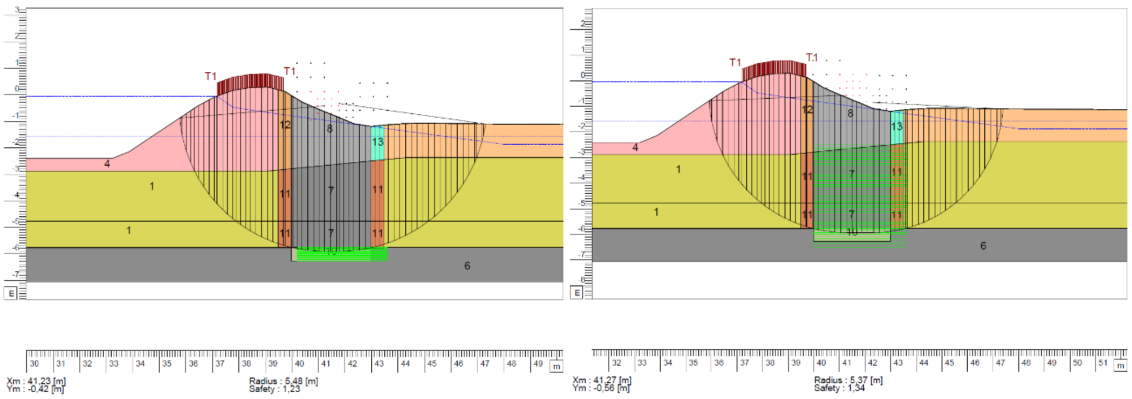

Figure 3.15; The critical slip surface of the final situation with a block of stabilised soil at the slope without a unit weight increase (left image: Uplift Van) and with a unit weight increase (right image: Uplift Van).

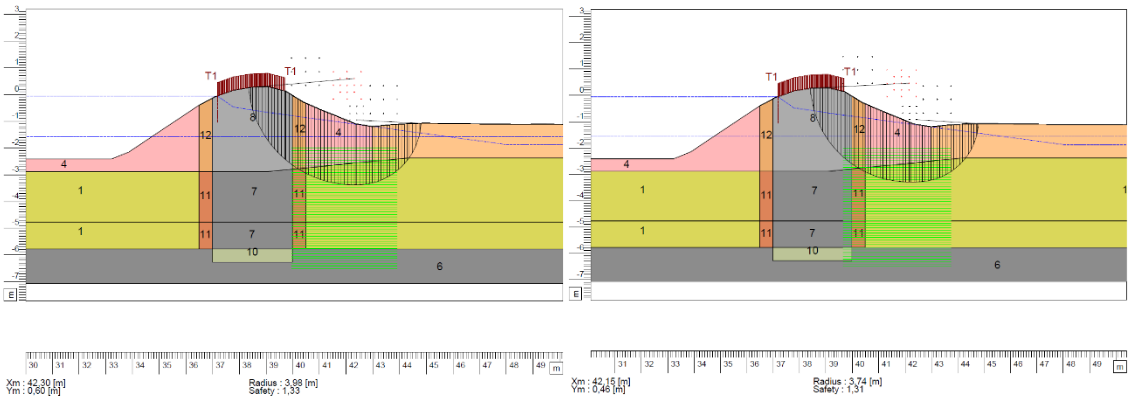

Figure 3.16; The critical slip surface of the final situation with a block of stabilised soil at the crest without a unit weight increase (left image: Uplift Van) and with a unit weight increase (right image: Uplift Van).

Upon inspection of table 3.5, it can be seen that increases in the Factor of Safety between 21 and 47% were achieved when comparing the normative Factors of Safety for each stabilisation spot at the levee. The largest increases in the Factor of Safety were achieved by stabilising soil at the crest of the levee under the assumption that the unit weight does not change. However, if a 10% increase in the unit weight is assumed instead, the largest increases in the Factor of Safety were achieved by stabilising soil at the slope.

When subsequently comparing the calculated Factors of Safety by both calculation models, it can be seen that similar values were found for the stabilisations at the crest and slope, but also that different values were found for the stabilisation at the toe. These results indicate that circular slip surfaces are normative at the crest and slope, which is evident from figure 3.15 and figure 3.16. However, these results also indicate that horizontal slip surfaces are normative for the stabilisation at the toe: the Factor of Safety as calculated with the Uplift Van calculation model is smaller than calculated with the Bishop calculation model. This was found because the circular slip surface passed through the sand layer, thereby generating more resistance against shearing than the horizontal slip surface which didn't pass through the sand layer (see figure 3.14).

Aside from this, it can be seen from table 3.5 that increases in the unit weight result in increases in the Factor of Safety for the levee reinforced with stabilised soil at the toe and the slope, but in decreases for the levee reinforced with stabilised soil at the crest. This is the result of increasing the mass at either the driving side (i.e. crest) or the resisting side (i.e. slope and toe) of the critical slip surface. When subsequently comparing the critical slip surfaces, it can be seen from figure 3.14, figure 3.15 and figure 3.16 that increases in the unit weight hardly cause a change in shape or position of the critical slip surfaces. Clearly, increases in the unit weight do not result in new paths of least resistance at this levee.

Lastly, the maximum Factors of Safety against inward macro-instability seem to have been reached for the stabilisations at the toe and the slope, but not for the stabilisation at the crest. Upon inspection of figure 3.14, figure 3.15 and figure 3.16, it can be seen that the critical slip surface has completely avoided the blocks of stabilised soil at the toe and the slope, but not at the crest. This indicates that further strengthening of the examined blocks of stabilised soil at the toe and the slope will not yield additional increases in the Factor of Safety, unless the unit weight is increased further. Since the main purpose of mass stabilisation is to increase the strength and not to increase the unit weight, further strengthening of these blocks of soil is pointless. At the crest on the other hand, the critical slip surface still passed through the stabilised soil at the modelled strength. Here, additional strengthening will yield additional increases in the Factor of Safety. Even though this is possible, it needs to be determined whether this would be desirable. After all, increasing the design value of the effective cohesion any further may result in a too large mean value of the effective cohesion. In the field, this would mean that a material with properties similar to a 'block of concrete' would be created, which is not desirable at levees. Why this is undesirable at levees is described in section 7.1.2.

3.3.2 Enkele Wiericke

The Factors of Safety that were obtained from the two-dimensional stability calculations with and without the unit weight increase for each of the examined blocks of stabilised soil at the levee at the Enkele Wiericke are presented in table 3.6. The critical slip surface before and after reinforcement with stabilised soil without an increase in the unit weight at respectively the toe, slope and crest are presented in figure 3.17, figure 3.18 and figure 3.19. The comparison of the critical slip surfaces after reinforcement with stabilised soil with and without an increase in the unit weight at respectively the toe, slope and crest are presented in figure 3.20, figure 3.21 and figure 3.22. All presented critical slip surfaces were determined using the Uplift Van calculation model, unless major differences in the Factor of Safety or critical slip surface as determined by the Bishop and Uplift Van calculation model were obtained.

Upon inspection of table 3.6, it can be seen that increases in the Factor of Safety between 7 and 32% were achieved when comparing the normative Factors of Safety for each stabilisation spot at the levee. The largest increases in the Factor of Safety were achieved by stabilising soil at both the toe and the slope of the levee, regardless of the assumption on the unit weight.

Table 3.6; Achieved increases in the Factor of Safety for each of the examined blocks of stabilised soil at the levee of the Enkele Wiericke. The numbers in black represent the Factor of Safety of the initial situation. Numbers indicated in green meet the required Factor of Safety, whereas numbers indicated in red do not meet the required Factor of Safety.

Enkele Wiericke	Toe	Slope	Crest
No change in unit weight	*Bishop:* 0,96 -> 1,21 (+26%) *Uplift Van:* 0,95 -> 1,19 (+25%)	*Bishop:* 0,96 -> 1,19 (+24%) *Uplift Van:* 0,95 -> 1,19 (+25%)	*Bishop:* 0,96 -> 1,05 (+9%) *Uplift Van:* 0,95 -> 1,05 (+11%)
10% increase in unit weight	*Bishop:* 0,96 -> 1,27 (+32%) *Uplift Van:* 0,95 -> 1,26 (+33%)	*Bishop:* 0,96 -> 1,27 (+32%) *Uplift Van:* 0,95 -> 1,25 (+32%)	*Bishop:* 0,96 -> 1,03 (+7%) *Uplift Van:* 0,95 -> 1,02 (+7%)

An important result of the stability analyses for this case is that it was not possible to sufficiently increase the Factor of Safety of the levee with the examined block of stabilised soil at the crest with the modelled strength. The reason why the modelled strength was insufficient is due to the soil profile at this levee. At this levee, the weakest soil layer is the peat layer. In the initial situation, a large portion of the critical slip surface passed through this layer underneath the slope and at the toe as shown in figure 3.17. As a result, most of the additional resistance against shearing could be gained by stabilising this layer, which could only be done effectively at the slope and the toe. Therefore stabilising soil at the crest, where the critical slip surface did not pass through the peat layer, will not lead to big increases in the Factor of Safety. So clearly, the soil profile determines to a great extent where stabilisation at the levee is most effective.

Apart from this, it can also be seen from table 3.6 that the Bishop and the Uplift Van calculation models give similar Factors of Safety at all examined spots. Although this would indicate that a circular slip surface is normative for all three reinforcements, this was not necessarily the case at the toe. When examining figure 3.17 it can be seen that the slip surface as determined with the Uplift Van model is horizontal. Since the Factor of Safety calculated with the Uplift Van model is slightly lower than calculated with the Bishop calculation model, this theoretically implies that the horizontal slip surface is normative at the toe.

Furthermore, it was also found that increases in the unit weight of the stabilised soil at the crest resulted in reductions in the Factor of Safety, whereas increases in the unit weight of the stabilised soil at the slope and toe resulted in increases in the Factor of Safety. This is the result of increasing the mass at either the driving side (i.e. crest) or the resisting side (i.e. slope and toe) of the critical slip surface. When subsequently comparing the critical slip surfaces, it can be seen from figure 3.21 that at the slope increases in the unit weight of the stabilisation hardly cause a change in shape or position of the critical slip surface. At the crest on the other hand, the critical slip surface became bigger whereas at the toe the critical slip surface changed shape and retreated towards the slope of the levee. Clearly, at this levee increases in the unit weight of the stabilised soils do result in new paths of least resistance.

Lastly, the maximum Factor of Safety against inward macro-instability seem to have been reached for the stabilisation at the slope, almost for the stabilisation at the toe and not for the stabilisation at the crest. At the slope, the critical slip surface has completely avoided the block of stabilised soil. As a result, further strengthening of the soil will not yield additional increases in the Factor of Safety. On the other hand, at the toe the maximum Factor of Safety seems to have almost been reached. The critical slip surface as determined with the Bishop model completely avoids the block of stabilised soil, whereas the critical slip surface as determined with the Uplift Van model passes through the block. Since the difference in the Factor of Safety between both models was low, it is expected that after an additional increase in strength the critical slip surface as determined with the Uplift Van model will move to completely avoid the block of stabilised soil. Lastly, at the crest the critical slip surface still passed through the stabilised soil. Here, additional strengthening will yield additional increases in the Factor of Safety. Even though this is possible, it needs to be determined whether this would be desirable. After all, increasing the design value of the effective cohesion any further may result in a too large mean value of the effective cohesion. In the field, this would mean that a material with properties similar to a 'block of concrete' would be created, which is not desirable at levees.

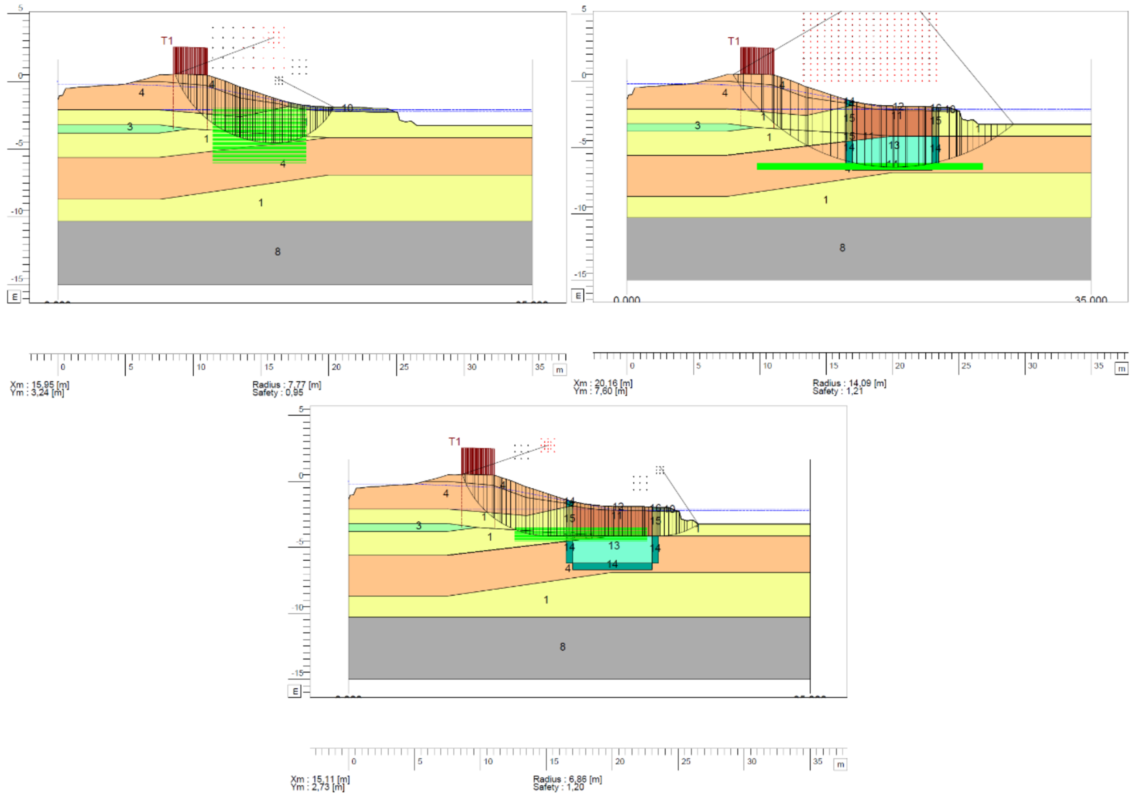

Figure 3.17; The critical slip surface of the initial situation (upper left image: Uplift Van) and of the final situation with a block of stabilised soil at the toe without a unit weight increase (upper right image: Bishop; lower image: Uplift Van).

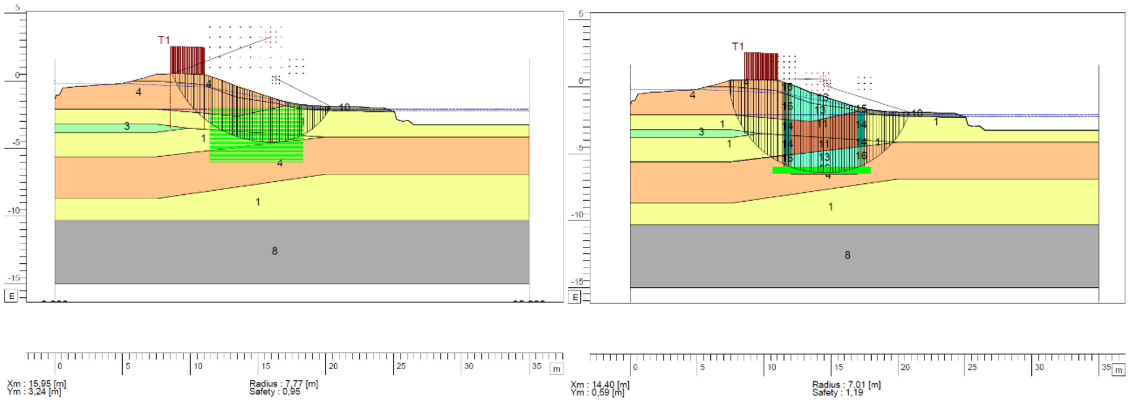

Figure 3.18; The critical slip surface of the initial situation (left image: Uplift Van) and of the final situation with a block of stabilised soil at the slope without a unit weight increase (right image: Uplift Van).

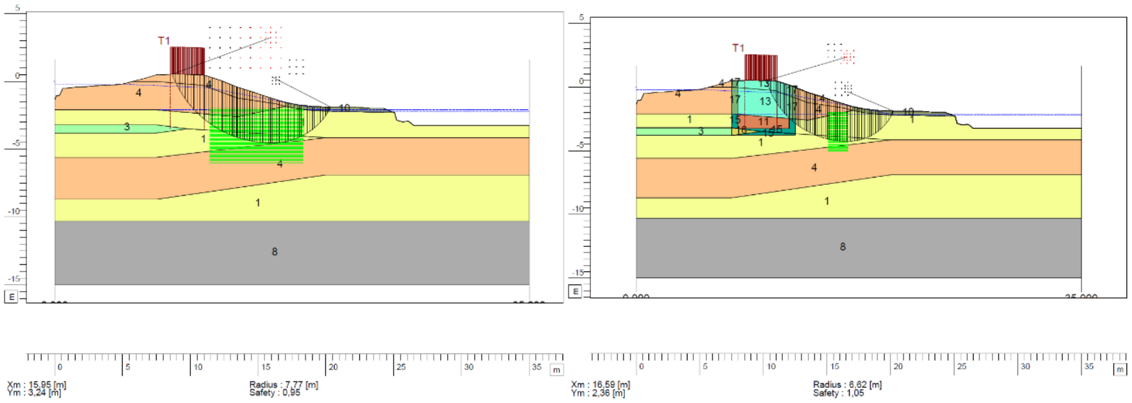

Figure 3.19; The critical slip surface of the initial situation (left image: Uplift Van) and of the final situation with a block of stabilised soil at the crest without a unit weight increase (right image: Uplift Van).

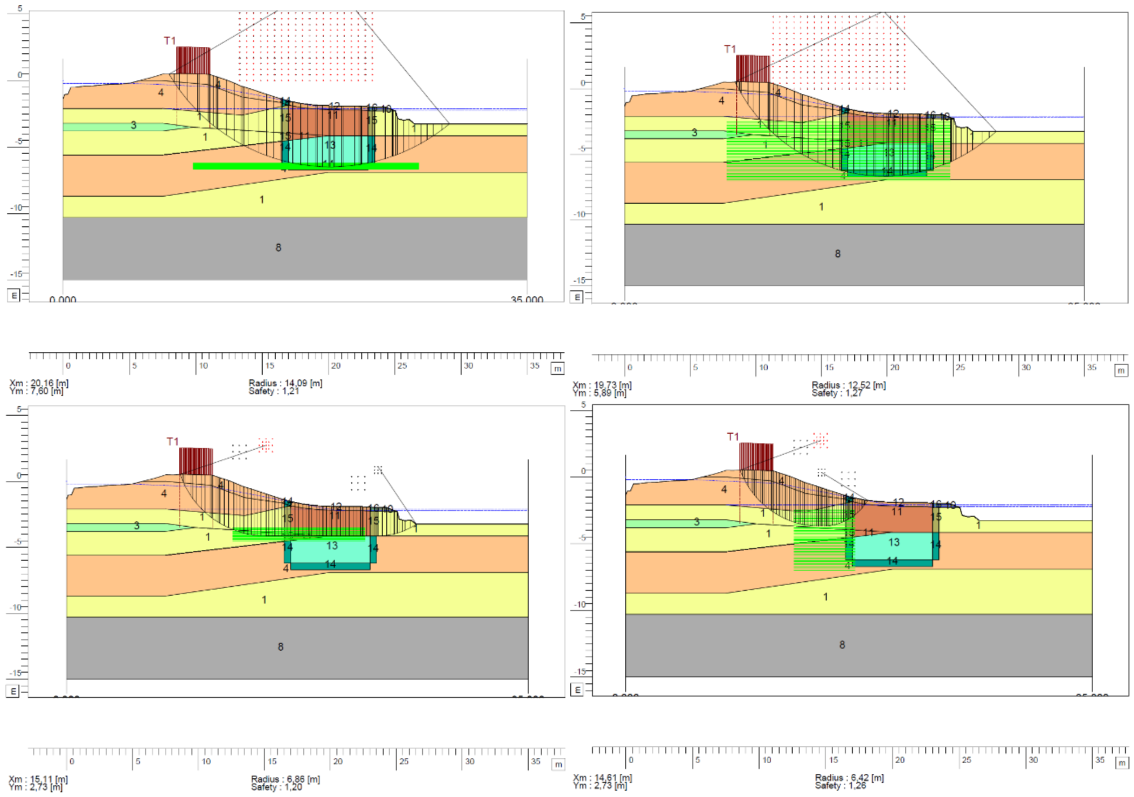

Figure 3.20; The critical slip surface of the final situation with a block of stabilised soil at the toe without a unit weight increase (upper left image: Bishop; lower left image: Uplift Van) and with a unit weight increase (upper right image: Bishop; lower right image: Uplift Van).

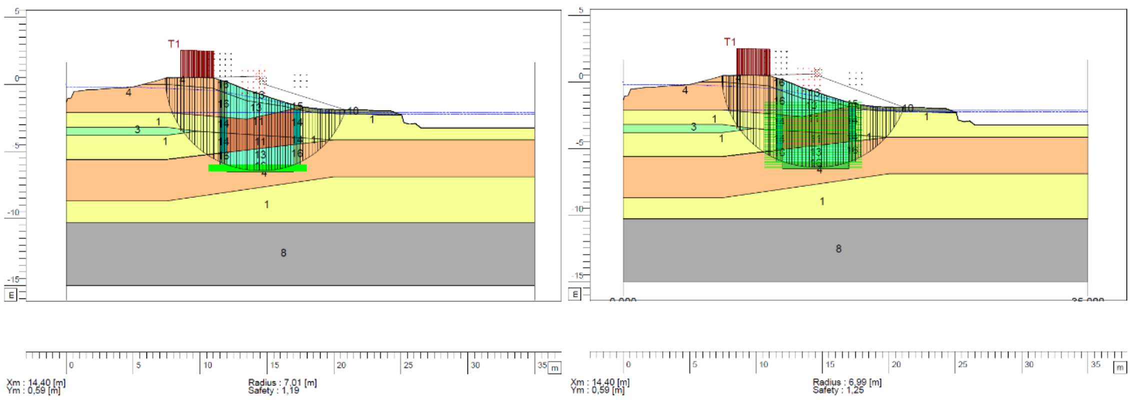

Figure 3.21; The critical slip surface of the initial situation (left image: Uplift Van) and of the final situation with a block of stabilised soil at the slope without a unit weight increase (right image: Uplift Van).

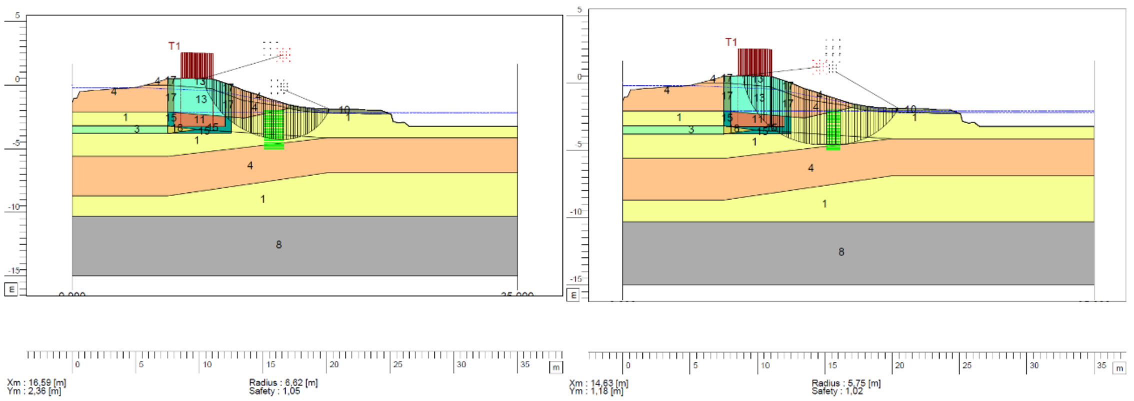

Figure 3.22; The critical slip surface of the initial situation (left image: Uplift Van) and of the final situation with a block of stabilised soil at the crest without a unit weight increase (right image: Uplift Van).

3.4 Preferred stabilisation position

When examining the Factors of Safety obtained for the soil stabilisations at the levee at the Monfoortse Vaart as presented in table 3.5, it can be seen that with the modelled strength the required Factor of Safety was reached for all examined blocks of stabilised soil. As a result, it was possible to reinforce the levee with all three examined blocks of stabilised soil. However, this does not mean that all examined stabilisations are equally preferred. Upon further inspection of table 3.5, the following was seen:

- The smallest increases in the Factor of Safety were obtained by stabilising the examined block of soil at the toe of the levee, regardless of the assumption on the unit weight;
- Under the assumption that the unit weight does not change, the largest increases in the Factor of Safety were obtained by stabilising the examined block of soil at the crest of the levee;
- Under the assumption of a 10% increase in the unit weight, the largest increases in the Factor of Safety were obtained by stabilising the examined blocks of soil at the slope and the crest of the levee.

When looking at these results, it can be said that for the levee at the Montfoortse Vaart the preferred spot for stabilisation of the soil is at the crest of the levee. At this spot, the largest increases in the Factor of Safety were obtained, with possibilities to increase the Factor of Safety even further upon additional strengthening (see section 3.3.1). Even though the increase in the unit weight did lead to a reduction in the Factor of Safety, this reduction only evaluated to a mere 2%. Aside from this, it is not guaranteed that the unit weight will increase much at all (see section 2.4.2), making the stabilisation at the crest of the levee the most preferred.

When subsequently examining the Factors of Safety obtained for the soil stabilisations at the levee at the Enkele Wiericke as presented in table 3.6, it can be seen that with the modelled strength the required Factor of Safety was reached for the examined block of stabilised soil at the toe and the slope of the levee. However, the required Factor of Safety was not reached with the modelled strength for the examined block of stabilised soil at the crest of the levee. As a result, the examined block of stabilised soil at the crest seemed unfavourable for the reinforcement of the levee. Apart from this, the following was noticed upon further inspection of table 3.6:

- The smallest increases in the Factor of Safety were obtained by stabilising the examined block of soil at the crest of the levee, regardless of the assumption on the unit weight;
- The largest increases in the Factor of Safety were obtained by stabilising the examined blocks of stabilised soil at the toe and slope of the levee, regardless of the assumption on the unit weight.

When looking at these results, it was concluded that the preferred spot for stabilisation at the levee at the Enkele Wiericke was at both the toe and the slope of the levee. Both examined stabilisations yielded similar increases in the Factor of Safety, regardless of the assumption on the unit weight of the stabilised soils. As explained in section 3.3.2, this may be the result of the stabilisation of the weak peat layer with both blocks. Because of the similar increases in the Factor of Safety, it seems that stabilising soil at the slope and at the toe are equally preferred. In this situation, a choice for the spot of stabilisation could be made based on for example the ease of implementation.

3.5 Conclusion

Two-dimensional stability analyses were carried out in which two real Dutch levees were theoretically reinforced with mass stabilisation. The purpose of these stability analyses was to determine whether it is possible to solve a stability deficit at an existing levee. To examine this, two sub-questions were formulated:

‘Which increases in the Factor of Safety can be realised by stabilising strips of soil at the levee?’

‘Where should the stabilisation of the soil at the levee preferably be carried out from an empirical point of view?’

The results from the two-dimensional stability analyses have shown that it was possible to reinforce both levees with mass stabilisation, with which increases in the Factor of Safety between 7% and 47% have been achieved. These numbers have been obtained using a synthetically chosen fixed strength, the analyses showing that this strength was sufficient to find the maximum Factor of Safety in most cases. However, the results of the analyses also show that further increases in the Factor of Safety are possible by further strengthening of the soil at the crest of both levees. This may not be desirable though as this may result in such high strengths that material properties similar to a 'block of concrete' will be obtained.

Furthermore, the results of the two-dimensional stability analyses have shown that the preferred spot for stabilisation is different in the two cases studied. For the levee at the Montfoortse Vaart stabilisation at the crest of the levee is preferred, whereas for the levee at the Enkele Wiericke stabilisation at either the slope or the toe is preferred. These different preferences are the result of the different soil profiles at both levees. Because of this, the preferred spot for stabilisation is case-dependent.

In conclusion, although the preferred spot for stabilisation enabling effective reinforcement is case-dependent, it is nonetheless possible to solve a stability deficit of a levee by applying mass stabilisation given the large increases in the Factor of Safety that were achieved.

3.6 Case selection

Since there was insufficient time available for this research, the achievability of the required strength of the stabilised soil and the practicability of mass stabilisation at levees was only examined for one stabilisation of a single case. For the following reasons, it was decided to examine this for the stabilisation of the peat and organic clay at the toe of the levee at the Montfoortse Vaart:

- The levee at the Monfoortse Vaart was selected instead of the levee at the Enkele Wiericke since more information was available to the author on the levee at the Montfoortse Vaart;
- The stabilisation of soil at the toe of the levee was considered more promising in terms of practicability than the stabilisation of soil at the slope or the crest (see chapter 7, section 7.4);
- For the laboratory research, a large quantity of soil had to be excavated using a mini-digger (see section 4.2.1). It was considered more feasible to do so at a small distance from the levee than by excavating soil from the levee itself;
- The excavated soil was more representative of the soil at the toe of the levee than of the soil at the slope or crest of the levee, since the soil at the slope and crest of the levee was different from the soil at the toe (see figure 3.5).

To subsequently determine the required strength of the stabilised peat and the stabilised organic clay, a number of two-dimensional stability calculations were made. In these calculations, the Factor of Safety against inward macro-instability was calculated for the levee for various combinations of the effective strength parameters of the stabilised soils at the toe. The results showed that there are four combinations of the design values of the effective strength parameters that the stabilised soils should at least have in order to achieve the desired increase in the Factor of Safety. Here the effective strength parameters were defined at 2% axial strain (stabilised organic clay) and 5% shear strain (stabilised peat), equal to the strains at which the effective strength parameters of the undisturbed organic clay and peat were determined (see appendix B). The obtained four combinations were subsequently converted to mean values by using the coefficients of variation and the partial material factors from table 3.7.

Table 3.7; The applied coefficients of variation and partial material factors for the cohesion and the tangent of the internal friction for the stabilised soil layers at the toe of the levee at the Montfoortse Vaart.

Soil type	$COV_{c'}$	$COV_{tan(\phi')}$	$\gamma_{mat;c'}$	$\gamma_{mat;tan(\phi')}$
	[-]	[-]	[-]	[-]
Peat	0,4	0,03	1,5	1,2
Organic clay	0,4	0,09	1,5	1,2

The coefficient of variation of the effective cohesion was set equal to 0,4 for both stabilised soils. This value was selected as twice the coefficient of variation of the effective cohesion from table 2.b of Dutch standard NEN 9997-1 (i.e. Eurocode 7 + Dutch national appendix) (Normcommissie 351 006 "Geotechniek", 2017). This value was chosen as no indications on the coefficient of variation of the effective cohesion were found in literature for stabilised soils. Since it was found that the variation in strength of the stabilised soil could be two times larger than that of the undisturbed soil (see section 2.4.1.4), the aforementioned doubling of the coefficient of variation of the effective cohesion was applied. For this doubling the coefficient of variation of the effective cohesion of the undisturbed soils could not be used, since these coefficients of variation were either negative or exceeding 3,0, making them unusable.

The coefficient of variation of the tangent of the effective angle of internal friction ($\tan(\phi')$) of both stabilised soils was set equal to those of the undisturbed soils. These coefficients of variation were selected as most variation in the shear strength (τ) of the stabilised soil is expected to be caused by variations in the effective cohesion and to a lesser extent by variations in the effective angle of internal friction. After all, a large variation in $\tan(\phi')$ would imply that the stabilised soil would be able to have unrealistically large friction angles.

The partial material factors for the effective strength parameters were chosen from Module C of the Dutch STOWA guideline for the assessment of the safety of regional flood defences (Stichting Toegepast Onderzoek Waterbeheer, 2015c). These factors were selected rather high to include additional safety, since suitable values of the partial material factors were not known for mass stabilised soils.

After conversion from design values to mean values, the unconfined compressive strength was determined for each of the four combinations using a correlation. The obtained four combinations of the effective strength parameters and the corresponding unconfined compressive strength are presented in table 3.8. Since the unconfined compressive strength will mostly be used as a strength criterion in the laboratory (see chapter 4), the required unconfined compressive strength was set equal to 50 kPa to ensure that the unconfined compressive strength always meets all four options.

The complete derivation of the required design values of the effective strength parameters, the conversion of these design values to mean values and the subsequent derivation of the unconfined compressive strength is presented in appendix C.

Table 3.8; The required mean values for the cohesion, the angle of internal friction and the unconfined compressive strength (UCS) of the stabilised soil layers at the toe of the levee at the Montfoortse Vaart for the different options.

Soil parameters	**Option 1**		**Option 2**		**Option 3**		**Option 4**		**Unit**
	Peat	**Organic clay**	**Peat**	**Organic clay**	**Peat**	**Organic clay**	**Peat**	**Organic clay**	
$\boldsymbol{c'}$	≥ 4,4	≥ 4,4	≥ 8,7	≥ 8,7	≥ 13,1	≥ 13,1	≥ 17,4	≥ 17,4	[kPa]
$\boldsymbol{\phi'}$	≥ 36,1	≥ 39,1	≥ 30,5	≥ 33,3	≥ 24,7	≥ 27,1	≥ 18,7	≥ 20,7	[°]
$\boldsymbol{UCS}$	≥ 17	≥ 19	≥ 30	≥ 32	≥ 41	≥ 43	≥ 49	≥ 50	[kPa]

4 Laboratory research

4.1 Introduction

The technical feasibility of applying mass stabilisation for reinforcing levees is dependent on among others the achievability of the required strength of the stabilised soils as determined for the selected design. Additionally, it is also important to know how the strength of the stabilised soil changes in time. To examine this, the following sub-question was formulated:

'How do the strength properties of the soil(s) to be stabilised from the selected case change as a result of stabilisation with a preselected binder and dosage?'

In order to examine the strength parameters, laboratory research was carried out according to the laboratory research plan presented in section 4.2. From the results of the laboratory research a suitable binder recipe for stabilisation of the examined soils is selected in section 4.3. Subsequently, the development of the strength for the examined soils due to stabilisation with the selected binder is presented in section 4.4. Next, the changes in strength for the examined soils due to stabilisation with the selected binder are presented in section 4.5. From the results conclusions were drawn in section 4.6 with which the above sub-question was answered.

4.2 Setup laboratory research and field work

According to the selected design from section 3.6, a peat and an organic clay are to be stabilised in order to reinforce the levee (see figure 3.5). In order to examine the changes in the properties of this peat and organic clay due to stabilisation, field work and a five-part laboratory research at the geotechnical laboratories of Fugro NL Land B.V. and Delft University of Technology was carried out. An overview of the steps taken during this research to examine the properties of the stabilised soils is shown in figure 4.1.

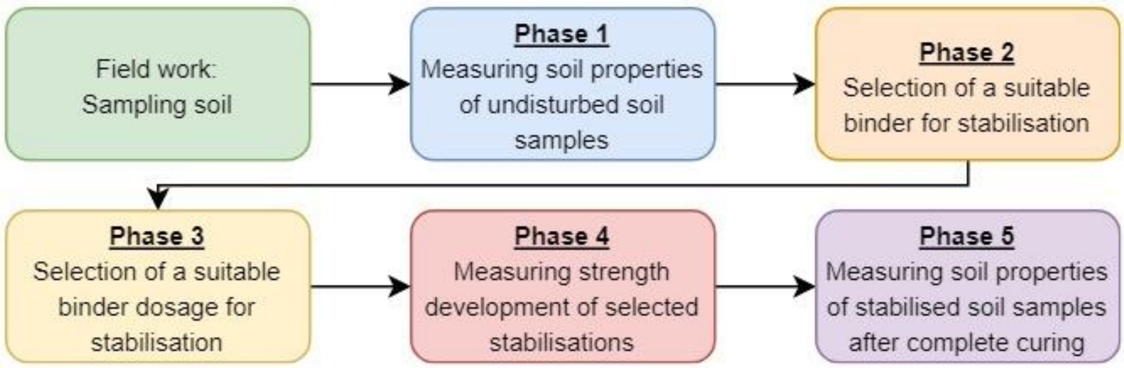

Figure 4.1; Flow chart showing the general steps taken during the laboratory research.

To examine the properties of the stabilised soils, field work was carried out first to obtain samples which were subsequently examined in the laboratory. After this, suitable binder recipes for the stabilisation of both soils meeting the strength requirements was searched for. As a result of limitations in this research in terms of time, budget and equipment, it was only possible to examine the properties of the stabilised soils in detail for a stabilisation with a single binder and dosage. For this reason, first a suitable binder was determined, after which an optimisation in the applied dosage was made separately for both soils. With the selection of the binder and dosage, the strength development of the soils stabilised with these binder dosages was examined and the time to complete curing was determined. After this, the properties of the stabilised soils were determined after complete curing and subsequently compared to the same properties of the undisturbed soils.

The detailed steps taken in each of the aforementioned parts is presented in the next subsections.

4.2.1 Field work

Before laboratory research into the strength properties of the peat and organic clay due to stabilisation could begin, samples of both soil types were required. From early estimations it was determined that about 1,0 m^3 of peat and 1,0 m^3 of organic clay were required. Since these quantities were too large to sample using thin-walled open-ended Ackermann tubes, a mini-digger was required to excavate the 2,0 m^3 of soil. Although the mini-digger remoulded the soil, it was not a problem since this soil would be mixed in the laboratory with a binder to make stabilised soil samples. However, since the excavated soil is remoulded, also a number of undisturbed soil samples were required. It was estimated that 5 fully filled thin-walled open-ended Ackermann tubes for both the peat and the organic clay would be sufficient.

On 9 October 2018 field work was carried in the Ecopark in Linschoten (Netherlands) to obtain the aforementioned quantities of both excavated (and thus disturbed) and undisturbed peat and organic clay. In the Ecopark an excavation of about 5,0 by 5,0 metres was made up to about 1,8 metres deep using a mini digger. From the excavation about 1,0 m^3 of organic clay and 1,0 m^3 of peat was taken. These soils were stored separately in two water- and airtight palletboxes.

Additionally, five 67 mm thin-walled open-ended Ackermann tubes were used per soil type to sample organic clay and peat in the excavated trench. The organic clay was sampled horizontally in the wall of the trench using five 67 mm thin-walled open-ended Ackermann tubes, whereas the peat was sampled vertically in the floor of the trench using five 67 mm thin-walled open-ended Ackermann tubes. The organic clay samples were taken at about 1,4 metres below ground level, whereas the peat samples were taken at about 1,8 metres below ground level. Although the groundwater level was not measured during the field work, manual drillings carried out by the author two months prior showed that the groundwater level at the site of the excavation was about 1,1 metres below ground level.

After the soil sampling was completed, the 10 Ackermann tubes and the four pallet boxes were transported towards the geotechnical laboratory of Fugro NL Land B.V. in Arnhem (Netherlands). More details on the executed field work and the manual drillings are presented in appendix E.

4.2.2 Phase 1 – Soil parameters of undisturbed soil samples

At the geotechnical laboratory of Fugro NL Land B.V., the undisturbed soil samples were extruded from the Ackermann tubes. After extrusion, the samples were visually inspected on irregularities and disturbance. A classification was also carried for one sample of both soil types (see appendix F). Although no signs of disturbance were observed, it was noticed that some of the tubes containing peat were not completely filled. Additionally, it was observed that the undisturbed peat samples were rich in wood.

After the visual inspection, the undisturbed soil samples were tested for strength, stiffness and index properties. In order to test for these properties, the geotechnical laboratory tests as presented in table 4.1 were carried out. These laboratory tests are necessary as information on the aforementioned properties is required for reference purposes (in particular for the strength properties) and for the production of stabilised soil samples during the laboratory research (see section 4.2.3). The results of the index tests, as required for the production of stabilised soils, are presented in appendix F and are not discussed in this report.

The drained shear strength properties of both undisturbed soil samples were determined using different geotechnical laboratory tests as shown in table 4.1. These laboratory tests were selected differently based on the requirements of the applied Dutch STOWA guideline as well as on recommendations from the Deltares laboratory protocol for soil investigation near flood defences. In accordance with the Dutch STOWA guideline, the organic clay samples were subjected to istropically consolidated undrained triaxial compression tests (CIUc triaxial tests) (Stichting Toegepast Onderzoek Waterbeheer, 2015c) (van Duinen, 2012). The peat samples on the other hand are recommended to be subjected to direct simple shear tests (DSS tests) according to Deltares laboratory protocol (Greeuw, van Essen, & van Duinen, 2016). However, during the research there were no DSS test setups available to the author in either geotechnical laboratory. As a result, it was decided to subject the peat samples to another shear test instead: the shearbox test.

Table 4.1; The laboratory tests that were carried out on the undisturbed soil samples during phase 1 of the laboratory research.

Laboratory test	Number of peat samples tested	Number of organic clay samples tested	Applied standard or guideline	Laboratory
Water content measurement	3	3	NEN-EN-ISO 17892-1	Fugro
Bulk density measurement (linear method)	3	3	NEN-EN-ISO 17892-2	Fugro
Fluid pycnometer method (particle density)	3	3	NEN-EN-ISO 17892-3	Fugro
Oedometer test	3	3	NEN-EN-ISO 17892-5	Fugro
Unconfined compression test	1	1	NEN-EN-ISO 17892-7	Fugro
CIUc triaxial test (single stage)	-	3	NEN-EN-ISO 17892-9	Fugro
Shear box test (single stage)	4	-	NEN-EN-ISO 17892-10	Delft

Furthermore, oedometer tests were carried out on both undisturbed soil samples to determine the stiffness properties of these soils, as well as to obtain an indication on the hydraulic conductivity of both soils. However, these tests were not required for this research as a focus was applied on the strength properties of the stabilised soils. Therefore the results of the oedometer tests will not be discussed in this report and are thus presented in appendix F instead.

Lastly, literature also recommends making measurements of the chemical and environmental properties of the undisturbed soils prior to stabilisation (CUR onderzoekscommissie D34 "Kalk-cementkolommen", 2001). These measurements were not carried out due to either time, budget or equipment restrictions.

4.2.3 Phase 2 – Binder selection

Next, a suitable binder for the stabilisation of the peat and the organic clay had to be found.

4.2.3.1 Applied binders

In this phase of the laboratory research, the following four different binder materials were applied:

- A rapid-curing Portland cement (CEM I 52,5 R);
- A blast-furnace slag cement (CEM III/B 42,5 N-LH/HS);
- Blast-furnace slag cement with gypsum (CEM III/B 42,5 N-LH/HS + FGD-gypsum);
- A supersulphated cement (CEM I 52,5 R + GGBS + FGD-gypsum).

These binder materials were chosen based on experiences of the functioning of various binders in Dutch soils from literature (see table 2.3 in section 2.2.3.2) and experience and suggestions from engineers from companies such as Fugro NL Land B.V., KWS Infra and Vliegasunie. The specifications of these binders, such as composition and physical and binding characteristics, are presented in appendix G.

The binder dosages that were applied during this phase of the laboratory research are presented in table 4.2. For all applied binders the total applied dosage was 150 kilogram binder per cubic metre of undisturbed soil. In terms of mass this equals 150 kg binder per 1300 kg undisturbed organic clay (12% m/m) and 150 kg binder per 990 kg undisturbed peat (15% m/m) based on the bulk densities of the undisturbed soil samples as measured during phase 1 (see appendix F). This binder dosage was chosen based on table 2.5 (see section 2.2.3.3), which showed that 150 kg binder/m^3 soil is a typical binder dosage for the stabilisation of peat and (strongly) organic clays.

Table 4.2; The binders and dosages that were applied in trial stabilisations during phase 2 of the laboratory research.

Soil type	Applied binder	Applied dosage [kg binder/m³ undisturbed soil]
Peat	CEM I 52,5 R	150
Peat	CEM III/B 42,5 N-LH/HS	150
Peat	CEM III/B 42,5 N-LH/HS + FGD-gypsum	120 CEM III/B + 30 FGD-gypsum (mixing ratio: 80/20 % m/m)
Peat	Super sulphated cement	127,5 GGBS + 15 FGD-gypsum + 7,5 CEM I (mixing ratio: 85/10/5 % m/m)
Organic clay	CEM I 52,5 R	150
Organic clay	CEM III/B 42,5 N-LH/HS	150
Organic clay	CEM III/B 42,5 N-LH/HS + FGD-gypsum	120 CEM III/B + 30 FGD-gypsum (mixing ratio: 80/20 % m/m)
Organic clay	Super sulphated cement	127,5 GGBS + 15 FGD-gypsum + 7,5 CEM I (85/10/5 % m/m)

4.2.3.2 Sample preparation

In the laboratory, the excavated peat and organic clay samples were subsequently stabilised in the geotechnical laboratory of Fugro NL Land B.V. with the aforementioned binders according to the laboratory soil stabilisation procedure outlined in appendix D. The mixtures were produced in a such a manner that a mixture representative of the field would be obtained if the same binder dosage would be applied in the stabilisation of the soil in the field. This was achieved in the laboratory by mixing the same relative amounts of soil and binder as would be mixed in the field. So in order to mimic the field stabilisation of 1,0 m^3 of undisturbed organic clay weighing 1300 kg with 150 kg binder, a mass of organic clay was mixed in the laboratory with a mass of binder equal to 12% of the mass of the organic clay used in this mixture. Here the 12% is the ratio between added binder and soil (150/1300) expressed as a percentage. Similarly, the field stabilisation of 1,0 m^3 of peat weighing 990 kg with 150 kg binder was mimicked by mixing a mass of peat in the laboratory with a mass of binder equal to 15% (150/990) of the mass of the peat used in this mixture. By mixing the soil and binder in this manner, mixtures with the compositions from table 4.3 were produced.

Table 4.3; The compositions of the produced mixtures during phase 2 of the laboratory research.

Mixture	Soil	Cement	Gypsum	GGBS	Added water
	[% m/m]	[% m/m]	[% m/m]	[% m/m]	[% m/m]
Peat + 150 kg/m³ CEM I	86,8	13,2	-	-	-
Peat + 150 kg/m³ CEM III	86,8	10,5	2,6	-	-
Peat + 120 kg/m³ CEM III + 30 kg/m³ FGD-gypsum	86,8	13,2	-	-	-
Peat + 150 kg/m³ super-sulphated cement	86,8	0,7	1,3	11,2	-
Organic clay + 150 kg/m³ CEM I	72,0	10,3	-	-	17,6
Organic clay + 150 kg/m³ CEM III	72,0	8,3	2,1	-	17,6
Organic clay + 120 kg/m³ CEM III + 30 kg/m³ FGD-gypsum	72,0	10,3	-	-	17,6
Organic clay + 150 kg/m³ super-sulphated cement	72,0	0,5	1,0	8,8	17,6

The mixtures themselves were produced by first premixing the soil to a homogeneous mass, after which the binder was gradually added and mixed with the premixed soil to a homogenous mass. The mixing times and mixing speeds that were maintained during the different stages of the mixing procedure for both soils are listed in table 4.4. These mixing times and mixing speeds were the empirically determined by means of trial stabilisations prior to phase 2.

Table 4.4; Applied mixing times and mixing speeds for the stabilisation of peat and the organic clay during phase 2.

Soil type	Premixing soil	Mixing soil and binder
Peat	5 min. at speed setting 1 (53 RPM)	5 min. at speed setting 1 (53 RPM)
Organic clay	5 min. at speed setting 1 (53 RPM), 1 minute at speed setting 2 (103 RPM) and 1 minute at speed setting 3 (157 RPM)	5 min. at speed setting 1 (53 RPM), 1 minute at speed setting 2 (103 RPM) and 1 minute at speed setting 3 (157 RPM)

The premixing of both soils was carried out for different reasons. The peat was premixed to a homogeneous mass in order to pulverise the many big chunks of wood that were present in the soil. The organic clay on the other hand was premixed with water to increase the water content of the excavated organic clay, explaining why additional water was part of the organic clay mixtures from table 4.3. It turned out that the water content of the excavated organic clay used in the mixtures was much lower than was measured for the undisturbed soil samples during phase 1, requiring the excavated organic clay to be premixed with water prior to stabilisation. This is why besides a measurement of the bulk density also a measurement of the water content of the undisturbed soil samples was required during phase 1.

After mixing is complete, four moulds were filled with each of the produced mixtures from table 4.3 within one to two hours after mixing. After the moulds were filled, a load of 8,0 kPa was mistakenly applied on top of the mixtures instead of the required 25,0 kPa. This load is required to mimic the in-situ effective stresses.

The required load was determined using an assumption on the execution of mass stabilisation at the levee at the Montfoortse Vaart. It was assumed that right after mixing a load of 8,0 kPa (about 0,5 m loosely deposited sand) was applied on top of the just stabilised soil. This preload of 8,0 kPa was also assumed in the implementation analyses (see section 5.2.1). The preload is applied in the field to enhance the strength development of the stabilised soil (see section 2.2.4). The effective in-situ field stress during curing was then calculated by adding:

- The effective field stress prior to stabilisation at the depths at which the undisturbed organic clay and peat samples were taken (about 15,0 kPa);
- The change in effective stress due to the assumed density increase of about 1,5 kN/m^3 caused by the mixing with 150 kg binder/m^3 soil at the depths at which the undisturbed organic clay and peat samples were taken (about 2,0 kPa);
- The assumed preload of 8,0 kPa.

Since the difference between the obtained load for the stabilised peat and stabilised organic clay was less than 1,0 kPa, it was decided to apply the same load of 25,0 kPa on both mixtures.

After the application of the load, the mixtures started to compress. This compression of the mixtures due to the applied 8,0 kPa load was recorded (see appendix F) in accordance with the laboratory soil stabilisation procedure from appendix D. However, sometimes also an expansion of the mixtures was measured after the initial compression. These expansions are explained by either of the following reasons:

- Expansions of 1,0 mm were likely to have been recorded as a result of measuring inaccuracy when manually reading off the compression (and expansion) using a measuring tape (see appendix D);
- Expansions larger than 1,0 mm were likely recorded as a result of hydration heat, resulting from the reactions between the cement particles and the pore water. Such large expansions were rarely recorded during this research.

4.2.3.3 Determination of the most suitable binder

In order to determine the most suitable binder, the mixtures were left to cure under the 8,0 kPa load for 7 days, after which the stabilised soil samples were extruded from the moulds and subjected to unconfined compression tests. The selection for 7 days of allowed curing was made, as after 7 days a lot of the strength will have developed already as shown by the curing curves obtained from literature (see section 2.4.1.3). Since the purpose is to determine which binder works best, it is not required to wait for curing to have finished.

The results of the unconfined compression tests were subsequently used to select a binder that will be applied during the remainder of the laboratory research. Criteria for the selection of a binder included:

- The 7-day unconfined compressive strength of the soil sample(s) stabilised with this binder must have at least exceeded the required unconfined compressive strength of 50 kPa (see section 3.6).
- The applied binder must meet the first criterion for both the peat and the organic clay, as this is easier for a contractor to apply in the field when dealing with multiple soil layers.

In the event multiple binder materials were found to be suitable for the stabilisation of both the peat and organic clay, other criteria could be applied to select the most suitable binder. Other criteria included:

- The binder should preferably have a rapid strength development based on information sheets;
- The selected binder should emit as little CO_2 as possible upon curing (meaning the dosage of Portland clinker in the binder should be as small as possible).

It should be noted that for the determination of the most suitable binder the unconfined compressive strength was used a selection criterion instead of the effective strength parameters with which the stability calculations from chapter 3 were carried out. The reason for this choice was that there were insufficient triaxial test and/or shearbox test setups available to the author during this phase of the research. Additionally, the couple of weeks it would have taken to carry out these tests on the stabilised soil samples with the limited equipment available was also considered too long before the laboratory research could be continued. After all, the research had to be carried out within a given timeframe.

4.2.4 Phase 3 – Binder dosage selection

After a suitable binder was found for the stabilisation of both the peat and the organic clay, a suitable binder dosage had to be found.

4.2.4.1 Applied binder dosages

From the results of phase 2 it was shown that the rapid-curing Portland cement (CEM I 52,5 R) was the most suitable binder, but the applied dosage of 150 kg binder/m^3 undisturbed soil was too large as strengths multiple times the required strength after complete curing were achieved (see section 4.3.1). This is undesirable for the following reasons:

- Adding more binder than necessary causes large increases of the project costs, which is undesirable for all parties involved;
- Adding more binder than necessary will result in a stabilised soil more reminiscent of a 'block of concrete' rather than a soil with an improved strength. This not desirable at levees (see section 7.1.2 for explanation).

As a result, an optimisation of the binder dosage was required. The optimisation of the binder dosage was examined in this phase by stabilising both the excavated peat and the organic clay with the binder dosages from table 4.5. Since the obtained unconfined compressive strengths were too large, the binder dosages applied in this phase were smaller than the in phase 1 applied dosage of 150 kg binder/m^3 undisturbed soil. Like in phase 2, the binder dosages are also presented in table 4.5 as the amount of binder added to the mass of 1,0 m^3 of undisturbed soil, expressed as a mass percentage.

Table 4.5; The applied binder dosages during phase 3 of the laboratory research.

Soil type	Applied binder	Applied dosage [kg binder/m^3 undisturbed soil]	Applied dosage [% m/m]
Peat	CEM I 52,5 R	50, 75, 100 and 125	5, 8, 10 and 13
Organic clay	CEM I 52,5 R	50, 75, 100 and 125	4, 6, 8 and 10

4.2.4.2 Sample preparation

Following the same approach described in phase 2 (section 4.2.3.2), the mixtures from table 4.6 were produced according to the laboratory soil stabilisation procedure outlined in appendix D using the same mixing times and mixing speeds as applied in phase 2 (see table 4.4).

Table 4.6; The compositions of the produced mixtures during phase 3 of the laboratory research.

Mixture	Soil	Cement	Added water
	[% m/m]	[% m/m]	[% m/m]
Peat + 50 kg/m³ CEM I	95,2	4,8	-
Peat + 75 kg/m³ CEM I	93,0	7,0	-
Peat + 100 kg/m³ CEM I	90,8	9,2	-
Peat + 125 kg/m³ CEM I	88,8	11,2	-
Organic clay + 50 kg/m³ CEM I	79,5	3,7	16,8
Organic clay + 75 kg/m³ CEM I	78,1	5,5	16,5
Organic clay + 100 kg/m³ CEM I	76,7	7,1	16,2
Organic clay + 125 kg/m³ CEM I	75,3	8,8	15,9

After mixing was complete, three moulds were filled with each of the produced mixtures from table 4.6 within one to two hours after mixing. After the moulds were filled, a load of 25,0 kPa was applied on top of the mixtures. Although different binder dosages were applied, resulting in different assumed density changes, this resulted in an approximate load between 24,0 and 25,0 kPa. These changes were considered negligibly small on the strength development and as a result a load of 25,0 kPa was applied on all samples.

4.2.4.3 Determination of the most suitable binder dosage

In order to determine the most suitable Portland cement dosage, the mixtures were left to cure under the 25,0 kPa load for 28 days, after which the stabilised soil samples were extruded from the moulds and subjected to unconfined compression tests. The selection for 28 days of allowed curing was made because after 28 days the curing reactions will likely have finished with Portland cement as a binder (see section 2.4.1.3). Since it is not desired that the stabilised soil continues to develop a significant amount of additional strength after the specified curing time has elapsed due to the issue of obtaining a 'block of concrete', 28 days of curing were allowed as it should be enough to reach the fully cured strength.

The results of the unconfined compression tests were subsequently used to select a binder dosage that will be applied during the remainder of the laboratory research. The sole criterion for the Portland cement dosage selection is that after 28 days of curing the required unconfined compressive strength of 50 kPa set in section 3.6 is reached or exceeded. The lowest Portland cement dosage at which this criterion was met was subsequently selected for the reasons listed in section 4.2.4.1.

Like in phase 2, the unconfined compressive strength was used as a selection criterion instead of the effective strength parameters. This was done for the same reasons as mentioned in section 4.2.3.3.

4.2.5 Phase 4 – UCS curing curve determination

With a suitable binder type and dosage found for the stabilisation of the peat and the organic clay, the strength development of these two stabilisations was determined.

4.2.5.1 Applied methods

In this phase, the strength development of the chosen mixtures was determined in terms of the unconfined compressive strength. The strength development in terms of the unconfined compressive strength over time is also known as a curing curve. In order to determine the curing curves for the stabilisation of the peat and organic clay with the Portland cement dosages from table 4.7, two different methods were applied:

1. <u>Preparation of multiple batches of stabilised soil</u>
   Multiple batches of the same mixtures were produced, each of which was divided over three moulds. Subsequently, all three samples from the same batch were left to cure for the same predetermined amount of time. After the curing time had elapsed, the stabilised soil samples were extruded from the three moulds and subsequently subjected to an unconfined compression test. By combining results from multiple batches left to cure for different amounts of time, a curing curve was constructed.

2. <u>Preparation of one single batch of stabilised soil</u>
   A single batch of both mixtures was produced, after which each mixture was divided over five moulds. Every sample from each mixture was subsequently left to cure for a different predetermined amount of time. After a specified curing time had elapsed, one sample was extruded from the mould and subsequently subjected to an unconfined compression test. By combining the results from the five samples, a curing curve was constructed.

The difference in the applied methods are found in the assumption on the reproducibility of a mixture. In method 1 the assumption was made that mixtures are easily reproducible, leading to mixtures with similar (strength) properties when left to cure under similar conditions for the same amount of time. The advantage of this method is that multiple measurements of the unconfined compressive strength could be made per considered time step using the moulds and equipment shown in appendix D. This allowed for spotting outliers in the obtained results.

On the other hand, in method 2 it was assumed that mixtures are not easily reproducible, leading to mixtures with different (strength) properties when left to cure under similar conditions for the same amount of time. The disadvantage of this method is that only a single measurement of the unconfined compressive strength could be made per considered time step. This is because of the limited amount of samples that could be produced due to the mould size and the used equipment to produce the mixtures.

Table 4.7; The applied binder dosages during phase 4 of the laboratory research.

Soil type	Applied binder	Applied dosage [kg binder/m^3 undisturbed soil]	Applied dosage [% m/m]
Peat	CEM I 52,5 R	50	5
Organic clay	CEM I 52,5 R	75	4

It should be noted that the strength development was determined in terms of the unconfined compressive strength instead of the effective strength parameters. This was done as there were insufficient triaxial test and shearbox test setups available to the author to carry out these tests within an acceptable timeframe. Even if sufficient setups were available, it would not have been possible to determine the strength of the samples with these tests at early curing times where a lot of the strength is expected to develop (see e.g. figure 2.7 and figure 2.8 from section 2.4.1.3). The triaxial and shearbox tests usually require some days to carry out which, when combined with the fact that the mixtures are first required to compress and partially cure under a load, make early-age measurements impossible. Because of both reasons, unconfined compression tests, which require minutes to carry out, were used instead of triaxial and shearbox tests for the determination of the strength development.

4.2.5.2 Sample preparation

Following the same approach described in phase 2 (section 4.2.3.2), the mixtures from table 4.8 were produced according to the laboratory soil stabilisation procedure outlined in appendix D using the same mixing times and mixing speeds as applied in phase 2 (see table 4.4).

Table 4.8; The compositions of the produced mixtures during phase 4 of the laboratory research.

Mixture	Soil	Cement	Added water
	[% m/m]	[% m/m]	[% m/m]
Peat + 50 kg CEM I /m^3 soil	95,2	4,8	-
Organic clay + 75 kg CEM I /m^3 soil	78,1	5,5	16,5

After the moulds were filled with a mixture within one to two hours after mixing, a load of 25,0 kPa was applied. The load was subsequently left on each sample for a predetermined amount of time, allowing the samples to cure. The amount of time the samples were left to cure for both methods is presented in table 4.9. It should be noted that in method 1 no batches were left to cure for 28 days. As a result of the assumption in method 1 that mixtures are well reproducible, it was assumed that similar 28-day strengths would be obtained for the examined mixtures as measured during phase 3. As a result, it was not deemed useful to reproduce the same mixture and letting it cure for 28 days only to measure the same strengths.

Table 4.9; The applied curing times for the mixtures for both methods during phase 4 of the laboratory research.

Applied method for curing curve measurement	Curing time [days]
Method 1	1, 2, 7, 10, 14 and 21
Method 2	1, 2, 7, 14 and 28

4.2.6 Phase 5 – Soil parameters of stabilised soil samples

In the final phase of the laboratory research, the strength and stiffness properties of the selected peat and organic clay mixture from table 4.8 were examined after the mixtures had fully cured. In order to test for strength and stiffness properties, the geotechnical laboratory tests as presented in table 4.10 were carried out. These laboratory tests are necessary to determine whether the required strength is achieved or exceeded, as well as to allow for comparison with the properties of the undisturbed soils.

Table 4.10; The laboratory tests that were carried out on the stabilised soil samples during phase 5 of the laboratory research.

Laboratory test	Number of stabilised soil samples tested		Applied standard or guideline	Laboratory
	Stab. peat	Stab. org. clay		
Oedometer test	1	1	NEN-EN-ISO 17892-5	Fugro
Unconfined compression test	1	1	NEN-EN-ISO 17892-7	Fugro
CIUc triaxial test (single stage)	-	3	NEN-EN-ISO 17892-9	Fugro
Shear box test (single stage)	4	-	NEN-EN-ISO 17892-10	Delft

Following the same approach described in phase 2 (section 4.2.3.2), the mixtures from table 4.8 were produced according to the laboratory soil stabilisation procedure outlined in appendix D using the same mixing times and mixing speeds as applied in phase 2 (see table 4.4). After mixing was complete, two moulds were filled with each of the produced mixtures from table 4.8 within one to two hours after mixing. Subsequently, a load of 25,0 kPa was applied on top of the mixtures. The mixtures were then left to cure under the 25,0 kPa load for 28 days, after which the samples were extruded from the moulds and subjected to the geotechnical laboratory tests presented in table 4.10. Although according to the obtained curing curves it seems that the mixtures neared a fully cured state around 7 days of curing (see section 4.4.2), the samples were left to cure for 28 days due to logistics.

It can be seen from table 4.10 that CIUc triaxial tests were applied on the stabilised organic clay samples, whereas shearbox tests were applied on the stabilised peat samples. This choice was made to allow for direct comparison of the test results between the undisturbed soil samples and their stabilised counterparts. These tests were carried out in this phase as an indication of the drained shear strength parameters of both mixtures after complete curing was required in order to examine the practicability of mass stabilisation at levees. Besides this, since the organic clay mixture had to be subjected to triaxial tests once and the stabilised peat mixture had to be subjected to shearbox tests once, far less triaxial and shearbox test setups were required to carry out these tests. This made it possible to carry out these tests in this phase of the laboratory research within an acceptable timeframe with the limited available equipment.

Lastly, oedometer tests were carried out on both stabilised soil samples to determine the stiffness properties of these soils, as well as to obtain an indication on the hydraulic conductivity of both stabilised soils. However, these tests were not required for this research as a focus was applied on the strength properties of the stabilised soils. Therefore the results of the oedometer tests will not be discussed in this report and are thus presented in appendix F instead.

4.3 Determination of suitable binder recipe

Prior to the examination of the strength properties of the stabilised soils, a suitable binder recipe had to be determined with which the strength requirements set in section 4.2.3.3 and 4.2.4.3 could be met. A suitable binder recipe was determined by finding a suitable binder first, after which the binder dosage was optimised. The determination of the suitable binder and binder dosage is presented in the next subsections.

4.3.1 Binder selection

During phase 2 of the laboratory research, force-controlled unconfined compression tests were carried out to determine the 7-day unconfined compressive strength of peat and organic clay samples stabilised with a dosage of 150 kg of four different binders per cubic metre of undisturbed soil (i.e. 15% m/m peat and 12% m/m organic clay). The results of the unconfined compression tests carried out with the force rates listed in table 4.11 are presented in figure 4.2. The detailed test results are presented in appendix F.

Table 4.11; The applied force rates on the different stabilised soil samples during the unconfined compression tests.

Applied binder	Force rate (peat mixture) [N/s]	Force rate (organic clay mixture) [N/s]
CEM I 52,5 R	25	20
CEM III/B 42,5 N-LH/HS	20	10
CEM III/B 42,5 N-LH/HS + FGD-gypsum	20	5
Super sulphated cement	10	5

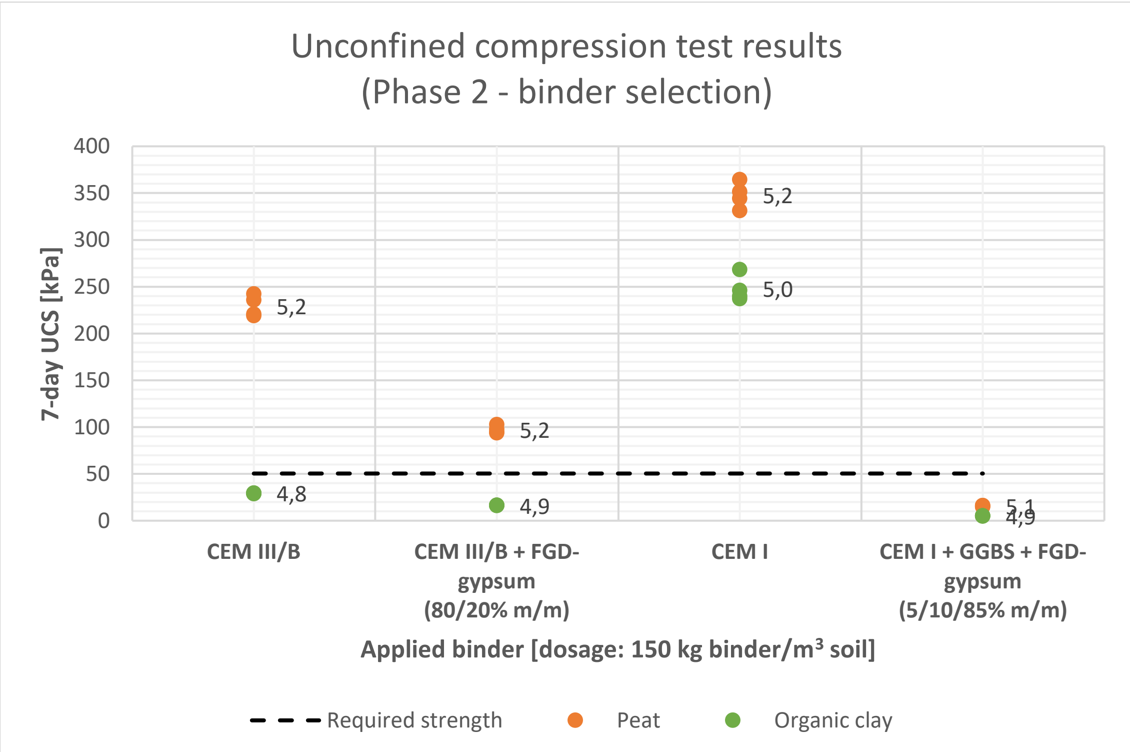

Figure 4.2; Unconfined compression test results from phase 2 of the laboratory research. The numbers in the graph represent the measured water-to-binder factor (w/b) of each tested mixture.

The unconfined compression tests were carried out with two major deviations from the procedure described in the applied Dutch standard NEN-EN-ISO 17892-7 (*'Unconfined compression test'*):

- Force-controlled unconfined compression tests were carried out instead of strain-controlled unconfined compression tests;
- Most stabilised soil samples did not meet the minimum required height-to-diameter ratio of 1,8.

Force-controlled unconfined compression tests were carried out instead of strain-controlled unconfined compression tests, because the compression machine in the geotechnical laboratory of Fugro NL Land B.V. was unable to accurately carry out a strain-controlled unconfined compression test. This problem could not be solved, so force-controlled tests had to be carried out. Another deviation from the procedure was the height-to-diameter ratio of most tested stabilised soil samples. The moulds used to make the stabilised soil samples were produced without keeping the compression of the mixtures due to loading into account. As a result, most samples had a recorded height-to-diameter between 1,6 and 1,8 after compression. This problem could not be solved due to budget and time restrictions, so it was decided to measure the unconfined compressive strengths in all phases of the laboratory research as is.

Upon inspection of figure 4.2, it can be seen that the measured unconfined compressive strength exceeded the required 50 kPa for a single organic clay mixture and for most peat mixtures. Clearly, not all binders were suitable to stabilise both soil samples with as the strength requirement was not met. This may be caused either due to any or a combination of the following reasons:

- There is a chemical mismatch between the soil and the applied binder, preventing a quick or proper hardening of the mixture;
- The applied binder dosage is too low;
- The clinker content, the component that causes cementation, may have been too low.

For the selection of the binder it was required that the measured strength exceeds 50 kPa and that the same binder were to be applied for the stabilisation of both the peat and the organic clay. Based on the results of the unconfined compression tests, the only binder suitable meeting both requirements was the rapid-curing Portland cement (CEM I). This binder was therefore selected to continue the laboratory research with. However, the applied binder dosage was clearly too large as strengths up to 7 times the required strength were measured. This is undesirable, as it is expensive to add more binder than needed to reach the required strength. Besides this, applying such large dosages results in materials with properties similar to a ‘block of concrete’, which is not desirable in levees. As a result, an optimisation of the binder dosage for both soil types is required.

4.3.2 Binder dosage selection

During phase 3 of the laboratory research, force-controlled unconfined compression tests were carried out to determine the 28-day unconfined compressive strength of peat and organic clay samples stabilised with four different dosages of Portland cement. The results of the unconfined compression tests carried out with the force rates listed in table 4.12 are presented in figure 4.3. The detailed test results are presented in appendix F. The unconfined compression tests were carried out with the same deviations from the procedure as listed in section 4.3.1.

Table 4.12; The applied force rates on the different stabilised soil samples during the unconfined compression tests.

Applied dosage	Force rate (peat mixture) [N/s]	Force rate (organic clay mixture) [N/s]
50 kg CEM I/m^3 undisturbed soil	10	10
75 kg CEM I/m^3 undisturbed soil	15	10
100 kg CEM I/m^3 undisturbed soil	20	10
125 kg CEM I/m^3 undisturbed soil	30	15

Upon inspection of figure 4.2, it can be seen that the measured unconfined compressive strength exceeded the required 50 kPa for most organic clay and all peat mixtures. Only the strength of the organic clay stabilised with 50 kg Portland cement per cubic metre of undisturbed soil did not meet the required strength.

For the selection of the binder dosage it was only required that the measured strength exceeds the required unconfined compressive strength of 50 kPa. However, given that it is desired to mix as little binder as possible to reach the required strength of 50 kPa, the following binder dosages were selected:

- 50 kg Portland cement per cubic metre of undisturbed peat (5% m/m);
- 75 kg Portland cement per cubic metre of undisturbed organic clay (4% m/m).

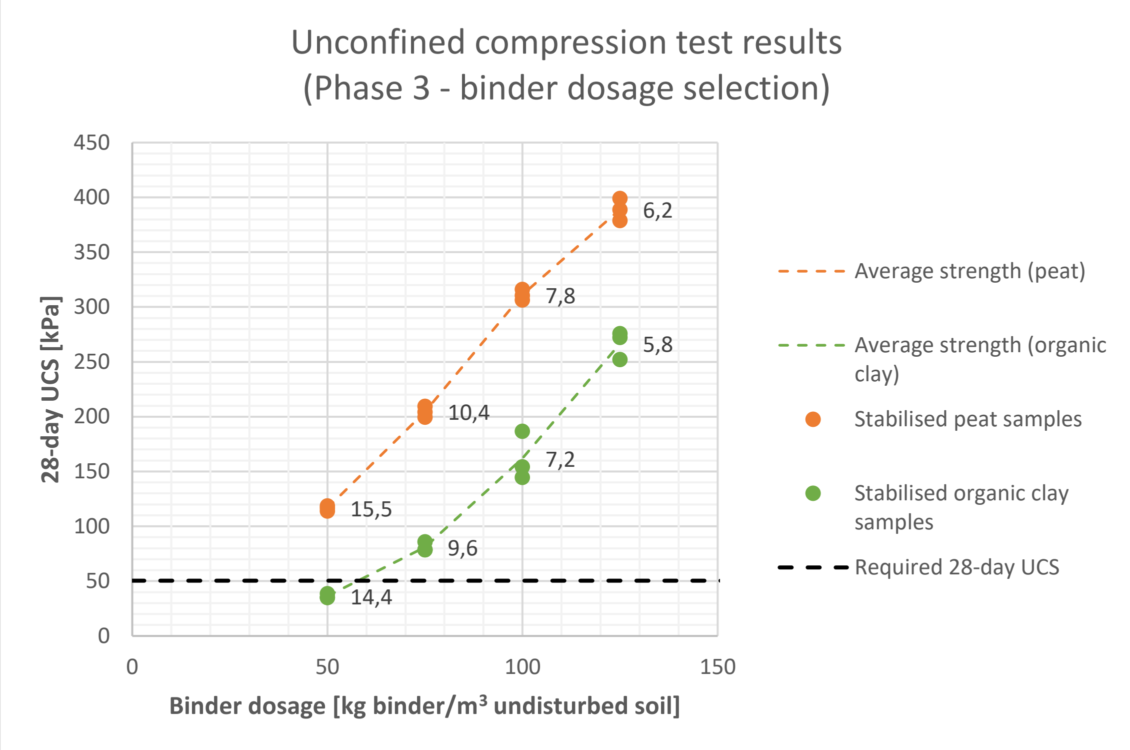

Figure 4.3; Unconfined compression test results from phase 3 of the laboratory research. The numbers in the graph represent the measured water-to-binder factor (w/b) for each produced mixture.

4.4 Curing curves (strength development)

The strength development for the peat and organic clay samples stabilised with respectively 50 and 75 kg Portland cement per cubic metre of undisturbed soil was determined in terms of unconfined compressive strength. In order to do this, two different methods were applied. The results of each method are presented in the next subsections.

4.4.1 Method 1 – multiple batches

During method 1 of phase 4 of the laboratory research, force-controlled unconfined compression tests were carried out on multiple batches of both peat and organic clay stabilised with respectively 50 and 75 kg Portland cement per cubic metre of undisturbed soil (i.e. respectively 5% m/m and 4% m/m). Each batch had cured for a different amount of days. The results of the unconfined compression tests carried out with the force rates listed in table 4.13 are presented in figure 4.4. The detailed test results are presented in appendix F. The unconfined compression tests were carried out with the same deviations from the procedure as listed in section 4.3.1.

Table 4.13; The applied force rates on the different stabilised soil samples during the unconfined compression tests.

Mixture	Force rate [N/s]
Peat + 50 kg CEM I/m^3 undisturbed peat	10
Organic clay + 75 kg CEM I/m^3 undisturbed organic clay	10

Upon examination of figure 4.4, it can be seen that there is no clear or physically logical trend in the strength development over time. This applies in particular to the peat mixture, where large differences in strength by both increases and reductions were measured between consecutive time steps. The trend in the organic clay mixture is already somewhat clearer, which seems to indicate that the strength developed until 2 to 7 days, but it cannot be concluded based on the measured strengths whether the end of curing was reached at 2 or 7 days of curing or somewhere in between.

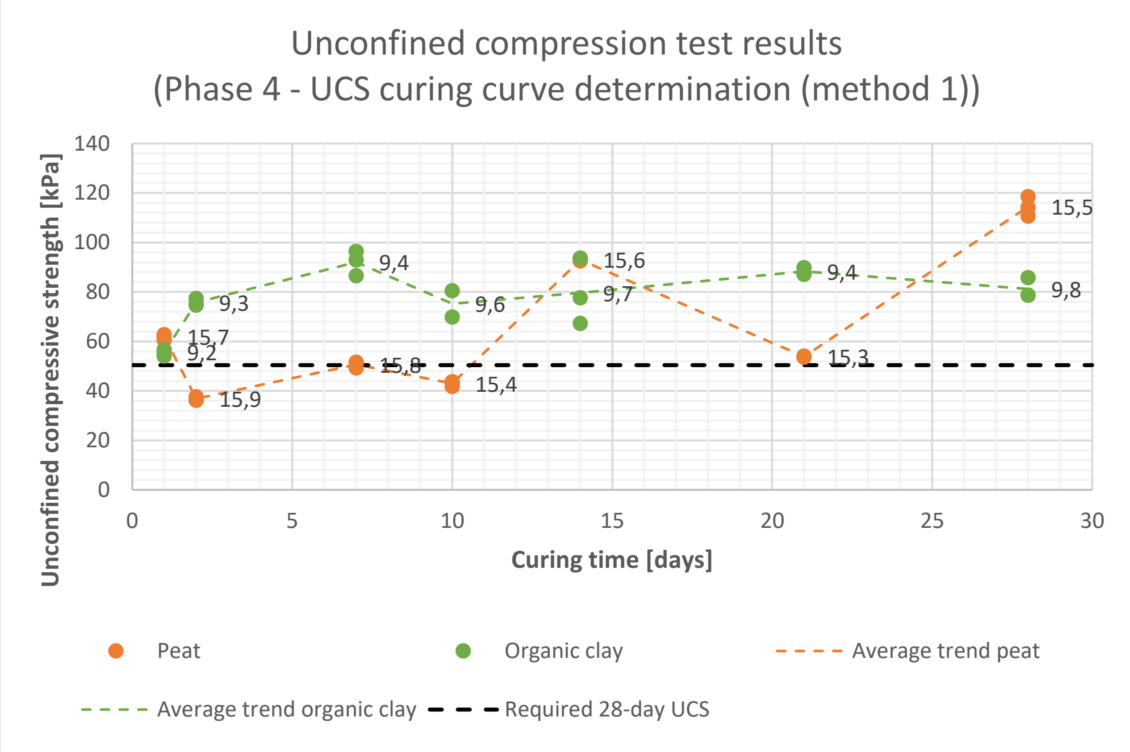

Figure 4.4; Unconfined compressive strengths for the peat and organic clay samples stabilised respectively with 50 and 75 kg Portland cement/m³ undisturbed soil during method 1 of phase 4 of the laboratory research. The numbers in the graph represent the measured water-to-binder factor (w/b) of each tested mixture.

At first it was thought that the water-to-binder factors could be used to explain the strange recorded strength differences between batches. In general, increasing the water-to-binder factor should result in weaker materials, whereas reducing the water-to-binder factor should result in stronger materials. When subsequently looking at figure 4.4, this seems to be recorded to some degree for the organic clay mixtures, but the opposite had been recorded for the peat mixtures. When comparing the water-to-binder factor of the peat mixture at 21 and 28 days, it can be seen that although the water-to-binder factor of the mixture left to cure for 21 days is lower than for the mixture left to cure for 28 days, the strength of the mixture cured for 21 days is smaller than the strength of the mixture cured for 28 days. A similar recording was made between the peat mixtures cured for respectively 7 and 10 days. Clearly, the water-to-binder factor cannot be used to explain the recorded strength differences.

Instead, other factors are responsible for the measured physically impossible consecutive increases and reductions in the strength in time. After a thorough discussion with laboratory technicians of Fugro NL Land B.V., the following possible explanations for the measured strengths were devised:

- Soil heterogeneity between the masses of excavated soil used for the production of the many batches of the same mixtures in the laboratory;
- (Minor) differences in the applied laboratory stabilisation procedure between batches.

The expected primary cause of these recorded differences in strength was the heterogeneity between the masses of the excavated soils used for the production of stabilised soil samples. Although the same masses of excavated soils were stabilised with the same masses of binder in the same way, it is not unlikely to think that one mass of soil had a slightly different composition or water content than other samples. This may be most applicable to the peat, where for example differences in the amount of wood chunks present in between batches could have been very different (this was not quantified for each peat sample in the laboratory). A consequence of these differences would be that one always inadvertently ends up with different mixtures, regardless of mixing the same quantities of material in the same manner. The result of this is that the strengths of the mixtures cannot be compared directly and thus cannot be used to construct the curing curves. However, if the soil heterogeneity is indeed the main reason why the large strength differences were measured, then this may have a significant impact on the stabilisation in the field. This is discussed in more detail in section 7.1.2.

Although the heterogeneity between the excavated soil samples was the most plausible explanation, it may also be possible that minor differences in the same applied laboratory soil stabilisation procedure were partly responsible for the observed differences. Although the same mixing times and mixing speeds were used for the production of the stabilised soil samples in phase 4, it may have been possible that the Portland cement was added to the soil faster than during another mixing procedure or that one mixture had been mixed slightly longer than another. This too, will inadvertently result in different mixtures with different strength properties.

Because of these two possible explanations, the strength results from figure 4.4 were obtained. Since these strength results are not suitable for finding a clear trend in the strength development over time for the examined peat and organic clay mixture, it was decided to redo phase 4 of the laboratory research. Instead of attempting to compare the strengths of slightly different mixtures produced according to the same recipe, it was decided to make a single batch of both the peat and organic clay mixture and determine the strength change of these batches only. Using this method, the effects of soil heterogeneity and slight variations in the mixing procedure on the obtained strength should have been minimised.

4.4.2 Method 2 – single batch

During method 2 of phase 4 of the laboratory research, force-controlled unconfined compression tests were carried out on a single batch of both peat and organic clay stabilised with respectively 50 and 75 kg Portland cement per cubic metre of undisturbed soil (i.e. respectively 5% m/m and 4% m/m). Each sample produced from this batch had cured for a different amount of days. The results of the unconfined compression tests carried out with the force rates listed in table 4.14 are presented in figure 4.5. The detailed test results are presented in appendix F. The unconfined compression tests were carried out with the same deviations from the procedure as listed in section 4.3.1.

Table 4.14; The applied force rates on the different stabilised soil samples during the unconfined compression tests.

Mixture	Force rate [N/s]
Peat + 50 kg CEM I/m^3 undisturbed peat	10
Organic clay + 75 kg CEM I/m^3 undisturbed organic clay	10

Upon examination of figure 4.5, it can be seen that a much clearer strength development can be observed compared to the results from method 1 (see figure 4.4). From figure 4.5 it was seen that the unconfined compressive strength seemed to develop logarithmically based on the relatively few measurements in time, which was in line with expectations due to the logarithmic strength increases found in literature (see section 2.4.1.3). It was seen that for both mixtures most of the strength seemed to develop during the first 48 hours of curing. Besides this, it seemed that for both mixtures the unconfined compressive strength developed mostly up to 7 days of curing, after which little to no additional strength with further curing was recorded. For the organic clay, also a minor reduction in the strength was recorded at 14 days of curing compared to the 7-day strength. This difference amounted to about 5 kPa and may be explained by slight compositional variations in the mixture. Although in general these few measurements of the strength of both mixtures already provided a much clearer trend in the strength development, making method 2 preferred over method 1, more measurements, at both the examined times and other times in between, are required to get a much better and more reliable strength development for both mixtures.

Since the strength was also found to be logarithmic, the same measured strengths were also plotted on a logarithmic timescale as shown in figure 4.6. Upon examination, it was seen that the recorded strengths could be approximated using two trend lines. One trend line matched the early-age strengths, where the cementation process was still ongoing, whereas the other trend line matched the final strengths, where the cementation process was hardly occurring anymore or had ended. The intersection between both trend lines is expected to show the time at which the curing process changed, indicating the cementation is ending and the material is approaching a fully-cured state. For both mixtures, this seemed to be around 6 or 7 days of curing based on the limited measurements of the strength. However, it should be noted that this not the definitive moment in time at which curing comes to a halt, since no measurements of the unconfined compressive strength had been made between 2 and 7 days of curing.

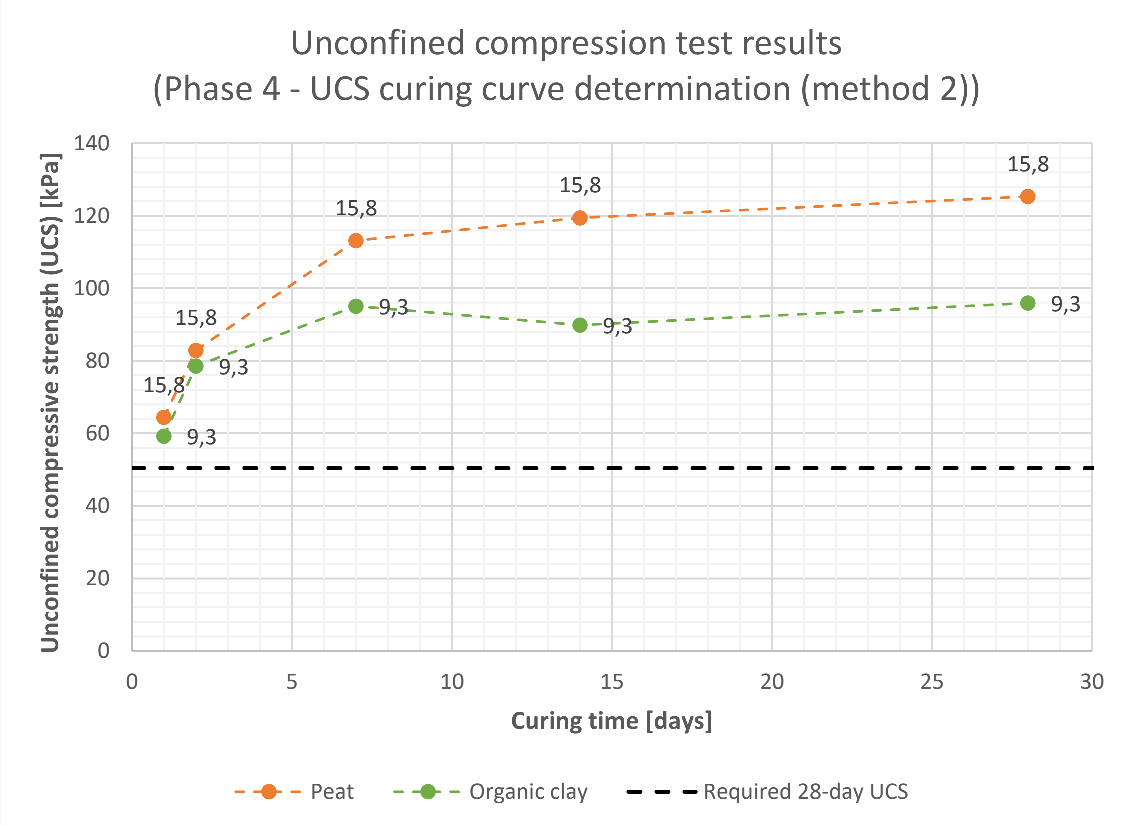

Figure 4.5; Unconfined compressive strengths for the peat and organic clay samples stabilised respectively with 50 and 75 kg Portland cement/m³ undisturbed soil during method 2 of phase 4 of the laboratory research. The numbers in the graph represent the measured water-to-binder factor (w/b) of each tested mixture.

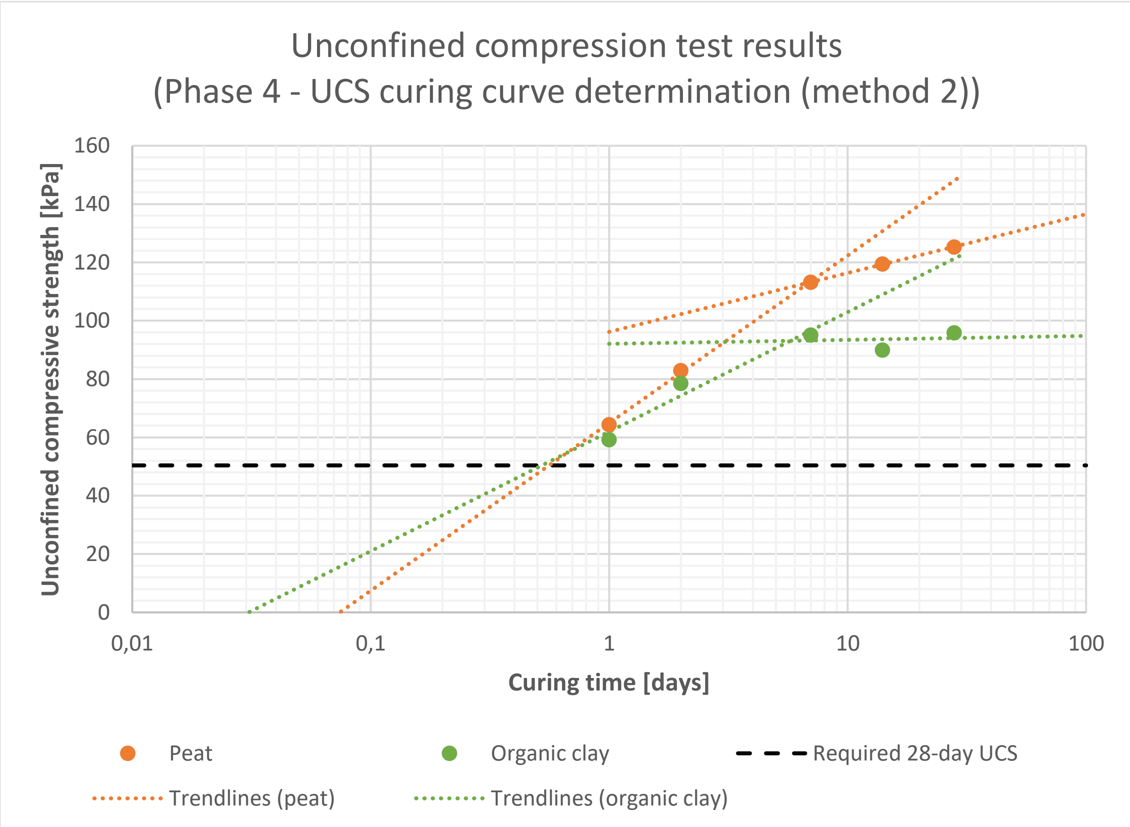

Figure 4.6; Curing curve of the peat and organic clay stabilised with respectively 50 and 75 kg Portland cement/m³ of undisturbed soil on a logarithmic time scale.

The results from figure 4.6 could perhaps also be used to obtain an indication on the time required for the cementation to start by extrapolating the tangent line matching the early-age strengths to an unconfined compressive strength of 0 kPa. For the peat mixture it seems that the chemical reactions that cause the cementation start at approximately 0,075 days. For the organic clay mixture this is at approximately 0,030 days. These two times correspond to about 105 and 45 minutes after mixing. Upon a comparison of these times with the times recorded in the product sheet of the applied Portland cement (see appendix G), it was found that cement in the organic clay mixture seemed to start binding about 60 minutes earlier than usual, whereas the same cement in the peat mixture seemed to start binding at the same average time as written on the product sheet. Although these extrapolations provide an indication on the start of binding of the Portland cement in the mixtures, more strength measurements are required to obtain a more reliable indication on the start of binding.

4.5 Strength comparison undisturbed and stabilised soil

During phase 1 and phase 5, the strength properties of respectively the undisturbed peat and organic clay samples and the peat and organic clay samples stabilised with respectively 50 and 75 kg Portland cement per cubic metre of undisturbed soil were determined. In this section, these strength properties are compared between the undisturbed soil samples and their stabilised counterparts.

4.5.1 Peat and stabilised peat

For both the undisturbed peat and the stabilised peat, unconfined compression tests and shearbox tests were carried out to determine the unconfined compressive strength and the effective strength parameters. The comparison of both strength parameters is presented in the next subsections.

4.5.1.1 Unconfined compressive strength

One undisturbed peat and one stabilised peat sample cured for 28 days were subjected to an unconfined compression test. The undisturbed peat sample was subjected to a strain-controlled unconfined compression test in a triaxial cell without filling the cell with water. The unconfined compression test was carried out according to the procedure outlined in Dutch standard NEN-EN-ISO 17892-7 (*'Unconfined compression test'*). The triaxial cell was used for the undisturbed peat sample, because the compression machine could not accurately measure in the range of the expected force required to bring the undisturbed peat sample to failure. The stabilised peat sample on the other hand was subjected to a force-controlled unconfined compression test in a compression machine and was carried out with the same deviations from the procedure outlined in NEN-EN-ISO 17892-7 as listed in section 4.3.1. The results of the unconfined compression tests carried out with the rates listed in table 4.15 are presented in table 4.16. The recorded stress-strain responses are presented figure 4.7.

Besides these measurements, also measurements of the sample dimensions and a number of soil parameters for both tested samples were made in accordance with NEN-EN-ISO 17892-7. These measurements are presented in appendix F.

Table 4.15; The applied force and strain rate on the different samples during the unconfined compression tests.

Sample	Force rate	Strain rate
	[N/s]	[%/h]
Undisturbed peat	-	43,6
Peat + 50 kg CEM I/m^3 undisturbed peat	10	-

Table 4.16; The unconfined compression test results for the undisturbed peat sample and the stabilised peat sample after 28 days of curing.

Mixture	UCS	ε_f
	[kPa]	[%]
Undisturbed peat	16	10,3
Stabilised peat (Peat + 50 kg CEM I/m^3 undisturbed peat)	115	4,3

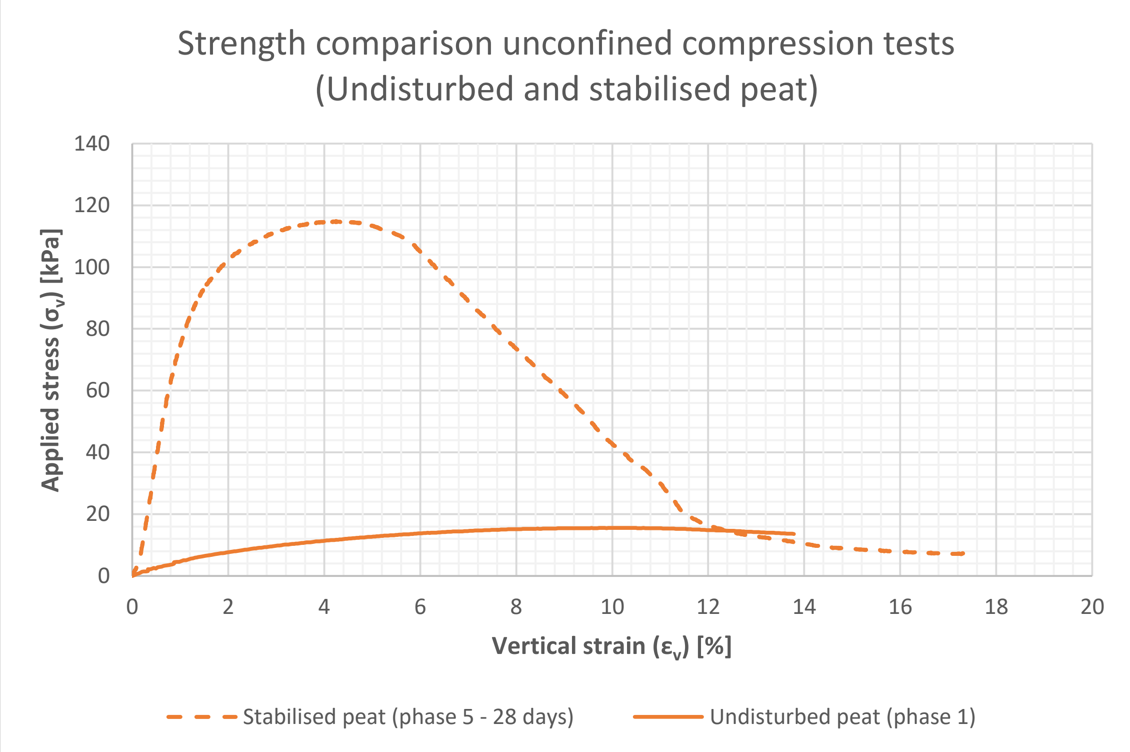

Figure 4.7; Stress-strain response comparison between the undisturbed peat sample and the peat sample stabilised with 50 kg Portland cement/m³ undisturbed peat. The stabilised peat sample had cured for 28 days.

Upon examination of figure 4.7, it can be seen that very different stress-strain responses were recorded between the tested undisturbed peat sample and stabilised peat sample. The following was noted from the graph:

- The maximum stress that could be applied (i.e. unconfined compressive strength) on the stabilised peat sample is about 7 times larger than the maximum stress that could be applied on the undisturbed peat sample;
- The peak stress for the stabilised peat sample is reached at much lower strains than the peak stress for the undisturbed peat sample (i.e. the strength of the stabilised peat is mobilised faster than the strength of the undisturbed peat);
- After the peak stress for the stabilised peat sample is exceeded, a big reduction in strength is recorded.

All three recorded matters indicate that the stabilised peat sample is much more brittle than the undisturbed peat sample. This was also visually observed during the unconfined compression tests, with the stabilised peat sample breaking instead of bulging upon loading.

4.5.1.2 Drained shear strength parameters

Four undisturbed peat samples and four stabilised peat samples cured under load for 28 days were subjected to shearbox tests according to the procedure outlined in Dutch guideline NEN-EN-ISO 17892-10 (*'Direct shear tests'*). The undisturbed peat samples were consolidated under the normal stresses from table 4.17 for about 8 hours and then sheared to failure with the displacement rates listed in table 4.17. On the other hand, the stabilised peat samples were consolidated under the same normal stresses for about 24 hours and then sheared to failure with the displacement rates listed in table 4.17. The measured shear stress – shear strain responses for all tested undisturbed and stabilised peat samples are presented in figure 4.8.

Besides these measurements, also measurements of the sample dimensions and a number of soil parameters and other parameters for all tested samples were made in accordance with NEN-EN-ISO 17892-10. These measurements, along with all other details of the shearbox tests carried out, are presented in appendix F.

Although the stabilised peat samples were extruded from the moulds after 28 days of curing under load, the samples were not subjected to shearbox tests on the same day. The samples first had to be transported from the geotechnical laboratory of Fugro NL Land B.V. in Arnhem to the geotechnical laboratory of Delft University of Technology in Delft. After that, the shearbox tests had to be carried out one by one since only one shearbox test setup was available. As a result, all shearbox tests on the stabilised peat samples were carried out between day 29 and day 37. The amount of time the stabilised peat samples had cured at the start of the shearbox test (consolidation stage) and at the end of the shearing stage are presented in table 4.17. Even though each tested stabilised peat sample had cured a different amount of time, it was expected that the strength of the samples was very similar. Since the strength of the examined stabilised peat mixture was found to be hardly increasing between 14 and 28 days of curing (see figure 4.5), it was expected that hardly any additional strength would have developed after 28 days of curing.

Table 4.17; The applied normal stresses and displacement rates on the different samples during the shearbox tests. The curing time for each tested stabilised peat sample at the start and end of the shearbox test are indicated as well.

Sample	Applied normal stress [kPa]	Rate of displacement [mm/min]	Curing time at start of test [days]	Curing time at end of shearing [days]
Undisturbed peat	16,0 (field stress)	0,013	N/A	N/A
	60,0	0,013	N/A	N/A
	120	0,013	N/A	N/A
	135	0,013	N/A	N/A
Stabilised peat (Peat + 50 kg CEM I/m^3 undisturbed peat)	16,0 (field stress)	0,050	31	32
	60,0	0,050	30	31
	120	0,050	29	30
	135	0,050	36	37

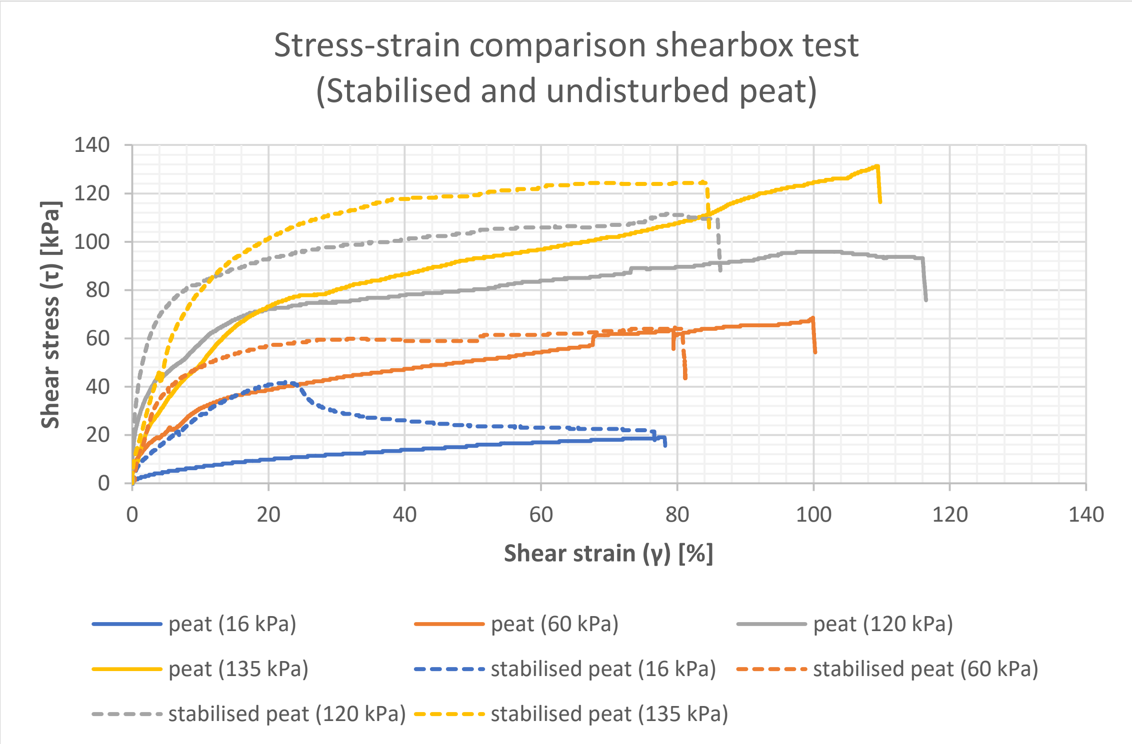

Figure 4.8; The shear stress-shear strain response of the undisturbed and stabilised peat samples as measured during the shearing stage of the shear box tests at four different normal stresses.[1]

[1] The shear strain here is defined as the ratio between the horizontal displacement measured during the test and the sample height after consolidation, expressed as a percentage (Powrie, 2004). An interpretation of the shearbox test similar to a direct simple shear test (DSS test) was applied, as the strength interpretation of the stabilised peat had to be done at a specified shear strain to remain consistent. Since no DSS test setup was available to the author, this interpretation of the shearbox test measurements was made.

Upon examination of the shear stress – shear strain responses from figure 4.8, a number of things were noted. First of all, it can be seen from figure 4.8 that for each stabilised peat sample larger shear stresses were needed to shear the sample compared to the undisturbed peat samples sheared under the same normal stresses. This was a logical result, as the result implied that the stabilised peat samples were stronger and more resistant to shearing than the undisturbed peat samples. This result was desirable, as the intent was to improve the strength of the peat by means of stabilisation.

Secondly, all undisturbed and most stabilised peat samples show that an increasing shear stress is needed to shear the sample with increasing shear strain. However, the stabilised peat sample sheared under 16 kPa normal stress shows a different shear stress – shear strain response. When examining figure 4.9, it can be seen that this stabilised peat sample dilated during shearing, whereas all other stabilised peat samples contracted to varying degrees during shearing. A reason for this different response may be that this stabilised peat sample was sheared under a normal stress lower than the pre-consolidation pressure. The stabilised peat sample sheared under 16 kPa would then behave as an overconsolidated soil, explaining the dilative response and the observed peak in shear stress. It was not unlikely to assume that the pre-consolidation stress was larger than 16 kPa, as the stabilised peat sample had cured under 25 kPa load, thereby justifying the assumption on overconsolidated soil behaviour.

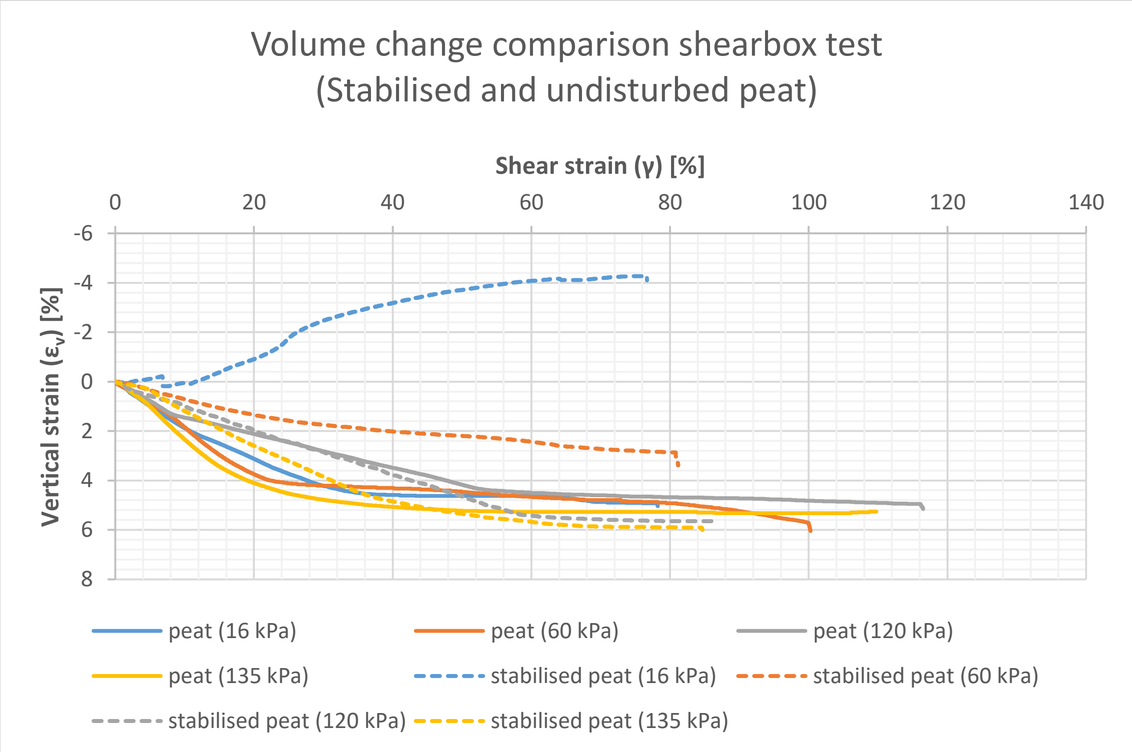

Figure 4.9; The vertical strain of the undisturbed and stabilised peat samples with increasing shear strain as measured during the shearing stage of the shear box tests at four different normal stresses.

Subsequently, all shear stress – shear strain responses of both the undisturbed and stabilised peat samples from figure 4.8 were used to derive the mobilisation of the effective strength parameters of both the undisturbed and stabilised peat. The obtained mobilisation of the effective strength parameters of both the undisturbed and stabilised peat is presented in figure 4.10.

It can be seen from figure 4.10 that the mobilisation of the effective cohesion of the stabilised peat has a shape similar to the shear stress response of the stabilised peat sheared under 16 kPa normal stress. Clearly, the overconsolidated soil behaviour of that particular stabilised peat sample heavily influences the obtained mobilisation of the effective cohesion. Similarly, the mobilisation of the effective cohesion of the undisturbed peat at large shear strains is affected by the sudden jump in the shear stress – shear strain response of the undisturbed peat sample sheared under 60 kPa normal stress (see figure 4.8). This jump may have been caused by shearing over a small piece of wood.

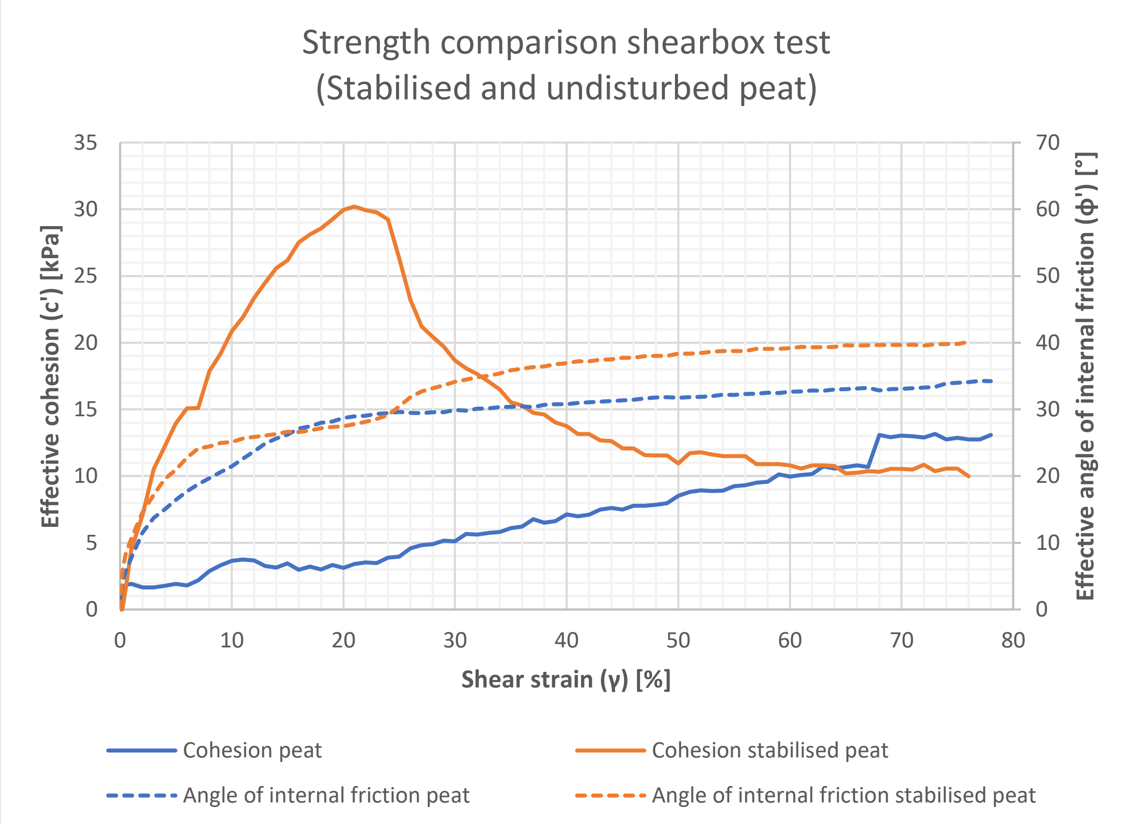

Figure 4.10; Comparison of the mobilisation of the effective cohesion and the effective angle of internal friction for the undisturbed and (fully) cured stabilised peat samples subjected to single stage shearbox tests.

When analysing the mobilisation of the effective strength parameters from figure 4.10, it can be seen that the effective cohesion of the stabilised peat is mobilised faster and to larger values than the effective cohesion of the undisturbed peat. After the maximum effective cohesion of about 30 kPa was mobilised at about 20% shear strain, the effective cohesion dropped with increasing shear strain. On the other hand, the effective angle of internal friction of both the undisturbed and stabilised peat mobilised similarly to similar values. Lastly, it seemed that at shear strains exceeding 60%, the effective cohesion and angle of internal friction of the stabilised peat and the undisturbed peat are similar, indicating similar strengths.

Subsequently, the effective strength parameters of the stabilised peat were determined at 5% strain in accordance with the Dutch STOWA guideline and to keep consistent with the effective strength parameters of the undisturbed peat that were determined at 5% shear strain from direct simple shear tests (see appendix B). This selection also fitted well for the stabilised peat, as at 5% shear strain not all strength had been mobilised yet (see figure 4.8 and figure 4.10). The measured unconfined compressive strength and effective strength parameters of the stabilised peat were subsequently compared to the strength requirements set for the stabilised peat as shown in table 4.18.

Table 4.18; The comparison of the strength requirements and the measured strengths for the stabilised peat.

Soil parameters	Option 1	Option 2	Option 3	Option 4	Measured	Unit
c'	≥ 4,4	≥ 8,7	≥ 13,1	≥ 17,4	13,9	[kPa]
ϕ'	≥ 36,1	≥ 30,5	≥ 24,7	≥ 18,7	20,9	[°]
UCS	≥ 17	≥ 30	≥ 41	≥ 49	115	[kPa]

From table 4.18 can be seen that the measured unconfined compressive strength meets the requirements of all four options, whereas the measured combination of the effective strength parameters does not meet any of the options. Although the measured effective strength parameters differs, possibly due to a too low binder dosage applied, the measured combination of the effective strength parameters was still sufficiently high to reinforce the levee. This is discussed further in section 4.5.2.2.

4.5.2 Organic clay and stabilised organic clay

For both the undisturbed organic clay and the stabilised organic, unconfined compression tests and isotropically consolidated undrained triaxial compression tests (CIUc triaxial tests) were carried out to determine the unconfined compressive strength and the effective strength parameters. The comparison of both strength parameters is presented in the next subsections.

4.5.2.1 Unconfined compressive strength

One untreated organic clay and one stabilised organic clay sample cured for 28 days were subjected to an unconfined compression test. Like with the peat and stabilised peat sample, the untreated organic clay sample was subjected to a strain-controlled unconfined compression test in a triaxial cell, whereas the stabilised organic clay sample was subjected to a force-controlled unconfined compression test in a compression machine. This was done for the same reasons as listed in section 4.5.1.1. The strain-controlled unconfined compression test was carried out according to the procedure outlined in Dutch standard NEN-EN-ISO 17892-7 without filling the triaxial cell with water. The force-controlled unconfined compression test was carried out with the same deviations from the procedure outlined in NEN-EN-ISO 17892-7 as listed in section 4.3.1. The results of the unconfined compression tests carried out with the rates listed in table 4.19 are presented in table 4.20. The recorded stress-strain responses are presented figure 4.11.

Besides these measurements, also measurements of the sample dimensions and a number of soil parameters for both tested samples were made in accordance with NEN-EN-ISO 17892-7. These measurements are presented in appendix F.

Table 4.19; The applied force and strain rate on the different samples during the unconfined compression tests.

Sample	Force rate	Strain rate
	[N/s]	[%/h]
Undisturbed organic clay	-	44,6
Organic clay + 75 kg CEM I/m^3 undisturbed organic clay	10	-

Table 4.20; The unconfined compression test results for the undisturbed organic clay sample and the stabilised organic clay sample after 28 days of curing.

Mixture	UCS	ε_f
	[kPa]	[%]
Undisturbed organic clay	24	11,5
Stabilised organic clay (Organic clay + 75 kg CEM I/m^3 undisturbed organic clay)	67	3,6

Upon examination of figure 4.11, it can be seen that very different stress-strain responses were recorded between the tested undisturbed organic clay and stabilised organic clay sample. The following was noted from the graph:

- The maximum stress that could be applied (i.e. unconfined compressive strength) on the stabilised organic clay sample is about 3 times larger than the maximum stress that could be applied on the undisturbed organic clay sample;
- The peak stress for the stabilised organic clay sample is reached at much lower strains than the peak stress for the undisturbed organic clay sample (i.e. the strength of the stabilised organic clay is mobilised faster than the strength of the undisturbed organic clay);
- After the peak stress for the stabilised organic clay sample is exceeded, a big reduction in strength is recorded.

All three recorded matters indicate that the stabilised organic clay sample is much more brittle than the undisturbed organic clay sample. This conclusion was also found for the stabilised peat sample in comparison to the undisturbed peat sample (see section 4.5.1.1).

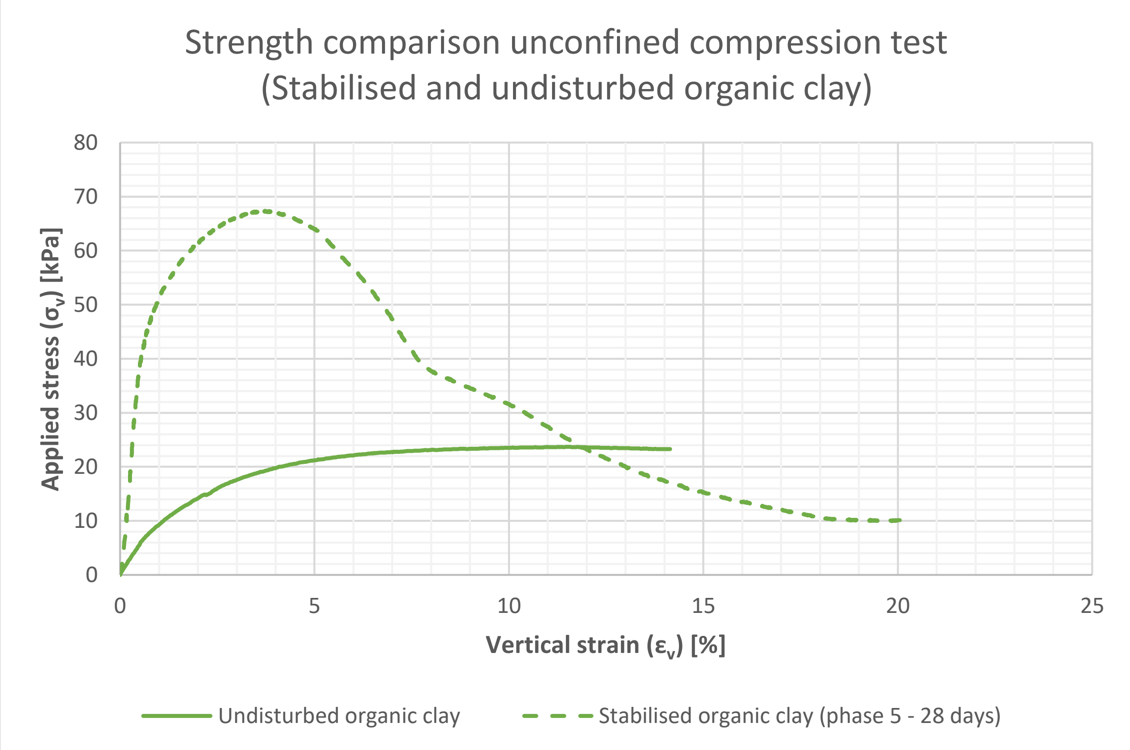

Figure 4.11; Stress-strain response comparison between the undisturbed organic clay and the organic clay stabilised with 75 kg Portland cement/m³ undisturbed organic clay. The stabilised organic clay sample had cured for 28 days.

4.5.2.2 Drained shear strength parameters

Three undisturbed organic clay and three stabilised organic clay samples cured for 28 days were subjected to single stage isotropically consolidated undrained triaxial compression tests (CIUc triaxial tests) according to the procedure outlined in Dutch guideline NEN-EN-ISO 17892-9 (*'Consolidated triaxial compression tests on water saturated soil'*). Both the undisturbed and stabilised organic clay samples were first saturated, then left to consolidate under the isotropic consolidation stresses from table 4.21 for about 24 hours and finally brought to failure with the strain rates listed in table 4.21. As a result of this procedure, all three stabilised organic clay samples were built in the triaxial cells on day 28, but brought to failure on day 30. This was not expected to be problematic, as all stabilised organic clay samples were tested simultaneously. In addition, it was expected that the 28-day and 30-day strength would be similar, as the examined organic clay mixture hardly gained any more strength between 7 and 28 days of curing (see figure 4.5).

The measured stress-strain responses and stress paths recorded for all samples during the CIUc triaxial tests are presented figure 4.12 and figure 4.13 respectively. Besides these measurements, also measurements of the sample dimensions and a number of soil parameters and other parameters for all tested samples were made in accordance with NEN-EN-ISO 17892-9. These measurements, along with all other details of the CIUc triaxial tests carried out, are presented in appendix F.

Table 4.21; The applied consolidation stresses and strain rates on the different samples during the CIUc triaxial tests.

Sample	Applied isotropic consolidation stress [kPa]	Strain rate [%/h]
Undisturbed organic clay	15,0 (field stress)	3,0
	60,0	3,0
	120	2,4
Stabilised organic clay (Organic clay + 75 kg CEM I/m³ undisturbed organic clay)	15,0 (field stress)	2,0
	60,0	2,0
	120	1,9

Upon examination of the stress-strain responses from figure 4.12, it was seen that the deviator stress that is needed to shear each stabilised organic clay sample is larger than the deviator stress needed to shear each undisturbed organic clay sample when comparing samples consolidated under the same stresses. This was a logical result, as the result implied that the stabilised organic clay samples were stronger and more resistant to shearing than the undisturbed organic clay samples. This result was desirable, as the intent was to improve the strength of the organic clay by means of stabilisation.

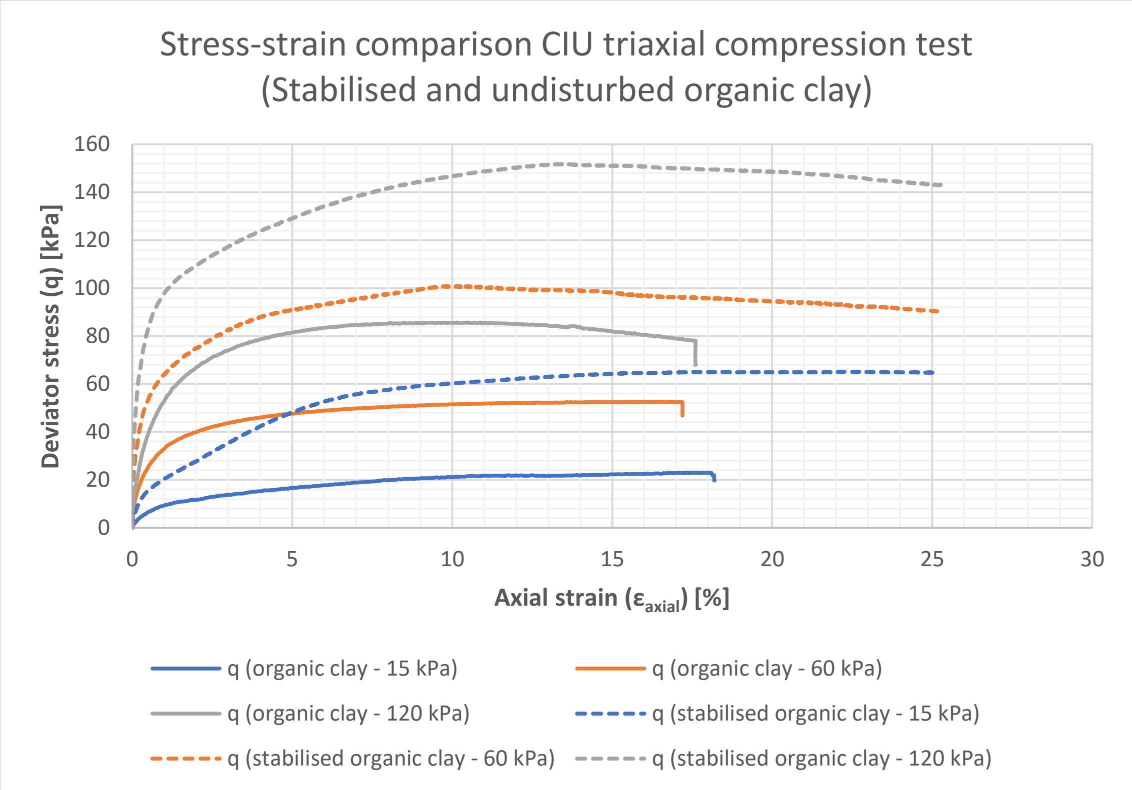

Figure 4.12; The stress-strain response of the undisturbed and stabilised organic clay samples as measured during the shearing stage of the CIUc triaxial tests at three different isotropic consolidation stresses.

When subsequently examining figure 4.13, it can be seen that the stress paths of the undisturbed and stabilised organic clay differed a lot. The undisturbed organic clay samples consolidated under 60 kPa and 120 kPa stress showed a stress path bending to the left. This implied that these undisturbed organic clay samples showed contractive soil behaviour during shearing. On the other hand, the undisturbed organic clay sample consolidated under 15 kPa stress showed a stress path bending neither to the left nor to the right, thereby showing neither contractive nor dilative soil behaviour during shearing. The fact that this different behaviour was observed, was likely caused by shearing a sample that had consolidated under a stress that was less than the pre-consolidation stress.

Similar to these observations for the undisturbed organic clay, it was also found that the stabilised organic clay showed different stress paths when consolidated under different stresses. The stabilised organic clay samples consolidated under 60 kPa and 120 kPa stress showed a stress path first bending to the left, after which the stress paths bended to the right. This implied these stabilised organic clay samples first showed contractive soil behaviour up to about 2% axial strain, with increasing pore water pressures during shearing, after which the samples showed dilative soil behaviour from about 2% axial strain and onwards, with decreasing pore water pressures during shearing. On the other hand, the stabilised organic clay sample consolidated under 15 kPa stress showed a stress path solely bending to the right. This implies that this stabilised organic clay sample showed purely dilative soil behaviour. Like with the undisturbed organic clay samples, these differences in behaviour during shearing were likely caused by shearing the samples either above or below the pre-consolidation stress.

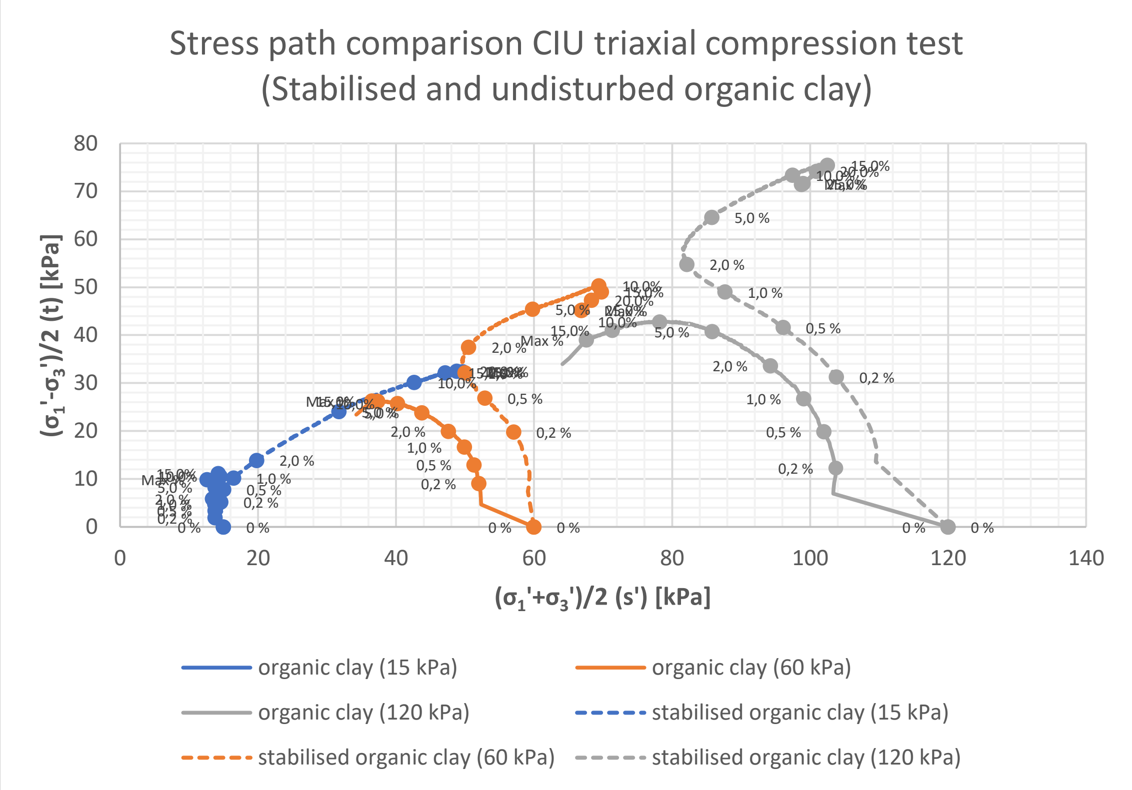

Figure 4.13; The stress paths of the undisturbed and stabilised organic clay samples as measured during the shearing stage of the CIUc triaxial tests at three different isotropic consolidation stresses. The numbers in the graph indicate the axial strain at various points on the graphs.

Subsequently, all stress paths of both the undisturbed and stabilised organic clay samples from figure 4.13 were used to derive the mobilisation of the effective strength parameters of both the undisturbed and stabilised organic clay. The obtained mobilisation of the effective strength parameters of both the undisturbed and stabilised organic clay is presented in figure 4.14.

Upon examination of figure 4.14, it was seen that both the effective cohesion and the effective angle of internal friction of the stabilised organic clay mobilised much faster to much larger values than both effective strength parameters of the undisturbed organic clay. The effective angle of internal friction of the stabilised organic clay mobilised to about 50° at about 6% axial strain after which a slight gradual drop to about 45° with increasing axial strain was measured. The angle of internal friction of the stabilised organic clay was quite large compared to the undisturbed organic clay, with increases between 15 to 20° having been measured depending on the axial strain one is looking at.

On the other hand, it was seen that the effective cohesion of the stabilised organic clay of about 3,0 kPa had already been fully mobilised at 1% axial strain, after which the effective cohesion dropped to zero with increasing axial strain. The 3,0 kPa effective cohesion of the stabilised organic clay was unexpectedly low compared to the effective cohesion mobilised for the undisturbed organic clay. It was expected that this low mobilisation of the effective cohesion was the direct result of using all three measured stress paths of the stabilised organic clay samples to derive the mobilisation of the effective strength parameters with. After all, at similar axial strains the stabilised organic clay samples showed different behaviour when consolidated under different stresses. Because of this observation, it was concluded that the consolidation stresses could have been chosen more carefully, perhaps more near field stress (15 kPa), to obtain a mobilisation of the effective cohesion more in line with expectations. Another possibility would have been to carry out more CIUc triaxial tests on stabilised organic clay samples consolidated under different stresses, such that a distinction in the mobilisation of the effective strength parameters for the stabilised organic clay below and above the pre-consolidation stress could have been made.

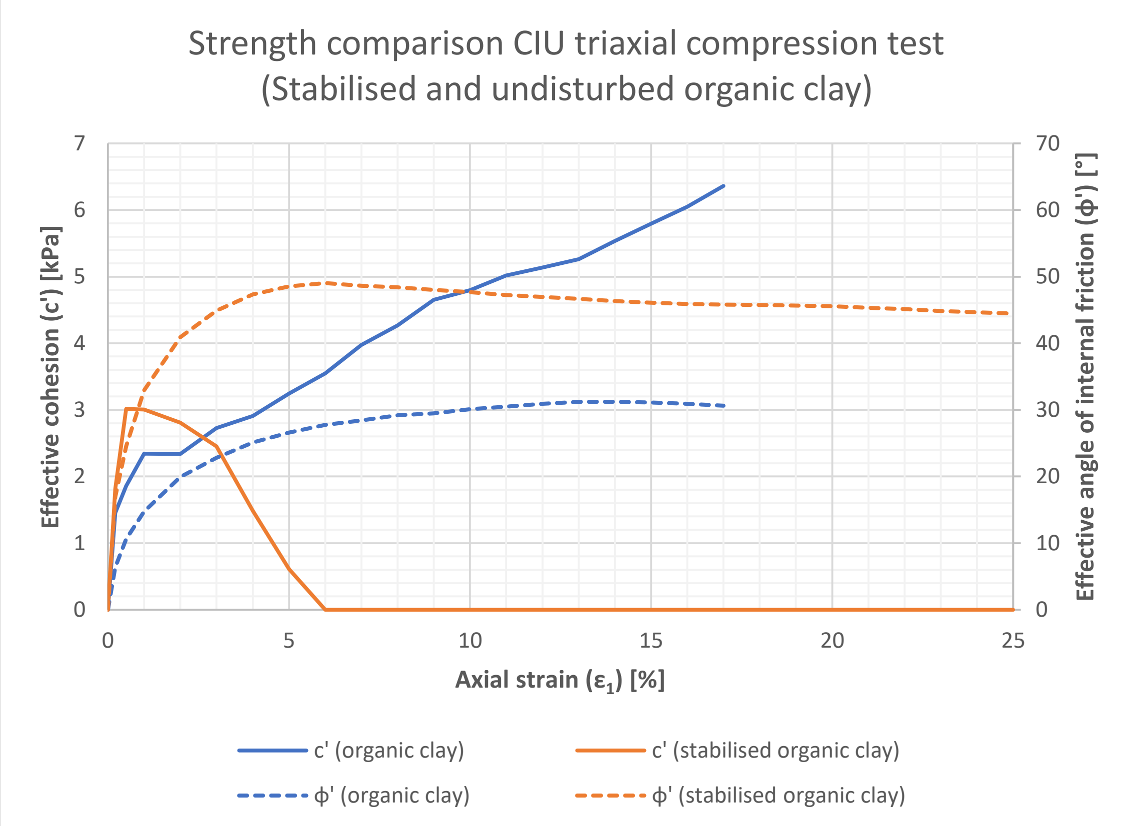

Figure 4.14; Comparison of the mobilisation of the effective cohesion and the effective angle of internal friction for the undisturbed and 28-day (fully) cured stabilised organic clay samples subjected to single stage isotropically consolidated undrained triaxial compression tests.

Subsequently, the effective strength parameters of the stabilised organic clay were determined at 2% axial strain in accordance with the Dutch STOWA guideline and to keep consistent with the effective strength parameters of the undisturbed organic clay that were determined at 2% axial strain from consolidated undrained triaxial tests (see appendix B). This selection also fitted well for the stabilised organic clay, as at 2% axial strain not all strength had yet been mobilised and the peak strength had not yet been exceeded (see figure 4.12). The measured unconfined compressive strength and effective strength parameters of the stabilised organic clay were subsequently compared to the strength requirements set for the stabilised organic clay as shown in table 4.22.

Table 4.22; The comparison of the strength requirements and the measured strengths for the stabilised organic clay.

Soil parameters	Option 1	Option 2	Option 3	Option 4	Measured	Unit
c'	≥ 4,4	≥ 8,7	≥ 13,1	≥ 17,4	2,8	[kPa]
ϕ'	≥ 39,1	≥ 33,3	≥ 27,1	≥ 20,7	40,9	[°]
UCS	≥ 19	≥ 32	≥ 43	≥ 50	67	[kPa]

From table 4.22 can be seen that the measured unconfined compressive strength meets the requirements of all four options, whereas the measured combination of the effective strength parameters does not meet any of the options. Trial stability calculations for the levee with stabilised soil at the toe (see figure 3.5) were carried out to determine whether the measured effective strength parameters were still sufficient. In these calculations, design values of the effective strength parameters of the stabilised peat and stabilised organic clay were used that were derived from table 4.18 and table 4.22 using the partial material factors and coefficients of variation from table 3.7. The results of the stability calculations showed that the required increase in the Factor of Safety could still be achieved using the measured effective strength parameters. Therefore, these combinations of the effective strength parameters were used to assess the practicability of mass stabilisation at levees, despite the fact that the required combinations of the effective strength parameters of both stabilised soils were not met.

4.6 Conclusion

Laboratory research was conducted in which the strength properties of an undisturbed and stabilised peat and organic clay were examined. The purpose of the laboratory research was to determine whether the required strength could be reached by stabilising the peat and organic clay sampled near the examined levee, as well as to determine how the strength of these two soils changed in time due to stabilisation. To examine this, the following sub-question was formulated:

‘How do the strength properties of the soil(s) to be stabilised from the selected case change as a result of stabilisation with a preselected binder and dosage?’

In the geotechnical laboratory of Fugro NL Land B.V., it has been examined which binder should be applied in which dosage to stabilise the peat or the organic clay to meet the required unconfined compressive strength of 50 kPa. Based on the results of the laboratory research, the following binder recipes have been selected:

- The stabilisation of peat with 50 kg rapid-curing Portland cement (CEM I 52,5 R) per cubic metre of undisturbed peat (i.e. corresponding to a dosage of 50 kg CEM I per 990 kg peat (5% m/m));
- The stabilisation of organic clay with 75 kg rapid-curing Portland cement (CEM I 52,5 R) per cubic metre of undisturbed organic clay (i.e. corresponding to a dosage of 75 kg CEM I per 1300 kg organic clay (4% m/m)).

On the basis of a select few measurements, the unconfined compressive strength of these two stabilised soils has been measured to have increased logarithmically up to about 7 days of curing, after which the strength increased relatively little up to 28 days of curing, if at all. Increases in the unconfined compressive strength from 16 to 125 kPa and from 24 to 96 kPa have been achieved for the examined stabilisation of respectively the peat and the organic clay, thus meeting the required strength of 50 kPa for both stabilisations. The stress-strain responses from the unconfined compression tests have also shown that these soil strength increases caused by stabilisation were accompanied by more brittle soil behaviour.

The effective strength parameters of both stabilised soils has been determined after 28 days of curing at either 2% axial strain or 5% shear strain in compliance with the Dutch STOWA guideline. However, the measured combination of the effective strength parameters of both stabilised soils has not met any of the required combinations of the effective strength parameters. This has been caused by either a too low binder dosage or by an improper selection of the consolidation stresses. Regardless, trial stability calculations with stabilised soil at the toe of the examined levee have shown that the measured combinations of the effective strength parameters were still sufficiently high to allow reinforcement of the levee.

5 Implementation analyses

5.1 Introduction

The technical feasibility of applying mass stabilisation for reinforcing levees is dependent on among others the practicability of the technique at levees. To examine this, the following sub-question was formulated:

'Is the application of mass stabilisation at the levee of the selected case practicable?'

In order to answer this sub-question, a single execution method for the stabilisation of the soil at the toe of the levee at the Montfoortse Vaart is examined. The approach to determining the feasibility of the examined execution method is described in section 5.2. Subsequently, the results of the implementation analyses are presented in section 5.3. As a consequence of the selected execution method and the obtained results on the examined execution, a few points of attention have been drawn up in section 5.4. Conclusions were then drawn in section 5.5 from the results to answer the above sub-question.

5.2 Analyses approach

In this section, the approach to determining the practicability of mass stabilisation at the levee at the Montfoortse Vaart using two-dimensional stability analyses is presented.

5.2.1 Examined implementation

The execution of mass stabilisation for the reinforcement of the levee at the Montfoortse Vaart was examined for the stabilisation of the soil at the toe of the levee (see figure 3.5) by a generic method of execution. For the determination of the practicability a fixed 25 metre long section of levee was considered. The 25 metres was selected as this is the typical width of a critical slip surface at regional flood defences (Stichting Toegepast Onderzoek Waterbeheer, 2015b).

The examined method of execution was a continuous stabilisation of 1,0 metre wide adjacent blocks of soil to a strip of stabilised soil as schematically shown in figure 5.1, which was modelled as follows based on section 2.2.4:

1. Stabilisation of a 1,0 metre wide block of soil (with length and depth from figure 3.5);
2. Application of a preload of 8,0 kPa on top of the stabilised soil (this preload was also considered during the laboratory research, see section 4.2.3.2);
3. Repetition of steps 1 and 2 until either:
    - The entire section of 25 metres is stabilised;
    - Further stabilisation would result in an unacceptably low Factor of Safety, jeopardising the stability of the levee during execution. In this case, a curing period of 24 hours was applied, after which steps 1, 2 and 3 were repeated;
4. The stabilised soils are cured under the preload until the desired strength is reached;
5. All preload is removed from all stabilised soils.

The following preconditions have been imposed for this generic method of execution:

- After every stabilisation of a 1,0 metre wide block of soil, a preload is applied on top of this block before the next block is stabilised;
- Below and around every block of stabilised soil 0,5 metre of remoulded soil is modelled, keeping into account the remoulding of the surrounding soil as a result of the in-situ mixing;
- In the event a 24-hour curing period is to be applied during the execution, because further stabilisation would otherwise result in unacceptable low Factors of Safety, 0,5 metre of soil is left untreated between the already stabilised and partially cured soil and the soil to be stabilised. This was included to prevent remoulding of the hardened soil upon adjacent stabilisation.

As a result of time restrictions, no variations or optimisations in the order of stabilisation or the size of the blocks stabilised at a time were considered in this research.

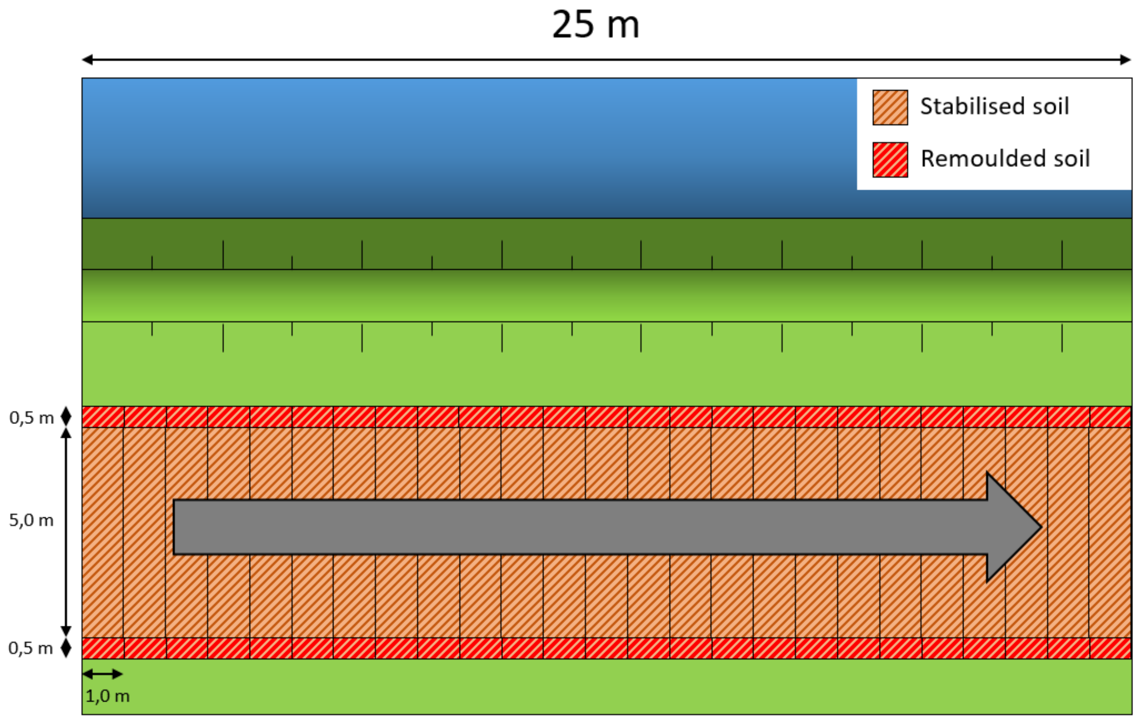

Figure 5.1; Top view of the examined execution method for mass stabilisation at the levee: a continuous stabilisation of 1,0 metre wide blocks of soil to a strip of stabilised soil over the examined section of 25 metres of levee.

5.2.2 Examined scenarios

In order to examine the feasibility of the examined method of execution described in section 5.2.1, a single two-dimensional stability analysis was carried for the 25 metres of levee per step taken during execution using weighted averages of the strength, density and preload over the considered section of 25 metres. The feasibility of the examined method of execution can be demonstrated by determining whether:

- The Factor of Safety during the execution does not fall below minimum allowable Factor of Safety from table 5.1. This minimum Factor of Safety was set equal to the Factor of Safety obtained in the initial situation at high water conditions (see table 3.1 from section 3.2.1.1);
- The required Factor of Safety at high water conditions from table 5.1 can be reached using the results of the laboratory research.

To examine the feasibility of the examined method of execution, four different scenarios were considered. In these four scenarios the assumptions on two main variables were varied: the density and the initial strength after stabilisation. This resulted in the four scenarios as shown in table 5.2. These two parameters were varied as these mostly influence the execution. It is unknown how strong the mixed soil initially is, as this could not be measured in the laboratory since the mixed soil was behaving like a mud. Besides this, it is unknown how much the unit weight of the stabilised soil (at depth) would change in the field.

Table 5.1; The calculated Factors of Safety and the required Factors of Safety for the levee at the Montfoortse Vaart.

D-GeoStability model	Minimum allowable Factor of Safety at NWC	Required Factor of Safety at HWC
Bishop	0,92	1,02
Uplift Van	0,91	1,07

Table 5.2; The assumptions on the initial strength and the unit weight of the stabilised soils directly after stabilisation.

	Scenario 1	Scenario 2	Scenario 3	Scenario 4
Unit weight	Increase	Increase	No change	No change
Initial shear strength after mixing	None	Remoulded	None	Remoulded

In each scenario the method of execution from section 5.2.1 was modelled for which the Factor of Safety against inward macro-instability was determined for each considered step during execution. The Factor of Safety was determined with both the Bishop and the Uplift Van calculation model for the same reasons as listed in section 3.2.3. The detailed process for determining the practicability of the examined execution method from section 5.2.1 in each scenario is shown in the flow chart in figure 5.2. It should be noted that for the examination of the practicability, no settlement analyses were carried out due to time constraints.

For determining the stability during execution, normal water conditions were assumed, for which an adapted version of the model used for high water conditions was made. In the adapted model in D-GeoStability, the same soil parameters from table 3.3 but a different schematisation of the phreatic groundwater level and a different water level at the outer side of the levee were used (see appendix B).

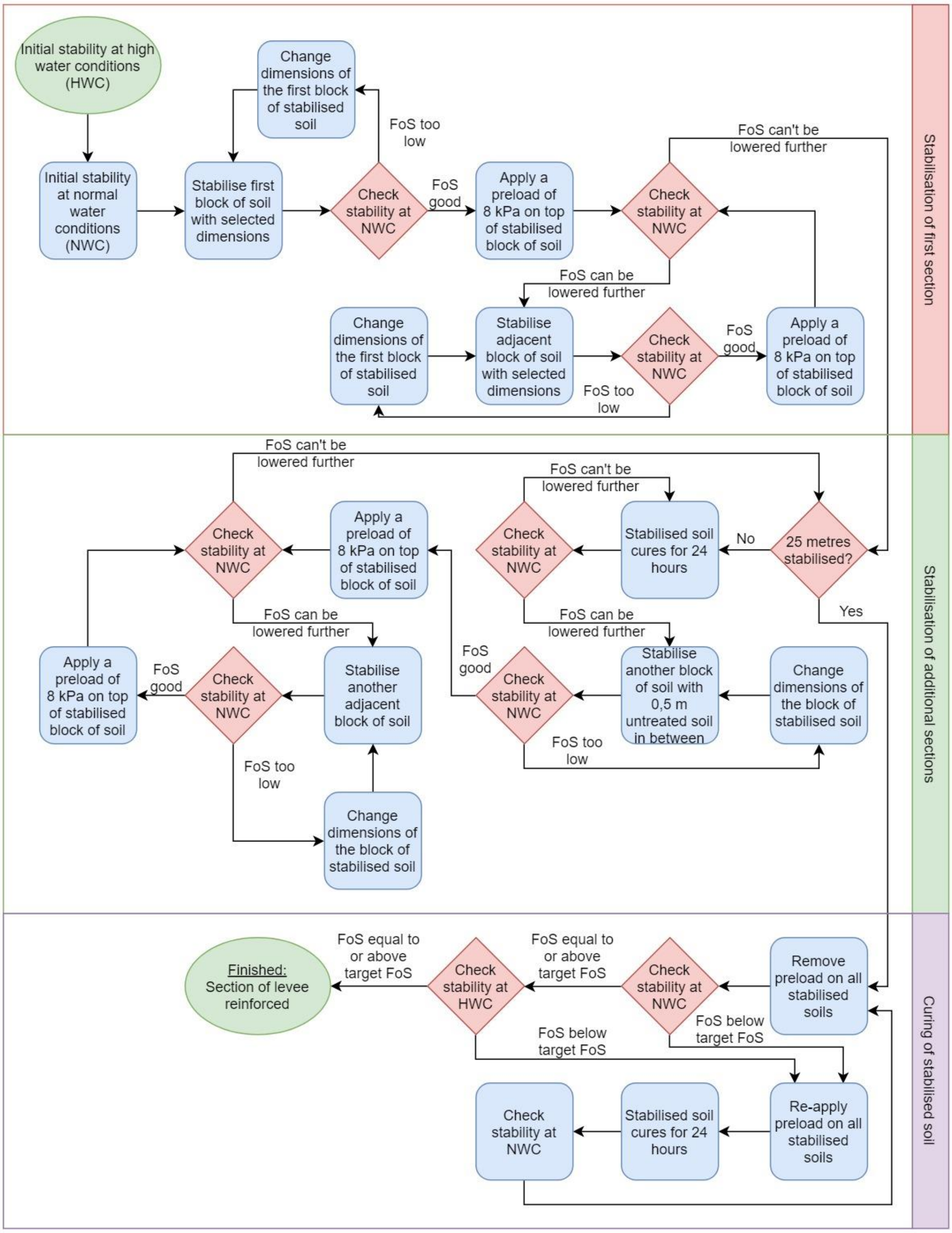

Figure 5.2; Flow chart of the approach to the implementation analyses.

After the feasibility of the examined execution method was examined in each scenario, the obtained implementation and development of the Factor of Safety during execution were presented. For creating the graph detailing the development of the Factor of Safety, the following assumptions were made:

- The speed of stabilisation of both the peat and the organic clay was set equal to 100 m^3/h (Forsman et al., 2015);
- The application and removal of the preload was considered to take 3 minutes per square metre;
- There are 24 hour working days for 7 days a week (i.e. no inactivity during execution).

5.2.3 Parameter determination

For the implementation analyses soil parameters were needed (in time) for the stabilised soils. In this section, the determination of the time-dependent strength, as well as the initial strength and the density of the stabilised soil after mixing are presented.

5.2.3.1 Strength

During the implementation analyses, a strength was assigned to the stabilised soils based on the number of days the soil had cured under load. Since the implementation analyses were carried out using the effective strength parameters as prescribed by the Dutch STOWA guideline, an indication on the effective strength parameters of the stabilised soils in time was required.

However, no measurements of the effective strength parameters were made in time during the laboratory research. So instead, an assumption was made on the development of the effective strength parameters in time. It was assumed that the percentage increase in the undrained compressive strength (and therefore also in the undrained shear strength) as measured during phase 4 of the laboratory research was equal to the percentage increase in the drained shear strength. The consequence of this assumption was that the percentage increase in the effective cohesion and the tangent of the effective angle of internal friction were also equal to the percentage increase in the drained shear strength when using the Mohr-Coulomb equation, which is therefore also equal to the percentage increase in the unconfined compressive strength. This assumption is shown mathematically by equations (5-1) and (5-2).

$$\tau_t = \alpha \cdot \tau_{28-day} = \alpha \cdot c'_{28-day} + \sigma'_n \cdot \alpha \cdot \tan\left(\phi'_{28-day}\right) \quad \textbf{(5-1)}$$

$$\alpha = \frac{UCS_t}{UCS_{28-day}} = \frac{S_{u;t;\sigma_3=0}}{S_{u;28-day;\sigma_3=0}} \quad \textbf{(5-2)}$$

<u>*where:*</u>

τ_t	*- drained shear strength at any curing time (t)*	*[kPa]*
τ_{28-day}	*- drained shear strength after 28 days of curing*	*[kPa]*
α	*- scaling factor to account for percentage increase in strength ($\alpha = 1{,}0$ at 28 days of curing and $\alpha < 1{,}0$ at earlier curing times)*	*[-]*
c'_{28-day}	*- effective cohesion after 28 days of curing*	*[kPa]*
σ'_n	*- effective normal stress*	*[kPa]*
ϕ'_{28-day}	*- effective angle of internal friction after 28 days of curing*	*[°]*
UCS_t	*- unconfined compressive strength at any curing time (t)*	*[kPa]*
UCS_{28-day}	*- unconfined compressive strength after 28 days of curing*	*[kPa]*
$S_{u;t;\sigma_3=0}$	*- undrained shear strength at any curing time (t) as determined in unconfined compression tests*	*[kPa]*
$S_{u;28-day;\sigma_3=0}$	*- undrained shear strength after 28 days of curing as determined in unconfined compression tests*	*[kPa]*

With this assumption, an indicative development of the effective strength parameters was made using the effective strength parameters determined from the results of the triaxial and shearbox tests at respectively 2% axial strain and 5% shear strain. For the use of equations (5-1) and (5-2), it was assumed that the effective strength parameters determined from the laboratory tests correspond to the 28-day strength of both stabilised soils, even though most samples subjected to these tests were brought to failure a few days layer (see section 4.5). This assumption could be made, as the curing curves from figure 4.5 showed that after about 7 days of curing the strength did not increase much anymore.

The determination of the effective strength parameters at 2% axial strain and 5% shear strain were selected in accordance with the applied Dutch STOWA guideline. This selection also fitted well with the brittle behaviour of the stabilised soils observed during the laboratory research. As a result of the brittle behaviour, the stabilised soils have a lot of strength at low strain levels, but also little strength at large strains as the stabilised soils will have failed by then (see stress-strain response in figure 4.7 and figure 4.11, as well as mobilisation of the effective cohesion (c') in figure 4.10 and figure 4.14). This also holds for the stabilised organic clay, even though the mobilisation of the effective strength parameters from figure 4.14 seems to suggest otherwise. From the stress-strain curves of figure 4.12 obtained from the triaxial tests on the stabilised organic clay samples, it can be seen that at 2% axial strain the material has not yet failed as the deviator stress of all three tested samples was still increasing, indicating the strength was still being mobilised.

Since design values were required for use in the stability analyses, the effective strength parameters determined from the laboratory test results were converted to design values using the same coefficients of variation and partial material factors as used for the derivation of the required unconfined compressive strength (see table 3.7 from section 3.6). This was done as there were insufficient tests carried out to allow for a reliable derivation of the 5% characteristic value. The resulting development of the design values of the effective strength parameters in time is presented in figure 5.3. The complete derivation of the development of the design value of the effective strength parameters in time is presented in detail in appendix H.

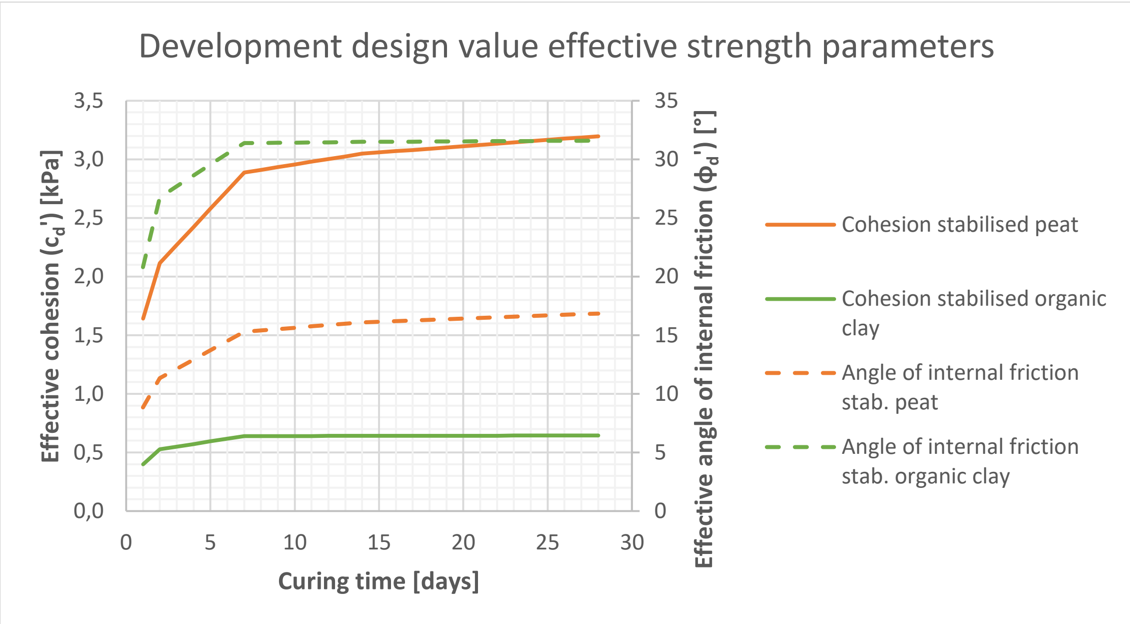

Figure 5.3; The applied development of the effective strength parameters in time for the stabilised peat and the stabilised organic clay during the implementation analyses.

Although the same strength development was modelled using figure 5.3 for all four scenarios, a different initial strength after stabilisation (at 0 days of curing) was assumed for each scenario. The assumed initial strengths are presented in table 5.3. The strengths that are shown for scenario 2 and 4 are based on the strengths of the remoulded peat and organic clay from table 3.3 (see section 3.2.3).

Table 5.3; The assumptions for each scenario on the initial strength directly after stabilisation.

Soil type	Strength parameter	Scenario 1	Scenario 2	Scenario 3	Scenario 4	Unit
Stabilised peat	c'_d	0,0	0,27	0,0	0,27	[kPa]
	ϕ'_d	0,0	5,2	0,0	5,2	[°]
Stabilised organic clay	c'_d	0,0	0,33	0,0	0,33	[kPa]
	ϕ'_d	0,0	13,6	0,0	13,6	[°]

5.2.3.2 Unit weight

During the implementation analyses, a different unit weight was assigned to the stabilised soils based on the scenario. For scenarios 1 and 2, an increased unit weight of the soil was assumed, whereas for scenario 3 and 4 no changes in the unit weight of the soil due to stabilisation were assumed. For scenario 3 and 4, the unit weight of the stabilised peat and organic clay was set equal to respectively the unit weight of the undisturbed peat and the undisturbed organic clay. For scenario 1 and 2, the unit weight of the stabilised soils was determined from the results of the laboratory research.

In accordance with the laboratory soil stabilisation procedure outlined in appendix D, the unit weight of the produced stabilised soils samples was recorded after extrusion and before and after loading while still in the mould. These unit weights were combined for all peat and organic clay samples stabilised during the laboratory research with respectively 50 and 75 kg Portland cement per cubic metre of undisturbed soil. From these unit weights, a suitable design value of the unit weight of the stabilised peat and organic clay prior to loading was derived using a partial material factor of 1,0 based on literature (see section 2.4.2). This derivation is presented in detail in appendix H.

For the implementation analyses of scenario 1 and 2, it was decided to only use the unit weight of the samples prior to loading. This choice was made as no settlement analyses were carried out. As a result, no compression of the stabilised soils was modelled during execution, so it was deemed illogical to apply the unit weight of the samples after loading.

The design value of the unit weight of both the stabilised peat and the stabilised organic clay used in each of the four scenarios is presented in table 5.4. For both stabilised soils, an approach analogous to Dutch standard NEN 9997-1 (i.e. Eurocode 7 + Dutch national appendix) for cohesive soils was applied, in which the characteristic unit weight of the cohesive soil above and below the phreatic surface was considered equal (Normcommissie 351 006 "Geotechniek", 2017). Since a partial material factor of 1,0 was applied, the design value of the unit weight of the stabilised soil above and below the phreatic surface was considered equal.

Table 5.4; The assumptions on the unit weight of the soil after stabilisation for each scenario.

Soil type	Soil parameter	Scenario 1	Scenario 2	Scenario 3	Scenario 4	Unit
Stabilised peat	$\gamma_{stab.;bulk;d}$	10,94	10,94	10,00	10,00	[kN/m^3]
	$\gamma_{stab.;sat;d}$	10,94	10,94	10,00	10,00	[kN/m^3]
Stabilised organic clay	$\gamma_{stab.;bulk;d}$	13,44	13,44	12,70	12,70	[kN/m^3]
	$\gamma_{stab.;sat;d}$	13,44	13,44	12,70	12,70	[kN/m^3]

5.3 Analyses results

In this section, the results of the implementation analyses for all four scenarios are presented.

5.3.1 Scenario 1

In scenario 1 it was assumed that the unit weight of soils increased due to stabilisation and that the soil directly after mixing had no strength. With these assumptions, the implementation of the stabilisation of the soil at the toe of the levee was examined. The obtained implementation is presented in figure 5.4. The development of the Factor of Safety against inward macro-instability during the obtained implementation is presented in figure 5.5. The graph shown in the figure is divided into multiple sections, each representing an action taken during the execution. A description of each of these actions is presented in table 5.5.

When inspecting figure 5.4, it can be seen that about 70% of the 25 metres of levee can be stabilised in one run with the unit weight and initial strength assumptions of scenario 1. This is quite a lot of soil before 24 hours of curing are required to finish the stabilisation. When subsequently inspecting figure 5.5, it can be seen that about 6 days are required to completely reinforce the 25 metres of this levee. After reinforcement, the Factors of Safety determined with both calculation models met their respective required Factors of Safety set for each model.

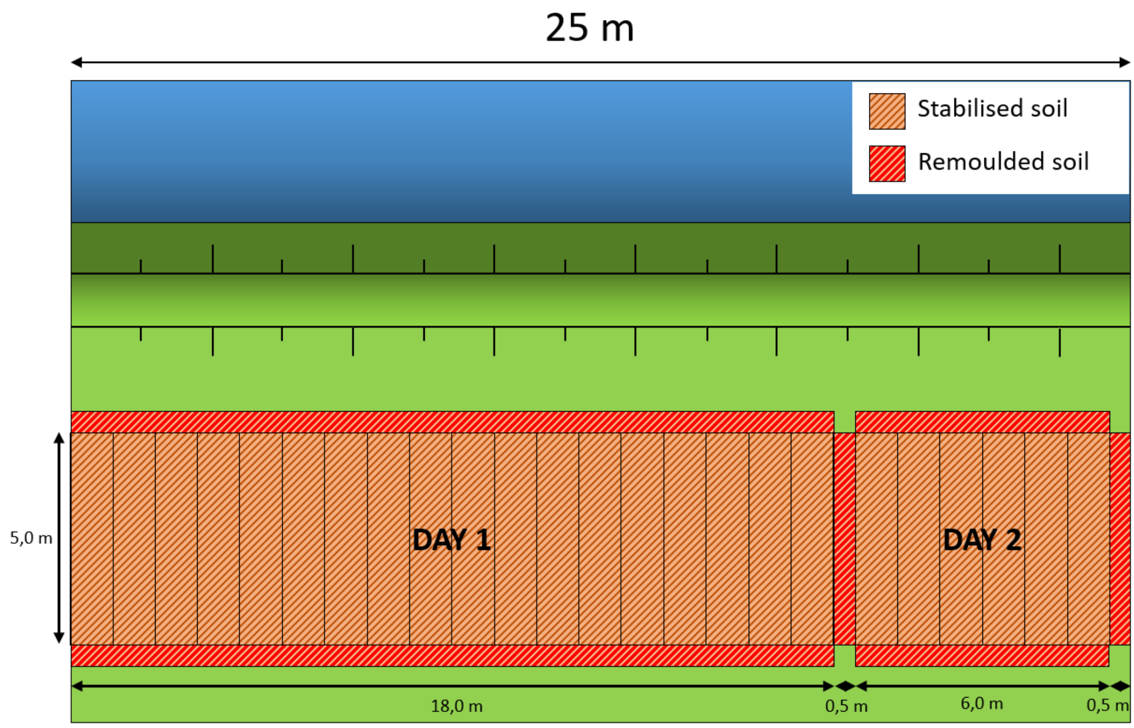

Figure 5.4; Top view of the levee showing the obtained implementation for scenario 1.

However, it can also be seen from figure 5.5 that the Factors of Safety calculated with the Bishop and the Uplift Van calculation model started disagreeing from about halfway through the stabilisation of the first section of the levee (i.e. at about 0,2 days). At this point, the Factor of Safety as determined with the Uplift Van calculation model suddenly increased with about 0,10 after continued stabilisation. This was not expected, as a reduction in the Factor of Safety like what was calculated with the Bishop calculation model was expected.

This measurement was likely the result of the difficulty with which the Uplift Van model can be controlled by the user to find the critical slip surface. The Uplift Van calculation model requires input from the user on possible locations for the centre points of the slip circles, after which the model applies an algorithm to search for the critical slip surface. Since the user has little control over the algorithm, it likely resulted in the anomaly in the obtained Factors of Safety for the Uplift Van model.

Besides this, also differences in the Factors of Safety as determined by the Bishop and Uplift Van calculation model were observed at later time steps during the execution. These differences arose as a result of the strengthening. When the stabilised soil started to cure and gain strength, while still loaded by the preload, the critical slip surface started to change shape and position. The critical slip surface started to pass underneath the stabilised soil block. The Uplift Van calculation model modelled this as a horizontal slip surface at the boundary of the stabilised soil and the sand layer, whereas the Bishop calculation model modelled this as a circular slip surface passing through the sand layer. As a result of this, the observed differences in the Factor of Safety were obtained. Although these differences in the Factor of Safety were obtained, both calculation models showed the same trends in the position of the critical slip surface. As a result of this, similar developments in the Factor of Safety determined with both calculation models were obtained as seen in figure 5.5.

After reinforcement was completed, the critical slip surface as shown in figure 5.6 was obtained at normal water conditions for both the Bishop and Uplift Van calculation model. From the figure can be seen that the critical slip surface retreated toward the slope of the levee and became smaller as a result of the stabilisation.

All further details on the results of the implementation analyses of scenario 1 are presented in appendix H.

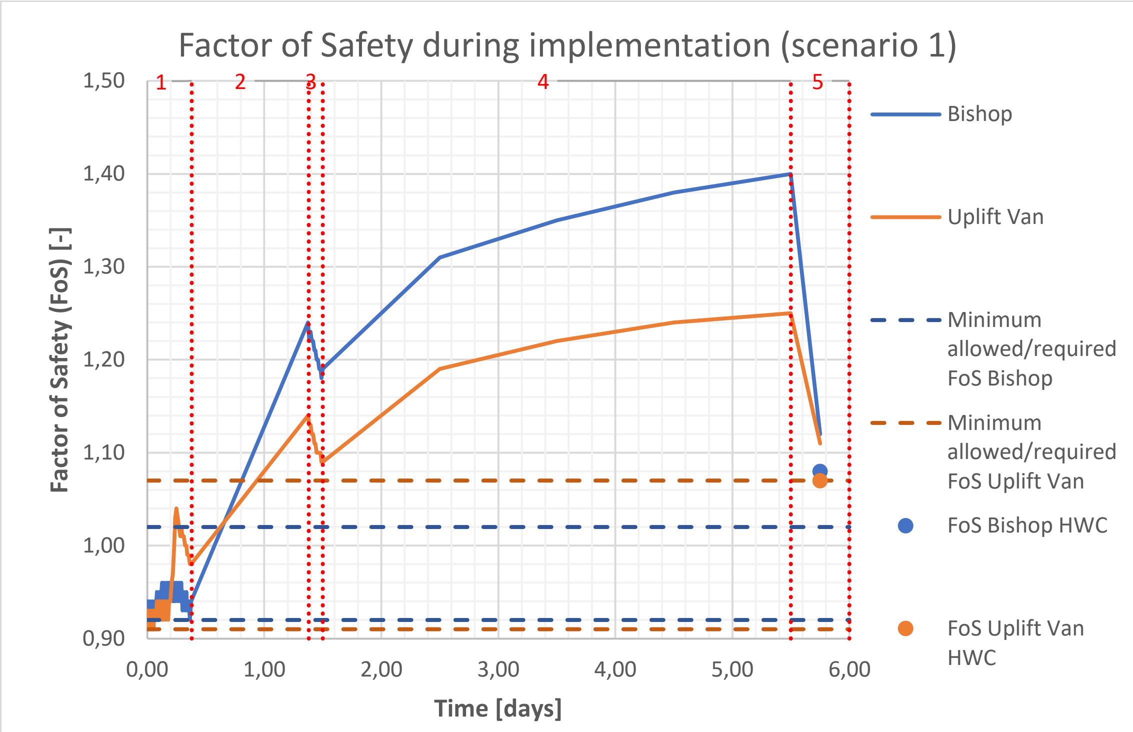

Figure 5.5; The development of the Factor of Safety during the execution of mass stabilisation in scenario 1. The red lines represent different actions taken during execution, the description of which is presented in table 5.5.

Table 5.5; Actions taken during the implementation of scenario 1.

Line number figure 5.5	Action
1	Stabilisation of first 18 metres of soil
2	24 hours of curing of all blocks of stabilised soil
3	Stabilisation of last 6 metres of soil
4	96 hours of curing (i.e. 4 days) of all blocks of stabilised soil
5	Removing all preload

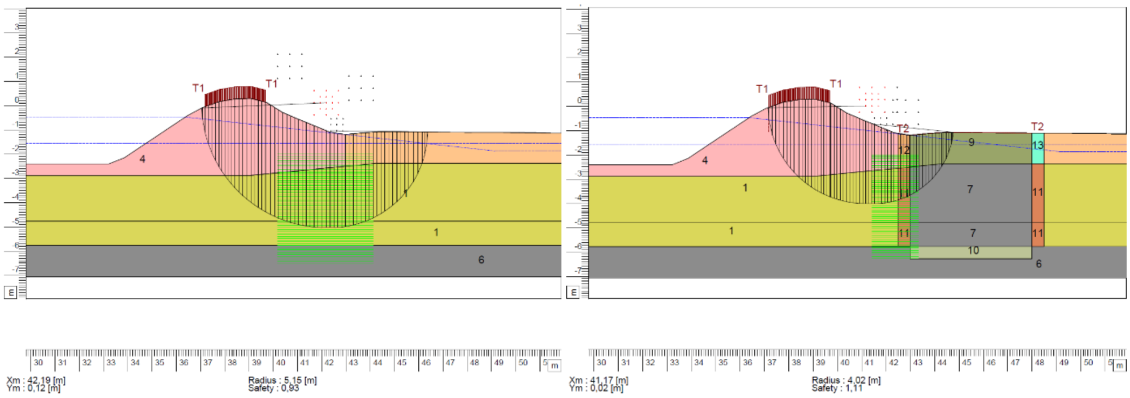

Figure 5.6; The critical slip surface (Uplift Van) at normal water conditions before and after reinforcement.

5.3.2 Scenario 2

In scenario 2 it was assumed that the unit weight of soils increased due to stabilisation and that the strength of the soil directly after mixing was a reduction of the initial strength. This is the most likely scenario to occur in the field. With these assumptions, the implementation of the stabilisation of the soil at the toe of the levee was examined. The obtained implementation is presented in figure 5.7. The development of the Factor of Safety during the obtained implementation is presented in figure 5.8. The actions taken during execution for each of the in figure 5.8 highlighted sections are presented in table 5.6.

When inspecting figure 5.7, it can be seen that entire 25 metres of levee can be stabilised in one run with the unit weight and initial strength assumptions of scenario 2. It should be noted from figure 5.7 that only 24,5 metres are stabilised instead of 25 metres. This was done with the assumption in mind that only one section of 25 metres of soil would be stabilised in a single day. Therefore the stabilised soil will already have hardened once the next 25 metres of levee will be reinforced with mass stabilisation. Since always 0,5 metre of soil would be left between soils stabilised at different days, it was decided to leave the final 0,5 metre of soil untreated.

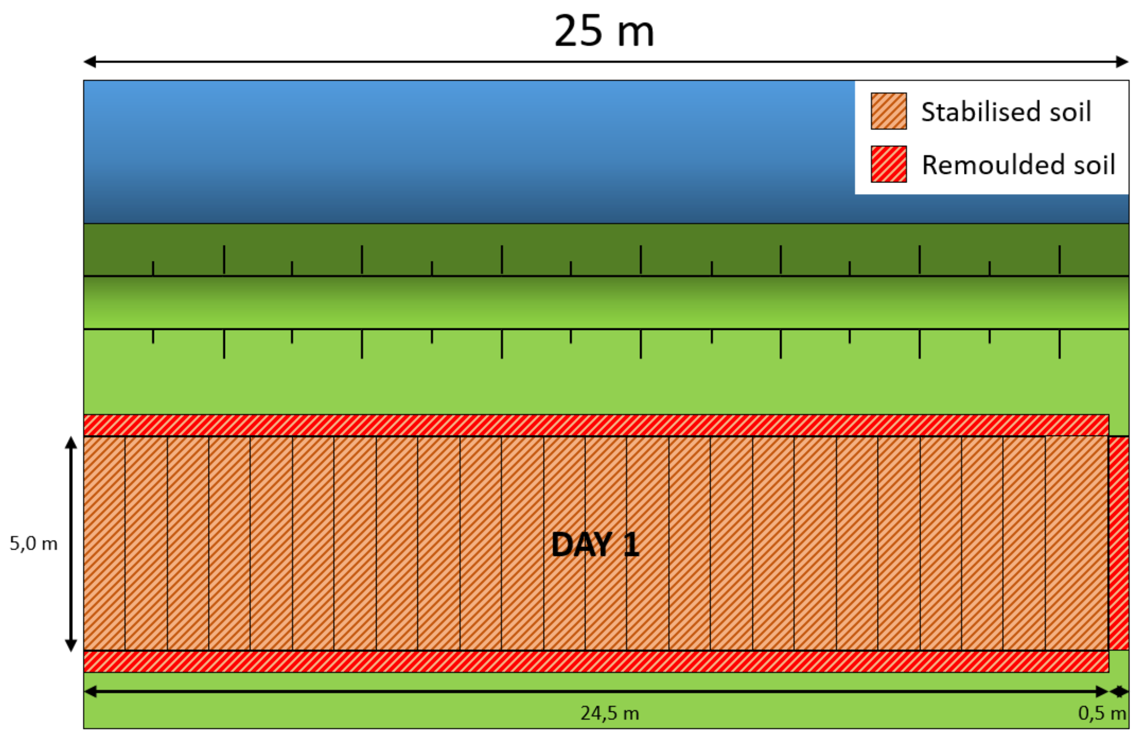

Figure 5.7; Top view of the levee showing the obtained implementation for scenario 2.

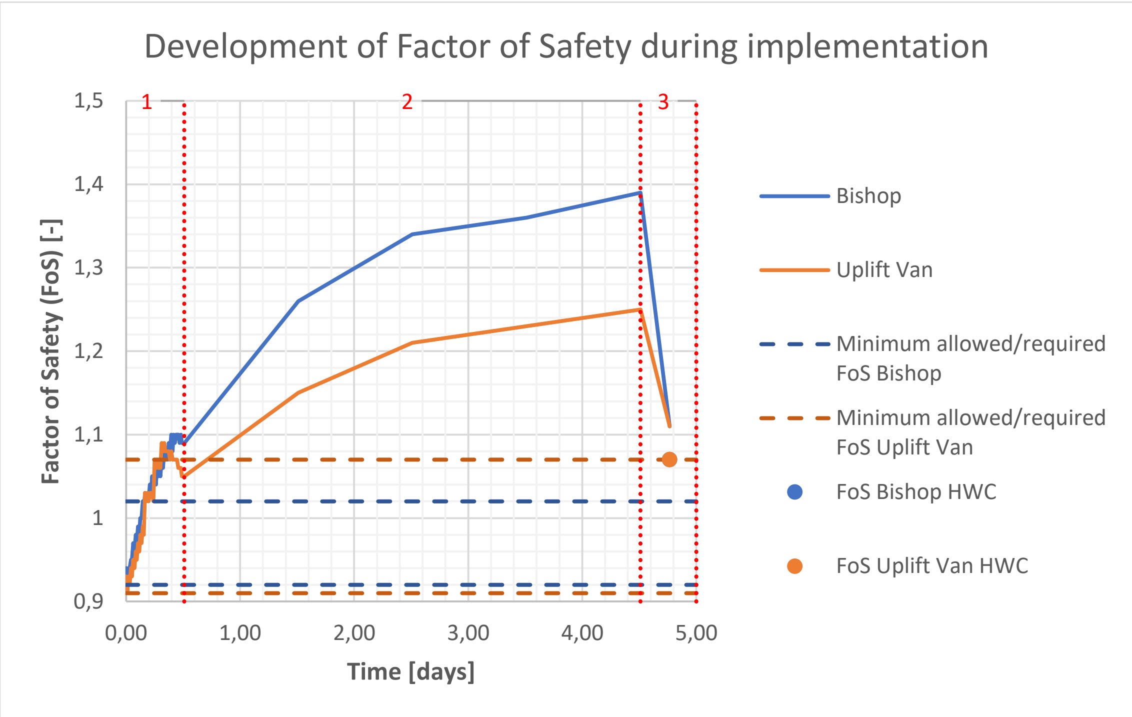

Figure 5.8; The development of the Factor of Safety during the execution of mass stabilisation in scenario 2. The red lines represent different actions taken during execution, the description of which is presented in table 5.6.

When subsequently inspecting figure 5.8, it can be seen that about 5 days are required to completely reinforce the 25 metres of this levee. Furthermore, it can be seen that the Factor of Safety against inward macro-instability increased during stabilisation. Apparently, after the preload of 8,0 kPa is applied after each block is stabilised, the reduction in stability due to stabilisation is compensated for by the preload under the assumptions of an increased unit weight and a small initial strength of the stabilised soil. As a result of this, the entire section of soil could be stabilised in one run.

Like with the results of scenario 1, the Factors of Safety as obtained with the Bishop and Uplift Van calculation models show similar trends, but do not match after the stabilisation of the entire 25 metres. As explained in section 5.3.1, this was caused by the differences in shape of the critical slip surfaces.

After reinforcement was completed, the critical slip surface as shown in figure 5.9 was obtained at normal water conditions for both the Bishop and Uplift Van calculation model. From the figure can be seen that the critical slip surface retreated toward the slope of the levee and became smaller as a result of the stabilisation. This is a similar result as obtained after reinforcement in scenario 1.

All other details on the results of the implementation analyses of scenario 2 are presented in appendix H.

Table 5.6; Actions taken during the implementation of scenario 2.

Line number figure 5.8	Action
1	Stabilisation of 24,5 metres of soil
2	96 hours of curing (i.e. 4 days) of all blocks of stabilised soil
3	Removing all preload

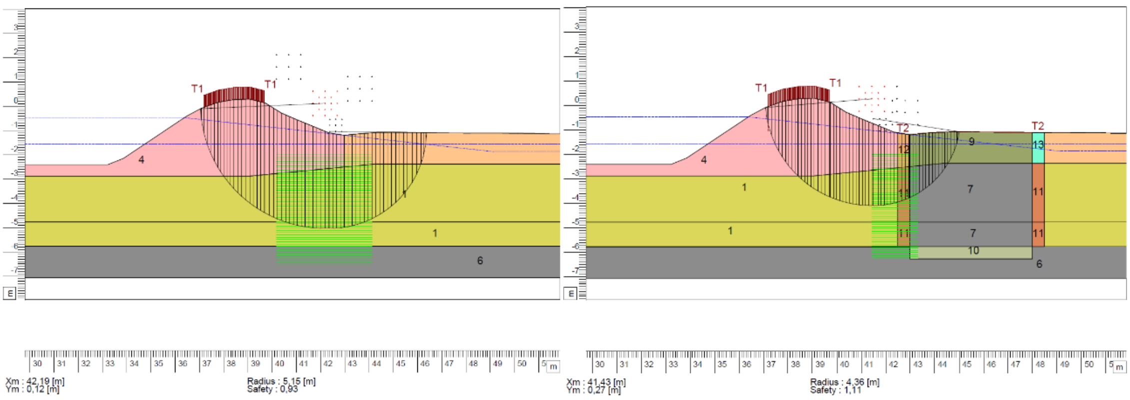

Figure 5.9; The critical slip surface (Uplift Van) at normal water conditions before and after reinforcement.

5.3.3 Scenario 3

In scenario 3 it was assumed that the unit weight of soils did not change due to stabilisation and that the soil had no strength directly after mixing. This is a very conservative scenario. With these assumptions on the unit weight and the initial strength, the execution of mass stabilisation at the toe of the levee was examined. The obtained implementation is presented in figure 5.10. The development of the Factor of Safety during the obtained implementation is presented in figure 5.11. The actions taken during execution for each of the in figure 5.11 highlighted sections are presented in table 5.7.

When examining figure 5.10, it can be seen that the first section of soil was not stabilised in the same manner as set in section 5.2.1 and examined in the other scenarios. This was the direct result of the sensitivity of the Factor of Safety to the stabilisation under the assumptions of no increased unit weight and no initial strength of the stabilised soil. If one remained stuck to the execution method of 1,0 metre wide blocks, it would not have been possible to carry out this stabilisation as a result of the small allowable reduction in the Factor of Safety. Clearly, the stabilisation of the first section of soil is therefore the most critical part of the execution. However, since it was still possible to stabilise the first section, but in a different manner, the stabilisation as shown in figure 5.10 was applied.

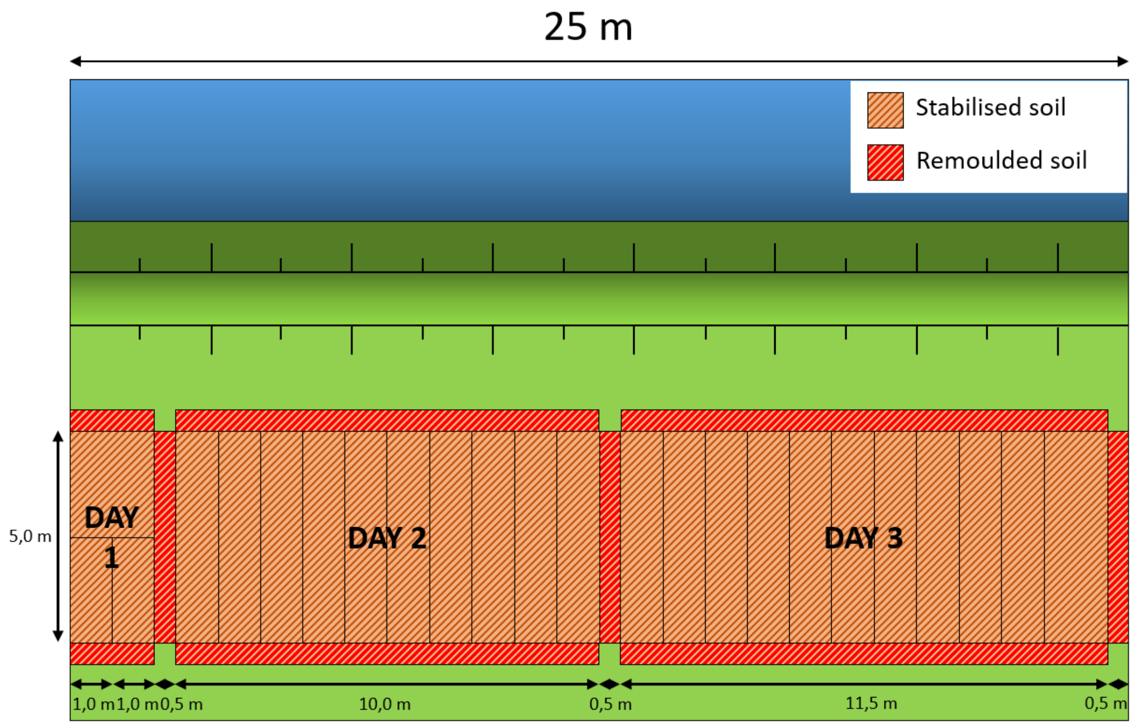

Figure 5.10; Top view of the levee showing the obtained implementation for scenario 3.

Upon further inspection of figure 5.10, it can be seen that the entire 25 metres of levee had to be stabilised in three sections under the unit weight and initial strength assumptions of scenario 3. This too was the result of the large reduction in the Factor of Safety upon stabilisation of a block of soil. Since the assumed reduction in strength in this scenario was much larger than in scenarios 1 and 2 and since there was no increase in the unit weight that compensated for some of the strength loss, these large reductions in the Factor of Safety were obtained upon stabilisation of a block of soil.

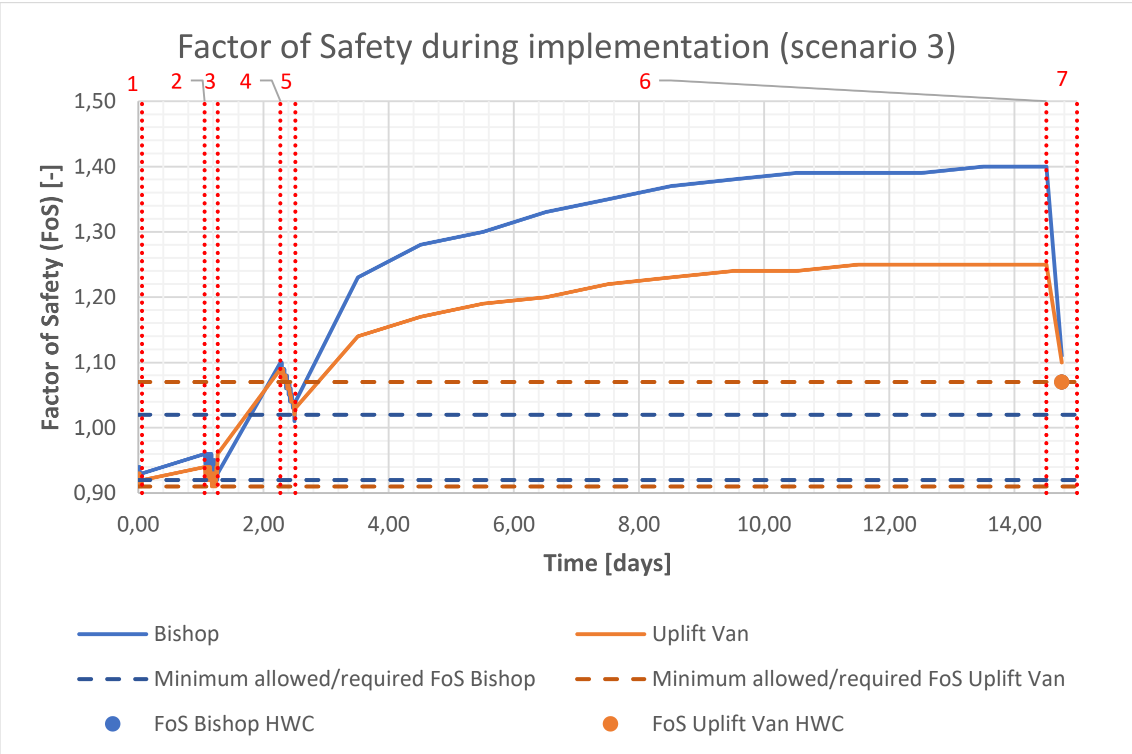

Figure 5.11; The development of the Factor of Safety during the execution of mass stabilisation in scenario 3. The red lines represent different actions taken during execution, the description of which is presented in table 5.7.

When subsequently inspecting figure 5.11, it can be seen that about 15 days are required to reinforce the 25 metres of this levee. This is quite a lot of time, more than double the time required to stabilise 25 metres of this levee in scenarios 1 and 2. This was mainly caused by the requirement of a larger strength of the stabilised soil as no increased unit weight was assumed that would have caused additional increases in the Factor of Safety. This required strength was only achieved after a much longer curing time.

Like with the results of scenarios 1 and 2, the Factors of Safety as obtained with the Bishop and Uplift Van calculation models do not match after the stabilisation of the entire 25 metres. This is the result of differences in the shape and position of the critical slip surface as determined by both calculation models.

After reinforcement was completed, the critical slip surface as shown in figure 5.12 was obtained at normal water conditions for both the Bishop and Uplift Van calculation model. From the figure can be seen that the critical slip surface retreated toward the slope of the levee and became smaller as a result of the stabilisation. This is a similar result as obtained after reinforcement in scenarios 1 and 2.

All other details on the results of the implementation analyses of scenario 3 are presented in appendix H.

Table 5.7; Actions taken during the implementation of scenario 3.

Line number figure 5.11	Action
1	Stabilisation of 2,0 metres of soil
2	24 hours of curing of all blocks of stabilised soil
3	Stabilisation of 10,0 metres of soil
4	24 hours of curing of all blocks of stabilised soil
5	Stabilisation of last 11,5 metres of soil
6	288 hours of curing (i.e. 12 days) of all blocks of stabilised soil
7	Removing all preload

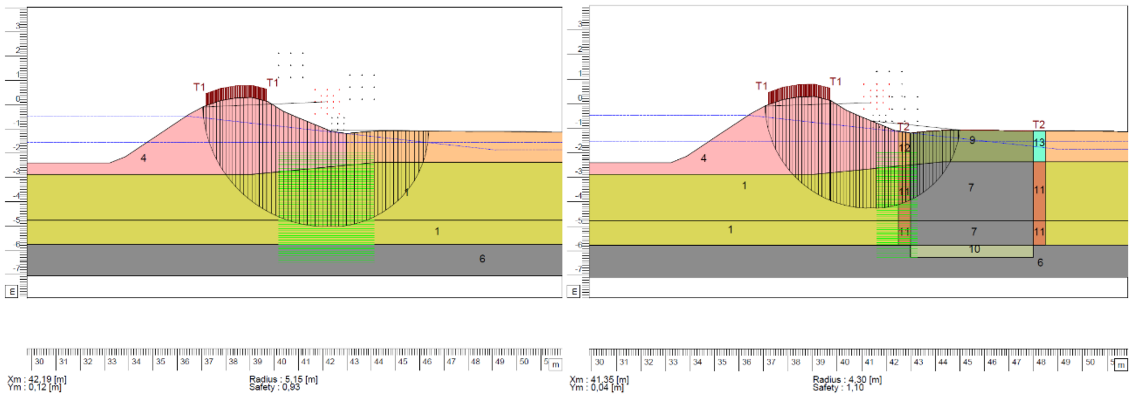

Figure 5.12; The critical slip surface (Uplift Van) at normal water conditions before and after reinforcement.

5.3.4 Scenario 4

In scenario 4 it was assumed that the unit weight of soils did not change due to stabilisation and that the strength of the soil directly after mixing was a reduced strength from the initial strength. With these assumptions, the execution of the soil stabilisation at the toe of the levee was examined. The obtained implementation is presented in figure 5.13. The development of the Factor of Safety against inward macro-instability during the obtained implementation is presented in figure 5.14. The actions taken during execution for each of the in figure 5.14 highlighted sections are presented in table 5.8.

When inspecting figure 5.13, it can be seen that entire 25 metres of levee can be stabilised in one run with the unit weight and initial strength assumptions of scenario 4. This result is similar to the result of scenario 2. Like in scenario 2, the final 0,5 of soil were not stabilised under the assumption that the next 25 metres of levee would be stabilised on another day, wanting to prevent remoulding of cured soils.

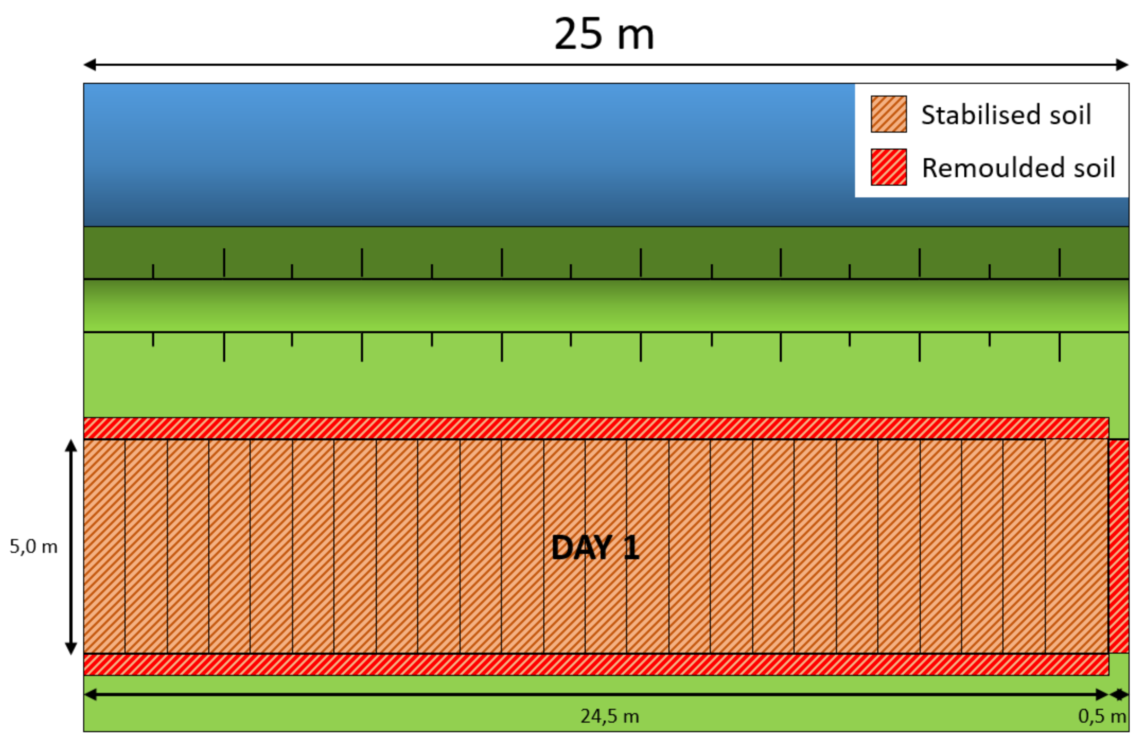

Figure 5.13; Top view of the levee showing the obtained implementation for scenario 4.

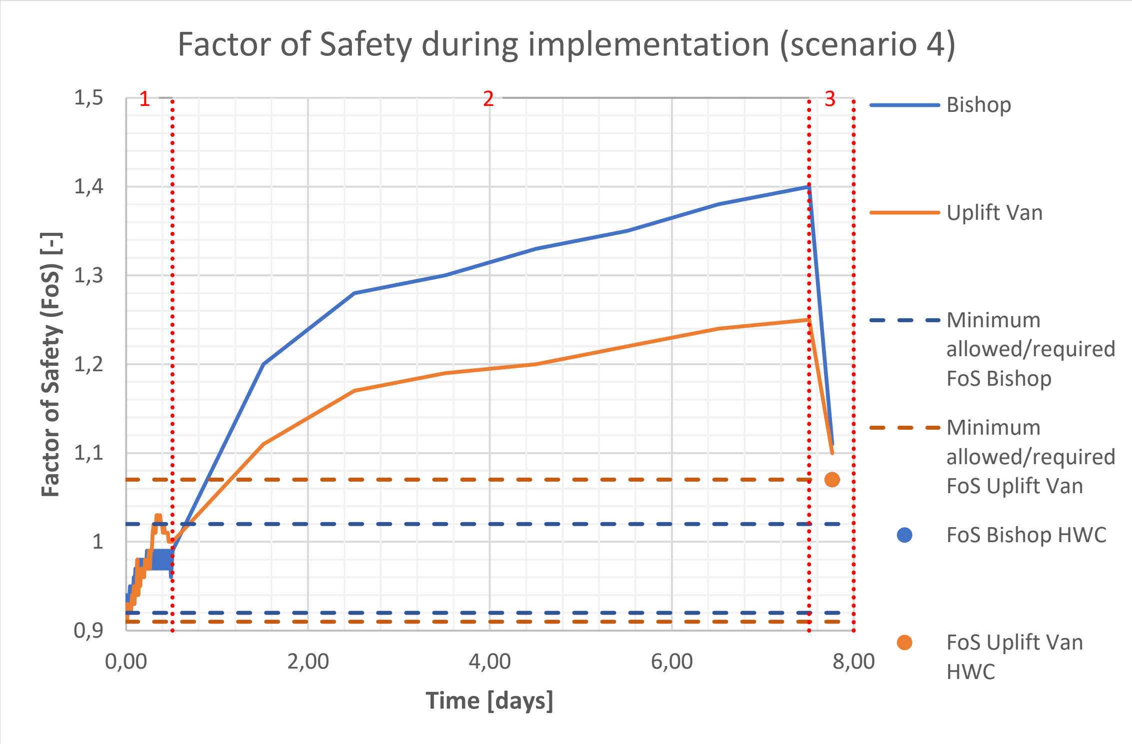

Figure 5.14; The development of the Factor of Safety during the execution of mass stabilisation in scenario 4. The red lines represent different actions taken during the implementation, the description of which is presented in table 5.8.

Table 5.8; Actions taken during the implementation of scenario 4.

Line number figure 5.14	Action
1	Stabilisation of 24,5 metres of soil
2	168 hours of curing (i.e. 7 days) of all blocks of stabilised soil
3	Removing all preload

When subsequently inspecting figure 5.14, it can be seen that about 8 days are required to completely reinforce the 25 metres of this levee. This is slightly more time than required in scenario 2. Additionally, it can be seen that halfway through the stabilisation (i.e. at about 0,2 days) the Factors of Safety as determined with the Bishop and Uplift Van calculation model started to disagree like in scenario 1. As mentioned in section 5.3.1, this was caused by the difficulty in controlling the Uplift Van calculation model.

After reinforcement was completed, the critical slip surface as shown in figure 5.15 was obtained at normal water conditions for both the Bishop and Uplift Van calculation model. From the figure can be seen that the critical slip surface retreated toward the slope of the levee and became smaller as a result of the stabilisation. This is a similar result as obtained after reinforcement in scenarios 1, 2 and 3.

All other details on the results of the implementation analyses of scenario 4 are presented in appendix H.

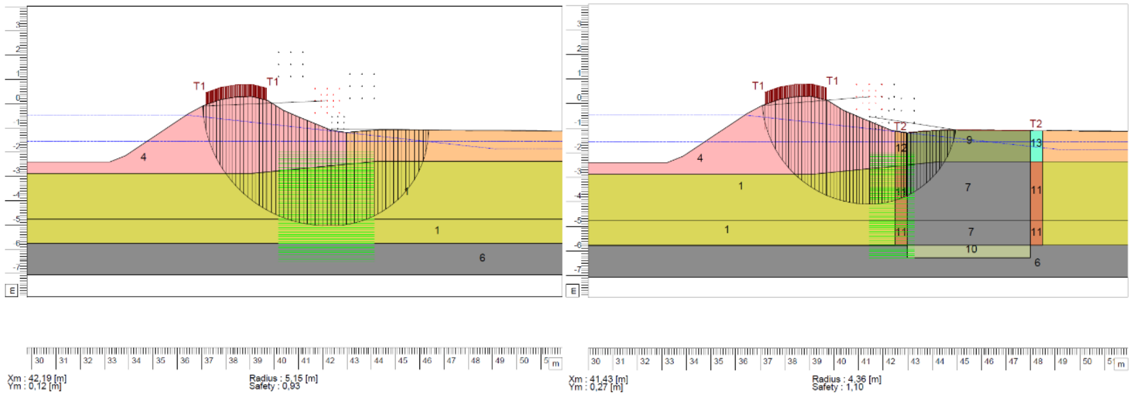

Figure 5.15; The critical slip surface (Uplift Van) at normal water conditions before and after reinforcement.

5.3.5 Comparison of results

In each scenario, it was possible to reinforce the levee by achieving the required Factor of Safety without lowering the Factor of Safety below the allowable minimum during execution. However, the obtained implementation and the duration until reinforcement varied between scenarios. In order to compare the results, an overview of the results of the implementation analyses is presented in table 5.9.

On the basis of this overview, it seems that between 1 to 3 days of stabilisation and 4 to 12 days of curing are required to completely reinforce the fixed section of 25 metres of the examined levee. However, this does not mean that a contractor will be working continuously on the same section of 25 metres for 5 to 15 consecutive days before the next section can be reinforced. After all, work only needs to be carried out for at the most 4 days (i.e. stabilisation in at the most 3 days and preload removal in 1 day). In addition, the contractor will likely try to save more time by working on multiple sections of levee at the same time. Because of this, a contractor can always continue while another section of stabilised soil is curing. This could for example be useful in scenario 1 and 3, where it takes multiple days to stabilise a single section of 25 metres of soil at the levee. Because of such possible time optimisations, the reinforcement times listed in table 5.9 cannot just be used to estimate the time needed to reinforce larger sections of the levee.

Table 5.9; Comparison of the results of the implementation analyses of the four examined scenarios.

Parameter	Scenario 1	Scenario 2	Scenario 3	Scenario 4
Unit weight	Increase	Increase	No change	No change
Initial strength after mixing	None	Reduced (remoulded)	None	Reduced (remoulded)
Time until completion reinforcement (incl. sufficient curing)	6 days	5 days	15 days	8 days
Number of sections of soil stabilised	2	1	3	1

When examining table 5.9 further, it was noticed that the assumption on the increased unit weight had a larger impact on the speed of the reinforcement than the assumption on a small strength of the soil directly after mixing had. This was evident when comparing the time required until complete reinforcement for scenario 1 and 4. The reason this result was obtained is that in scenario 1 less strength of the stabilised soil was required in the final situation than in scenario 4, because the assumed increase in the unit weight in scenario 1 caused an additional increase in the Factor of Safety. This resulted in reaching the required Factor of Safety much faster in scenario 1 than in scenario 4.

Furthermore, it was noticed that a large difference in the required time for complete reinforcement of the levee was recorded between scenario 3 and 4 even though the same strength development was assumed. This was the result of the assumptions on the initial strength and the unit weight of the stabilised soil in scenario 3, because 1,5 metres of soil was left untreated in scenario 3. On the other hand, with the assumptions in scenario 4, only 0,5 metre of soil was left untreated. The consequence of this was that in scenario 3 the levee had to be reinforced by increasing the strength of 23,5 metres of soil, whereas in scenario 4 the levee had to be reinforced by increasing the strength of 24,5 metres of soil. As a result, a slightly larger final strength of the stabilised soil was required in scenario 3 than in scenario 4, which was only reached after curing the soil for a longer amount of time. Combining this with the time it took to stabilise the three sections in scenario 3 (i.e. 3 days) as opposed to the single section in scenario 4 (i.e. 1 day) resulted in the completion times as presented in table 5.9.

Lastly, it was noticed that given the assumed development of the effective strength parameters of the stabilised soils in time (see figure 5.3), any further increase in the required Factor of Safety would probably not have allowed for complete reinforcement of the levee in scenario 3. In scenario 3 it required 15 days to complete the reinforcement because the required design strength of the stabilised soil was reached after 12 days of curing. At this point in time, the strength of the soils was hardly increasing anymore. If the required design strength of the stabilised soils had been larger as a result of stricter requirements on the target Factor of Safety at high water conditions, it would no longer have been possible to reinforce the levee with the assumed strength development from figure 5.3. In such an event, a different binder recipe, possibly in the form of an increased binder dosage, would have to be applied to obtain a larger strength of the stabilised soil with which the target Factor of Safety can be achieved.

5.4 Points of attention

While making the implementation analyses and during the interpretation of the results, a number of matters were noticed that require further attention.

5.4.1 Limitations of applied analyses

The applied two-dimensional stability analyses for examining the practicability of mass stabilisation for reinforcing levees have a few limitations:

- It is unknown what delays in the application of the preload are possible during execution;
- The time required for reinforcement of larger sections of levee cannot be estimated;
- It is unknown to what extent settlement of the stabilised soils influence the practicability;
- It cannot be said whether the levee will fail over smaller sections of levee during execution.

First of all, the applied analyses do not give an indication on the allowable delay in the application of the preload during execution as a result of the precondition set on the preload in section 5.2.1. Since the stabilisation of soil caused a reduction in the Factor of Safety, of which the absolute reduction was dependent on the examined scenario, only a limited quantity of soil could be stabilised before the Factor of Safety would drop below the lower limit. By subsequently applying a preload on top of the stabilised soil, the Factor of Safety increased which in turn allowed for larger quantities of soil to be stabilised. Since in practice the preload may be applied on a block of stabilised soil while the mixing machine is already stabilising the next block in order to save time, the delay with which the preload may be applied relative to the stabilisation of the soil is therefore quite important. However, due to time restrictions, the allowable delay of the preloading was not examined in the implementation analyses.

Secondly, the applied analyses do not give an indication on the time required to reinforce a larger section of levee (e.g. 1 km). In the analyses, only an indication was obtained on the duration to completely reinforce a fixed section of 25 metres of levee. This result cannot be extrapolated to larger sections of levee. When considering a larger section of levee, the execution may change which will change the time required to reinforce a larger section of levee. In addition, a contractor will try to save time by optimising the execution as much as possible, possibly by working at multiple spots along the levee. As a result, the results of the analyses cannot be used to obtain an indication on the time required to reinforce a larger section of levee using mass stabilisation.

Thirdly, in the applied implementation analyses, settlement of the stabilised soils due to the application of the preload was not included due to time constraints. However, settlement of the stabilised soil will influence the practicability of the stabilisation: the stabilised soil will compress, the density of the stabilised soil will increase and thus the stability of the levee will be influenced. Since settlement analyses were not carried out in this research, it is unknown to what extent the settlement influences the practicability of mass stabilisation at the examined levee.

Lastly, the implementation analyses were carried out assuming a 25 metre wide critical slip surface. A consequence of this assumption is that it was not known from these analyses whether the levee could or would fail over a smaller section of levee if the stability at that smaller section were to be insufficient during execution. After all, in each scenario the strength of the soil is reduced due to stabilisation, creating a temporarily weaker spot at the levee. To determine whether the levee fails during execution or remains standing due to three-dimensional effects such as arching, more advanced analyses will have to be carried out. Such effects are also relevant for examining the execution at larger sections of levee.

5.4.2 Optimisation of implementation analyses

In this study the practicability of the examined method of execution was examined by assessing the stability during execution. The stability was determined using two-dimensional stability analyses with weighted averages of the soil properties and the preload over a fixed section of 25 metres. This method is not very flexible and requires many recalculations if the starting points change.

A much more flexible method is to determine the weighted average of the Factor of Safety over any section of levee. For this method the Factor of Safety would have to be determined for each different cross-section of the levee, after which a weighted average of the Factor of Safety would be calculated based on the width of the levee over which the various calculated Factors of Safety apply. Using this method, the Factor of Safety would then have to be determined for every cross-section of the levee through:

- A block of stabilised soil cured for a different amount of time, both with and without a preload on top;
- Remoulded soil bordering the block(s) of stabilised soil;
- The undisturbed soil.

The advantage of applying the method with a weighted average of the Factor of Safety is that it is more flexible and can be used more easily to quickly examine:

- The practicability of different execution methods, such as with larger, smaller or discrete blocks of stabilised soil, possibly over different lengths of levee;
- The impact of a delay in the application of the preload on the execution;
- The influence of differences in the duration of preloading of soils stabilised at different times.

Contrary to the applied method with weighted averages of the soil properties and the preload, the method with a weighted average of the Factor of Safety is able to determine weak spots during execution. Since the Factor of Safety is determined for every deviating cross-section of the levee, it can be determined whether and if so, where the levee is locally weak during execution. This in turn could be used to determine over which section of levee the Factor of Safety has to be averaged if the practicability is examined over larger sections of levee. Because of all these advantages, and because this method is applied in practice, it is recommended to apply the method with weighted averages of the Factor of Safety for future stability analyses during execution of mass stabilisation at levees.

5.4.3 Soil behaviour

The implementations shown in section 5.3 were obtained using the measured developments of the strength of laboratory-produced stabilised soils. However, these implementations may differ from the implementations that would be obtained if the strength development of field-stabilised soils were used instead. The strength development in the field may be different than measured in the laboratory as larger volumes of soil are stabilised in the field and subsequently cured under uncontrollable conditions (e.g. fluctuating temperature, precipitation, etc.). These differences are discussed in more detail in section 7.2. As a result of these differences, the implementations should preferably be determined based on strength measurements of soils stabilised in the field, but also using the strength of undisturbed soils measured in a laboratory.

Besides this, the strength of the stabilised soil samples were evaluated using triaxial compression and shearbox tests. However, in the final situation of all four examined scenarios, the passive zone of the critical slip surface was found to be passing through the stabilised soils (see figure 5.6, figure 5.9, figure 5.12 and figure 5.15). This implies that if the stabilised soil would fail, it would shear in tension. Since no triaxial extension tests were carried out, the strength of the stabilised soils may be different than measured. This may result in different implementations. Even so, it is uncommon in Dutch geotechnical practice to carry out triaxial extension tests on soils, as the stability of levees is usually assessed using only the results from triaxial compression tests and direct simple shear tests.

5.5 Conclusion

Exploratory two-dimensional stability analyses were carried out to determine whether the stabilisation of the soil at the toe of the levee at the Montfoortse Vaart was practicable. At this levee the stability during a general execution method for regional flood defences was examined, for which the following sub-question was drawn up:

'Is the application of mass stabilisation at the levee of the selected case practicable?'

The results from the stability analyses carried out show that mass stabilisation is practicable at the levee under consideration with different assumptions in the density and initial strength of the stabilised soils. Although the analyses provide positive indications on the practicability, not all aspects related to the practicability were examined due to limitations of the applied analyses. As a result, further analyses into settlement and three-dimensional effects during execution are required before a definitive conclusion on the practicability can be drawn.

6 Conclusion

Mass stabilisation is an innovative soil improvement technique with which the stability of structures can be improved (Building Research Establishment (BRE), 2002). The technique has a wide range of applications, but the technique has never been applied to reinforce existing levees. Since mass stabilisation has a number of advantages compared to traditional methods of reinforcement, this research into the technical feasibility of applying mass stabilisation for reinforcing Dutch regional flood defences has been carried out. For this research, the following main research question has been drawn up:

'Is the application of mass stabilisation for improving the inward macro-stability of regional flood defences by stabilising strips of soil technically feasible?'

In order to determine whether the application of mass stabilisation for reinforcing levees is technically feasible, the following three aspects have been examined by means of two-dimensional stability analyses and laboratory research:

- The ability of mass stabilisation to solve a stability deficit at a levee;
- The achievability of the required effective strength parameters of the stabilised soil in compliance with Dutch safety standards;
- The practicability of mass stabilisation at levees.

First, a series of two-dimensional stability analyses has been carried out in which the influence of a strip of stabilised soil on the Factor of Safety has been examined for one cross-section of two real Dutch levees ('boezemkaden'). In the analyses, the strip of soil has been modelled with an improved strength at three different positions at the levee: at the toe, at the slope and at the crest. The results from the analyses show that increases in the Factor of Safety between 7% and 47% have been achieved, showing that mass stabilisation can be applied to solve a stability deficit at a levee. However, the absolute increase in the Factor of Safety due to stabilisation depends on the position at the levee at which the soil is stabilised. The most favourable position for stabilisation, leading to the largest increase in the Factor of Safety, is dependent on the soil profile and therefore case-specific.

Secondly, laboratory research has been carried to examine the changes in the strength properties of a peat and an organic clay sampled near one of the examined levees. The changes in the strength properties have been examined for the stabilisation of the peat and the organic clay with respectively 50 and 75 kg Portland cement per cubic metre of undisturbed soil (i.e. corresponding to respectively 5% m/m and 4% m/m). For both stabilised soils the unconfined compressive strength has been measured to have increased up to about 7 days of curing, after which the strength has changed little. Furthermore, after curing of both stabilised soils, the unconfined compressive strength requirement has been met, whereas the requirement for the effective strength parameters determined at 2% or 5% strain in compliance with Dutch safety standards has not. Yet, trial stability calculations have shown that the measured combination of the effective strength parameters of both stabilised soils is still sufficiently high to achieve the required increase in the Factor of Safety at the examined levee. Although by stabilising the soils sufficient strength has developed to reinforce the levee, the soils have also become more brittle after stabilisation.

Lastly, another series of two-dimensional stability analyses have been carried out in which execution of mass stabilisation for the same considered levee has been examined solely on the basis of strength. In the analyses different assumptions in the density and initial strength of the stabilised soils have been used. The results of the analyses indicate that mass stabilisation may be practicable at the levee. However, the practicability could not be fully examined due to limitations in the applied methodology.

In conclusion, the most important aspects of mass stabilisation have been examined and based on the findings mass stabilisation is considered a technically feasible technique for reinforcing regional flood defences. However, a number of other aspects related to the practicability still need to be considered before a definitive conclusion can be drawn on the technical feasibility.

7 Discussion

7.1 Variation in strength

7.1.1 Selection of coefficients of variation

In this research, the conversion from characteristic values to mean values and vice versa was done using predetermined values of the coefficients of variation of the effective cohesion and the tangent of the effective angle of internal friction. These coefficients of variation were assumed due to a lack of information on these particular parameters from literature. The coefficients of variation applied in this study are presented in table 7.1.

The coefficient of variation of the effective cohesion for both examined stabilised soils was set equal to 0,4. This value was selected as twice the coefficient of variation of the effective cohesion of table 2.b of Dutch standard NEN 9997-1, which shows characteristic values of various soil parameters of Dutch soils (Normcommissie 351 006 "Geotechniek", 2017). A value twice as large was assumed as observations from literature showed that the variation in strength of stabilised soils could be two times larger than the variation in strength of the undisturbed soils (see section 2.4.1.4). Even though the coefficient of variation of the cohesion of both undisturbed soils was also known, a doubling of these coefficients would not have been useful. Those coefficients of variation were unusable, because those coefficients were either negative or exceeding 3,0.

On the other hand, the coefficient of variation on the tangent of the effective angle of internal friction of both stabilised soils was set equal to those of the undisturbed soils. This choice was made as the angle of internal friction was not expected to be the primary component influencing the variability of the drained shear strength. After all, a large variation in the tangent of the effective angle of internal friction would imply that very large friction angles can be achieved, which is not very realistic. Since the effective cohesion can be increased to very large values, most of the variation in the drained shear strength must be caused by variations in the effective cohesion. This justified the choice for only increasing the coefficient of variation on the effective cohesion.

Table 7.1; The applied coefficients of variation and partial material factors for the cohesion and the tangent of the internal friction for the stabilised soil layers at the toe of the levee at the Montfoortse Vaart.

Soil type	$COV_{c'}$	$COV_{tan(\phi')}$
	[-]	[-]
Stabilised peat	0,4	0,03
Stabilised organic clay	0,4	0,09

Although the coefficients of variation were selected keeping observations from literature in mind, it cannot be said with certainty that these coefficients of variation are representative of the examined stabilised soils. This would have to be determined by carrying out sufficient laboratory tests or field tests, both of which were not feasible to do in this study.

Although it is possible that the coefficients of variation of the effective strength parameters of the examined stabilised soils may in fact be different than assumed, such differences are not expected to change the outcome of this study if it were to be repeated in the exact same way. Different coefficients of variation will result in a different target strength of the stabilised soils, which may require a different binder and/or binder to dosage to achieve. Although this leads to a different strength development of the stabilised soils and thus to different implementations, it will still be possible to stabilise the soil in all 4 scenarios examined during the implementation analyses regardless of the measured strength development. After all, the critical part during execution is the stabilisation of the first block(s) of soil. Since this was possible in each scenario, as this only depended on the unit weight and initial strength of the stabilised soils, the stabilisation of the 25 metres of levee will always be possible regardless of the measured strength development. However, if this study were to be expanded, it may very well be possible that a different coefficient of variation of the strength may in fact have a big impact on the result.

7.1.2 Impact of large strength variation

It is not impossible to obtain large variations in the strength of stabilised soils. This was for example observed in the measured unconfined compressive strengths during method 1 of phase 4 of the laboratory research as presented in figure 7.1. These large differences in strength, especially in the stabilised peat samples, were caused either by soil heterogeneity between the untreated soil samples used for mixing or by slight differences in the applied mixing procedure between batches. Such large differences have an impact on the execution, as it needs to be ensured that sufficient strength is developed along the entire trajectory where soil stabilisation has to take place. This could for example be achieved by increasing the binder dosage. This will result in a larger mean strength and, keeping the large variation in strength in mind, also in a sufficiently large design value of the strength. However, this will also result in a more brittle stabilised soil with properties more similar to a 'block of concrete' instead of a soil. This is less desirable at levees, as very stiff stabilised soils are not able to swell and shrink as easily as the undisturbed soils (mostly clays) can upon wetting or drying. It is expected that this could result in crack formation between the stabilised and undisturbed soils which may impact the stability of the levee. Since this is not desirable, it is advisable to keep the strengthening of the soil at levees limited.

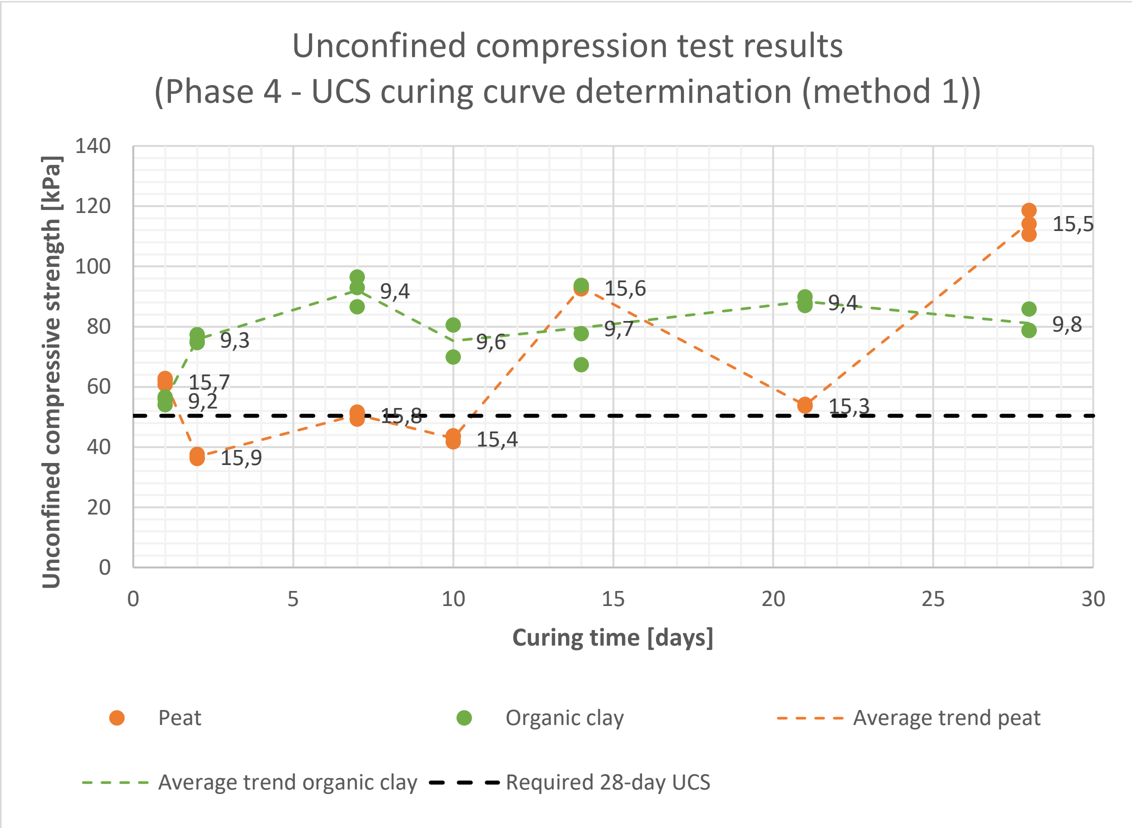

Figure 7.1; Unconfined compressive strengths for the peat and organic clay samples stabilised respectively with 50 and 75 kg Portland cement/m³ undisturbed soil during method 1 of phase 4 of the laboratory research. The numbers in the graph represent the measured water-to-binder factor (w/b) of each tested mixture.

7.2 Laboratory-based approach versus reality

For the determination of the technical feasibility of applying mass stabilisation for reinforcing levees, a laboratory-based approach was applied. Initial stability calculations were used to determine the required strength of the stabilised soils, after which many different soil-binder mixtures were produced in the laboratory to obtain stabilised soil samples meeting the strength requirements from the stability analyses. However, the strengths and the development of the strength measured for the stabilised soils in the laboratory only serve as an indication on what might occur in the field. Although reality is simulated as best as possible in the laboratory, it is, however, not possible to simulate the field perfectly.

Differences between the laboratory and the field include the following:

- In the laboratory, much more heterogeneous stabilised soil samples can be produced than in the field due to visual observation of the degree of homogeneity of the mixture during mixing;
- The size of the stabilised soil samples produced in the laboratory is not representative of the cubic metres of soil stabilised in the field;
- The curing conditions in the laboratory deviate from the curing conditions in the field;
- The mixer used in the geotechnical laboratory of Fugro NL Land B.V. is not representative of the actual 'mixer' used in the field.

First of all, mixtures produced in the laboratory are expected to be more homogeneous than the same mixtures produced in the field (CUR onderzoekscommissie D34 "Kalk-cementkolommen", 2001). This is mostly caused by the ability of the producer to visually assess whether the soil-binder mixture is homogeneous or not, which is difficult if not impossible to determine in the field when stabilising at depth. The expected result is that different strengths and likely larger variations in the strength will be recorded for soils stabilised in the field than for soils stabilised in the laboratory.

Secondly, in the laboratory stabilised soil samples of 150 mm in diameter and between about 240 and about 270 mm in height were produced, equalling a volume between about 4,25 and 4,75 litres. This is much smaller than the cubic metres of soil stabilised in the field. Such differences in scale may result in differences in the strength development because of differences in the total amount of hydration heat released during curing. Larger volumes of stabilised soil may release more hydration heat, possibly resulting in a faster strength development than measured in the laboratory.

Thirdly, the conditions under which the stabilised soil samples cure in the laboratory are likely different from the conditions under which stabilised soils cure in the field. The differences in the curing conditions include the availability of water and the ambient temperature. In the laboratory, the stabilised soil samples were in contact with free-standing water, whereas in the field the availability of water depends on the groundwater level, the rainfall, the evaporation and the hydraulic conductivity of the surrounding soil layers. Additionally, in the laboratory the stabilised soil samples are cured at room temperature (i.e. 19 to 21°C), whereas in the field the stabilised soil samples are cured at different temperatures, depending on the weather, season and time of day. All these differences in curing conditions may cause differences in the achievable strength and the development of the strength between soils stabilised in the field or in the laboratory.

Lastly, the mixer used in the geotechnical laboratory is not comparable to the mixing equipment used in the field. The differences between the mixers are found in both the build-up of the mixers as well as the power with which the soil is mixed. It was therefore not possible to simulate the field mixing in the laboratory with smaller, less powerful mixers. This does not necessarily have to be a problem though, since the objective in both the field and the laboratory is to obtain stabilised soil samples that are as homogeneous as possible.

As a result of these differences between the laboratory and the field, the laboratory measurements only serve as an indication on the possible behaviour of soils stabilised in the field. As a result, it is advised to carry out trial stabilisations in the field and record the strength properties of these stabilised soils in time. The obtained measurements could subsequently be used to verify laboratory measurements, optimise the binder dosage based on the field response, determine the variation in the field strength and optimise the execution scheme.

7.3 Drained stability analyses

In this research, the technical feasibility of reinforcing regional flood defences using mass stabilisation was determined by examining three different criteria. Two of those criteria were the ability of mass stabilisation for solving a stability deficit at levees and the practicability of soil stabilisation at levees. To examine both criteria, drained stability analyses were carried out.

In the Netherlands, the two Dutch levees that were examined in this research are both considered regional flood defences. Regional flood defences are flood defences that protect the land against flooding from inland water, such as from lakes, small rivers and canals (Rijkswaterstaat, n.d. b). In this specific case, both levees were classified as a 'boezemkade', which are relatively small levees holding back water that is maintained at a more or less constant level outside the levee. For such levees, the Dutch STOWA guideline reports that the stability at both high water and daily conditions must be assessed using drained stability analyses with effective strength parameters (i.e. Mohr-Coulomb model) (Stichting Toegepast Onderzoek Waterbeheer, 2015a).

This seems physically logical at daily conditions where the water level (and thus the load on the levee) and the phreatic surface are constant. If a levee is insufficiently stable at daily conditions, then a slow and gradual failure is expected, which is a drained process (Stichting Toegepast Onderzoek Waterbeheer, 2015a). On the other hand, at high water this type of levees holds back water at a level outside the levee of a few tens of centimetres higher than at daily conditions (van Vliet, de Bruin, de Vries, & Zwanenburg, 2017). As a result, the load on the levee only changes a few kilopascals and the phreatic surface in the levee changes only a little. Also, very small pore water overpressures will develop which can dissipate relatively quickly. Therefore there are few differences in the geohydrological conditions between daily conditions and high water. Because of this, the stability of the examined levees at high water is also assessed using drained stability analyses with effective strength parameters.

In the Netherlands, undrained stability analyses are currently only considered for primary flood defences. Primary flood defences are flood defences that protect the land from flooding from outside water, such as from the sea and big rivers (Rijkswaterstaat, n.d. b). For such levees, the water level outside the levee can rise quickly and frequently. When this happens, a possible slope failure should also occur quickly (van Duinen, 2014) because the large pore water overpressures generated in the poorly permeable soils cannot dissipate quick enough.

7.4 Practicability of mass stabilisation at the slope and crest of the levee

Solving a stability deficit at levees was examined by determining the influence of the stabilisation of the soil at the toe, the slope and the crest of the levee on the inward macro-stability. However, for the practicability of these stabilisations, only the stabilisation of the soil at the toe was examined. Although the practicability of mass stabilisation for the examined strips of stabilised soil at the slope and crest of the levee are not examined, it must first be checked whether such an examination is useful at all.

Stabilising a strip of soil at the toe of the levee is well doable. The excavator with the mixing attachment is able to reach the strip of soil to be stabilised rather well. In addition, the weight of the excavator at the toe of the levee will not negatively influence the stability of the levee. Besides this, the preload that is applied on top of the stabilised soil will increase the resistive moment, causing compensation for some of the loss in stability due to (temporary) strength loss of the soil as a result of in-situ mixing.

Stabilising a strip of soil at the slope of the levee is less well doable. Depending on the inclination and the height of the levee, the excavator may have to stabilise the soil at the upper and lower part of the slope by positioning itself at respectively the crest and the toe. In the event the excavator has to stand on top of the levee, the weight of the excavator will cause an additional loading of the levee. This additional loading will increase the driving moment, thereby negatively influencing the stability of the levee during execution. Aside from this, applying a preload on the stabilised soil at the slope will not be easy. The preload will not be stable on the slope unless applied with a small slope. Doing that is in essence the same as decreasing the slope of the levee, which means the importance of stabilisation is cancelled out because decreasing the slope also reinforces the levee.

Stabilising a strip of soil the crest of the levee is difficult. During stabilisation the excavator must be standing on top of the levee in order to be able to stabilise the soil at the crest of the levee. However, the weight of the excavator will exert a load on the levee, which negatively influences the stability of the levee. Furthermore, the preload that must be applied on top of the stabilised soil will also exert a load on the levee, also negatively influencing the stability of the levee during execution. This is why the execution of soil stabilisation at the crest is expected to be more difficult than at the toe of the levee.

Since the stabilisation of the soil at the toe of the levee has many advantages over the stabilisation of the soil at the slope or at the crest, the stabilisation of the soil at the toe is therefore preferable from an implementation perspective. Although the aforementioned aspects show that the execution of soil stabilisation at the slope and the crest is more difficult, it is not expected to be impossible. This is why additional research into the execution of soil stabilisation at the slope and crest of the levee may be useful, also because it is not always possible to stabilise the soil at the toe.

7.5 Applicability research results in general

Although in this study only the reinforcement of two real Dutch levees with mass stabilisation was considered, with the reinforcement of the levee at the Montfoortse Vaart examined in greater detail, the results of this study can nevertheless be used to assess whether mass stabilisation can be applied at other levees. The levees studied have a general geometry for regional flood defences and are built on soft soils, in this case in areas with predominantly peaty subsoil (see figure 7.2).

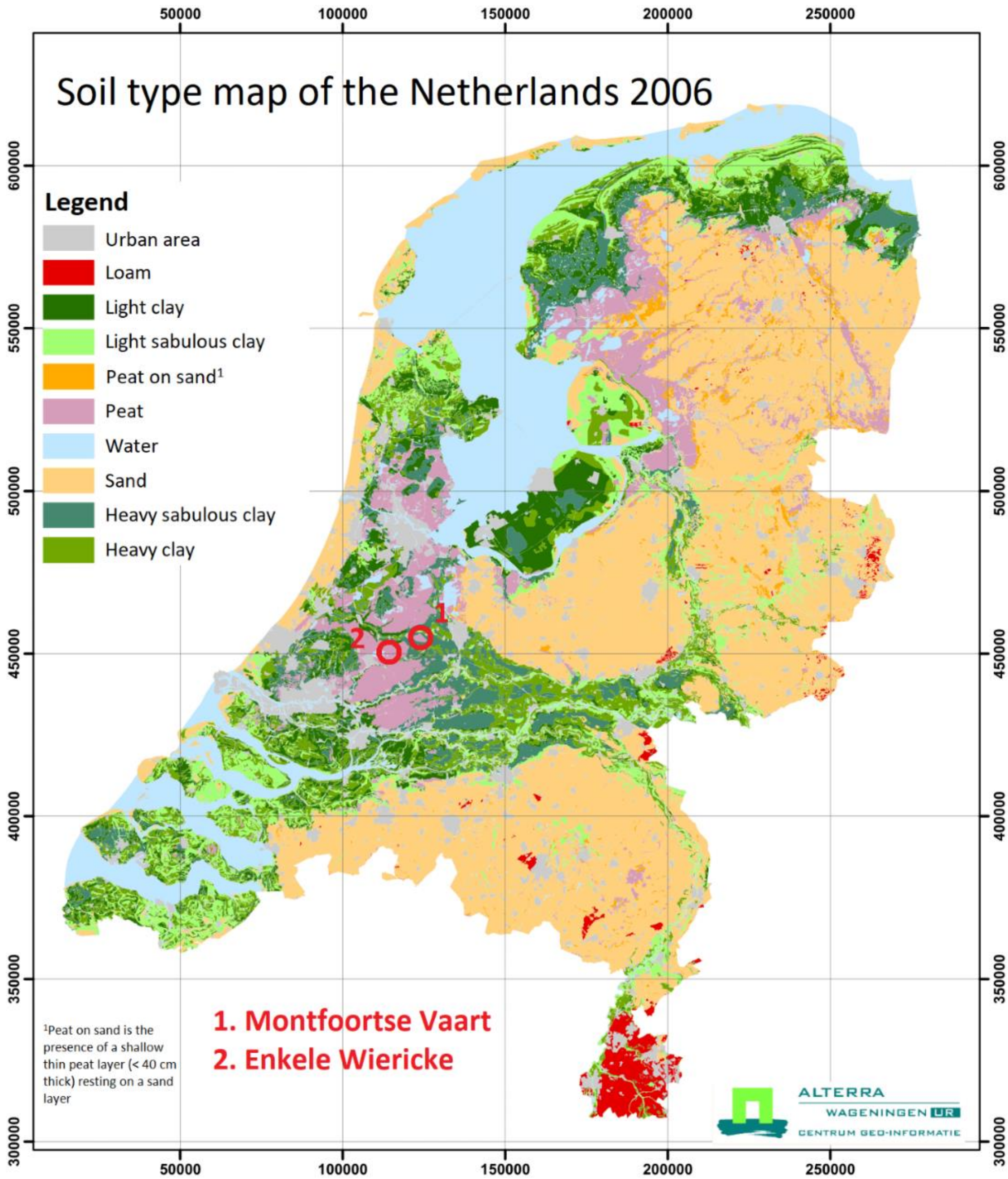

Figure 7.2; Global overview of the types of soil found in the Netherlands (Wageningen University & Research, 2006). The red circles indicate the location of the two levees examined in this research. The scale of the map is in metres.

Since the application of mass stabilisation is solely suited for soft soils, the examined levees were also selected for the presence of soft soils. Besides the locations of the levees considered, peaty subsoil is also found in parts of the northern Netherlands and in other parts of the western Netherlands (see figure 7.2). As this research has shown that a stability deficit at levees with a general geometry on peaty subsoil can be solved, this means that it is likely possible that such a reinforcement method can also be applied at other levees on peaty subsoils elsewhere in the country.

In addition to this, the methodology developed in this study can also be applied to determine the feasibility of reinforcing other levees on similar subsoils with mass stabilisation, even though the results obtained in research, especially with regard to the laboratory research, are solely applicable to the examined levees. Therefore this allows for a broader view of reinforcements of other levees with mass stabilisation despite the case-specific results.

8 Recommendations

8.1 Additional implementation analyses

In this research, the execution of mass stabilisation was examined for a continuous stabilisation using two-dimensional stability analyses with weighted averages on the strength, the unit weight and the preload over a fixed length. Although this type of analyses is useful for obtaining indications on the practicability of the technique at levees, it cannot be used to fully assess the practicability at any spot at the levee due to limitations of the two-dimensional stability analyses.

There are two major aspects of the implementation that could not be examined in this research with the applied method for examining the practicability:

- The settlement of the stabilised soil that occurs due to the preload and its influence on the practicability;
- Three-dimensional effects (arching) in the stability during execution.

The settlement of the stabilised soil that occurs as a result of the preloading has an influence on the stability during execution. After preloading, the stabilised soil compresses, the unit weight of the stabilised soil increases, the ground surface is lowered and thus the stability of the levee is influenced. However, it is currently unknown to what degree the settlement of the stabilised soil will influence both the stability during execution and the final stability of the levee. It is recommended that this is examined in future studies.

Besides the effect of the settlement, there could also be three-dimensional effects during execution that influence the stability. One of these effects is arching. This may play a role in the continuous stabilisation examined in this research as well as in different methods of execution (e.g. discrete blocks of stabilised soil, leaving soil untreated between blocks). For the examined method of execution, it is relevant to know how large the first block of stabilised soil is allowed to be, especially when considering very low initial strengths. Although the stability at the site of the stabilised soil is insufficient, the levee may not fail as part of the undisturbed soil bodies on either side of the stabilised soil block carry some of the load due to arching. A similar phenomenon could occur when considering a different method of execution. Discrete blocks of stabilised soil could be carrying most of the load, preventing failure of the levee even though the stability at the site of the undisturbed soils in between the blocks is insufficient. Since this could not be determined in this research with the analyses carried out, it is recommended to examine this in future studies.

In addition to these two aspects influencing the practicability, it is also recommended to expand upon the implementation analyses by examining the execution of mass stabilisation at the slope and the crest of the levee. Although it is expected that the execution of soil stabilisation at these spots will be more difficult (see section 7.4), it may nevertheless be useful to examine this as it could widen the possibilities for applying mass stabilisation for reinforcing levees.

8.2 Relationship between the laboratory and the field

The measurements carried out on stabilised soil samples produced in the laboratory are likely different from similar stabilisations carried out in the field as evident from literature (see section 2.4.1.2). The strength properties and the strength development of soils stabilised in the laboratory and in the field are expected to differ as a result of different curing conditions. Besides this, the variation in the strength between soil samples stabilised in the laboratory and soils stabilised in the field are also expected to differ due to expected differences in the degree of homogeneity that could be achieved. As a result of these expected differences, it cannot be determined to what extent the laboratory measurements are representative for the field. It is therefore recommended that research be carried out into the relation between the strength and the variation in strength of soils stabilised and cured in the laboratory under controlled conditions and soils stabilised and cured in the field.

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Appendix A - Borings and CPTs

A.1 Montfoortse Vaart

In this section, the borings that were used to make the model in D-GeoStability (see appendix B) for this levee are presented. In figure a.1 the location of the borings that were used as well as the location of the Montfoortse Vaart are shown. Boring 507 and 508 were used to set up the soil profile for the model and are shown in figure a.2 and figure a.3. Boring PB513 was used to determine the hydraulic head in the sand layer and is shown in figure a.4. The legend corresponding to the borings is presented in figure a.5. The borings and the legend for the borings were obtained from a soil investigation report for many regional flood defences in the region of Water Board *Hoogheemraadschap De Stichtse Rijnlanden* (HDSR).

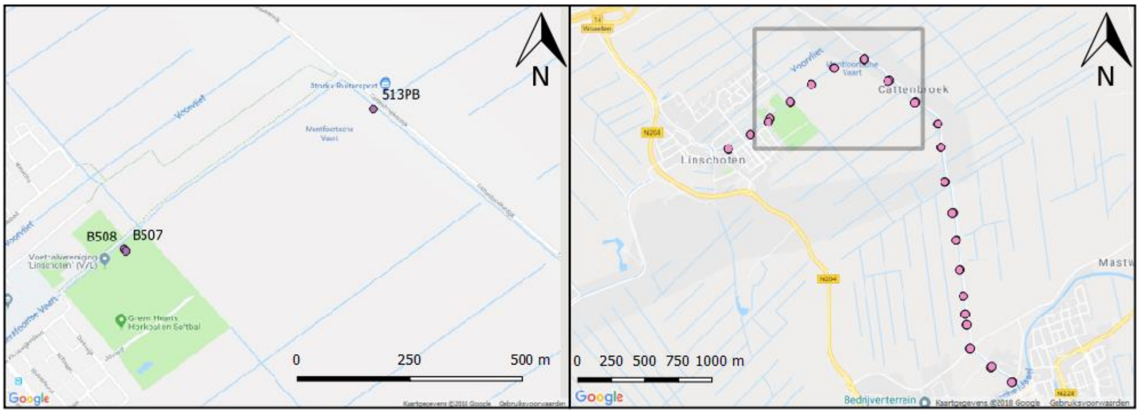
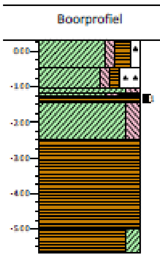


Figure A.1; Map of the location of the Montfoortse Vaart (right image) and the location of the used borings (left image) (Google, 2018a). The dots in the right image display all borings that were made along the Montfoortse Vaart . The dots in the left image represent the location of the borings used, with the name of each boring indicated.

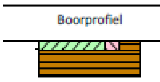
BORING : B507

Datum : 30-05-2016 | X : 123193.490 | Boormeester : EB
GWS : NAP -1.61 m | Y : 453090.060 | Beschrijver : EB
Maaiveld : NAP +0.32 m | Norm : NEN5104
Opmerking : = veldbeschrijving

Ongebruikte monsters : 2, 3, 4.



	Diepte [m t.o.v. NAP] van	tot	Omschrijving grondlaag	Kleur
1	+0.32	-0.48	Klei, zwak siltig, matig humeus, zwak puinhoudend	bruin
2	-0.48	-1.03	Klei, zwak siltig, zwak humeus, sterk puinhoudend	donkerbruin
3	-1.03	-1.18	Klei, matig siltig	grijs
4	-1.18	-1.46	Monster nr. 1	grijsbruin
5	-1.46	-2.48	Klei, matig siltig	grijs
6	-2.48	-3.98	Veen, mineraalarm	donkerbruin
7	-3.98	-4.98	Veen, mineraalarm, sterk houthoudend	donkerbruin
8	-4.98	-5.68	Veen, mineraalarm, matig kleiig	donkerbruin
	0.32	-5.68	Kleistop 1	



Monsternr.	Diepte [m t.o.v. NAP] van	tot	Omschrijving grondlaag	Kleur
1	-1.18	-1.26	Klei, matig siltig, sterk humeus	grijsbruin
	-1.26	-1.46	Veen, mineraalarm	bruin

Figure A.2; Boring B507 as used to set up the soil profile for the 2D model in D-GeoStability. This boring was taken at the crest of the levee (van den Belt, 2016).

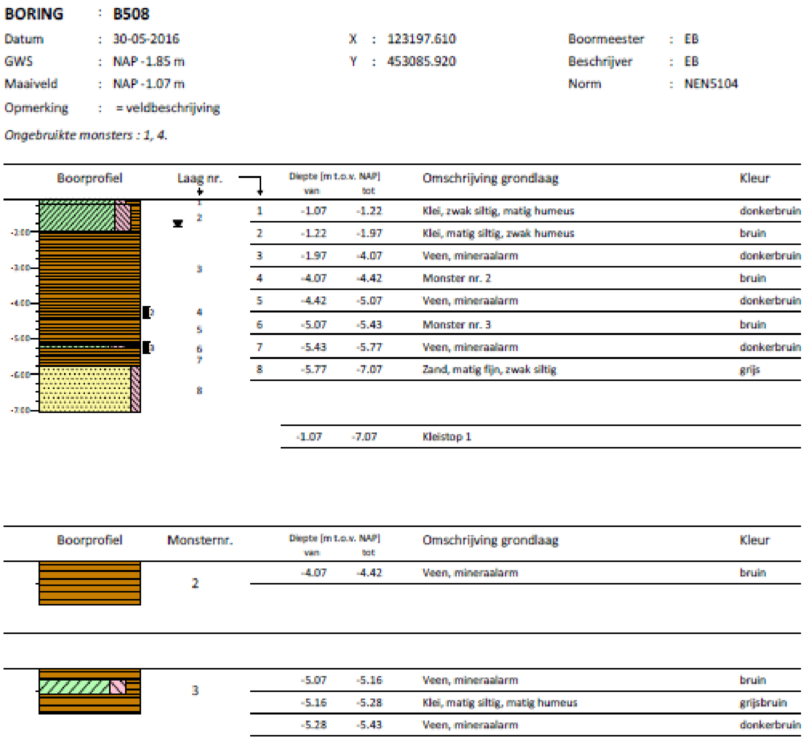


Figure A.3; Boring B508 as used to set up the soil profile for the 2D model in D-GeoStability. This boring was taken at the toe of the levee (van den Belt, 2016).

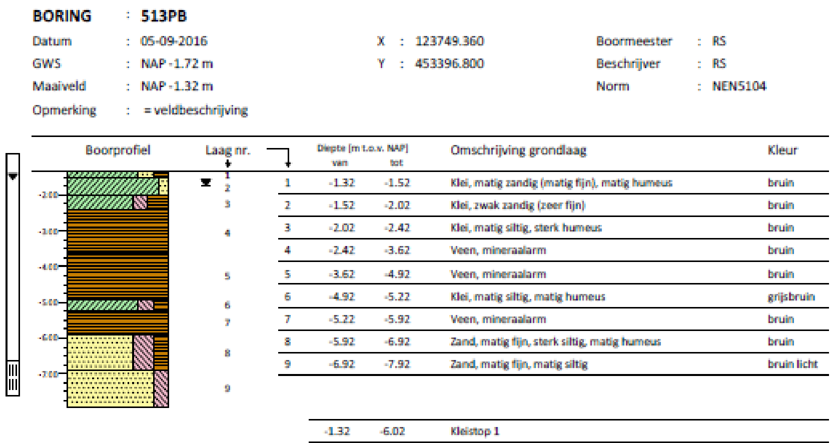


Figure A.4; Boring PB513 as used to determine the hydraulic head in the sand layer. This boring was taken a bit further away from boring B507 and B508. The hydraulic head of the sand layer was measured at -1,56 m NAP (van den Belt, 2016).

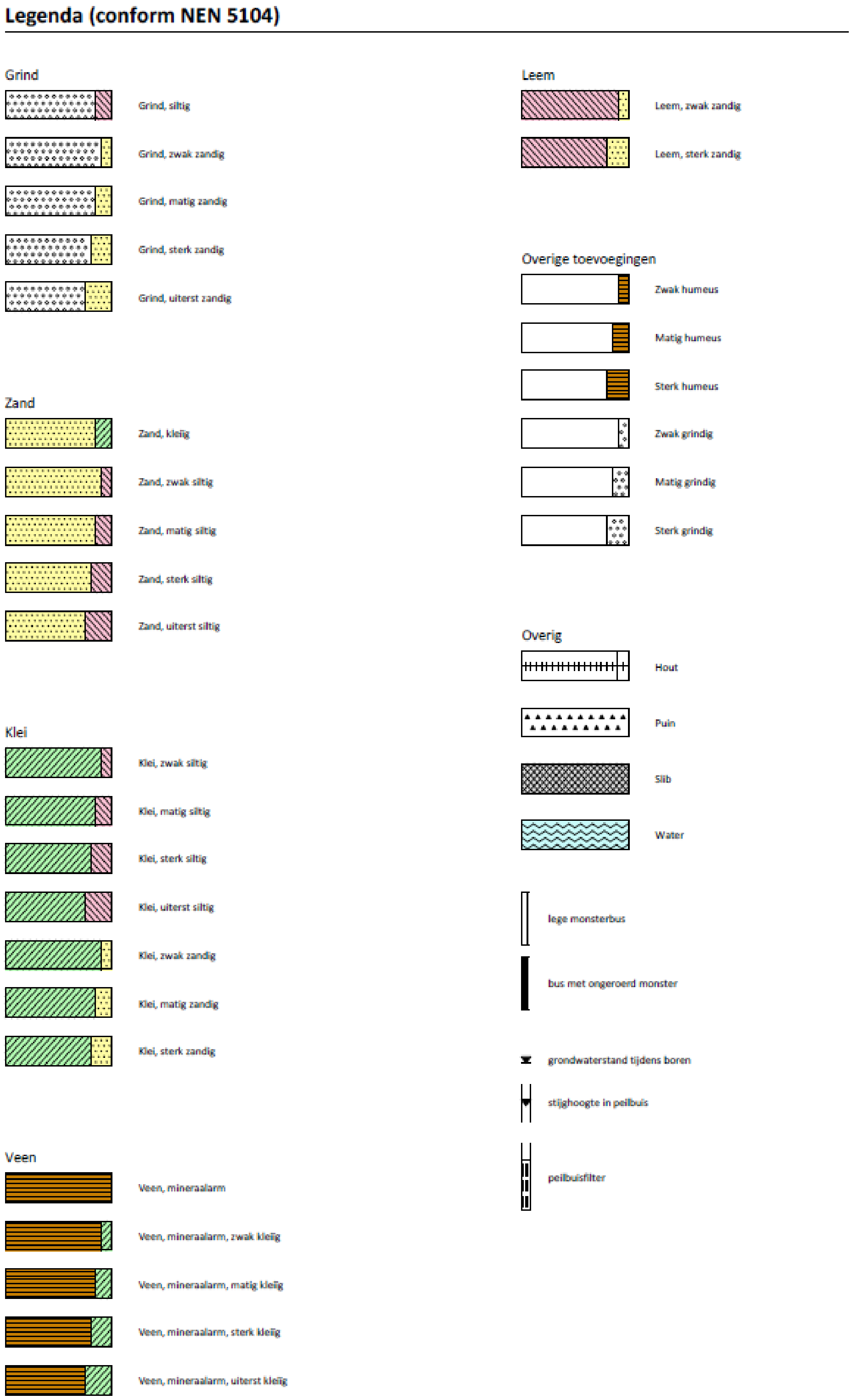


Figure A.5; Legend of the symbols and colours from the borings as shown in figure a.2, figure a.3 and figure a.4 (van den Belt, 2016).

A.2 Enkele Wiericke (polder Oukoop)

In this section, the borings that were used to make the model in D-GeoStability (see appendix B) for this levee are presented. Figure A.6 shows the location of the borings that were used as well as the location of the Enkele Wiericke. Boring T34 B12 KR, T34 B12 TAL and T34 B12 TE were used to set up the soil profile for the model and are shown in figure a.7, figure a.8 and figure a.9 respectively. The legend corresponding to the borings is presented in figure a.10. Besides the borings, a CPT at the crest was also used to set up the soil profile with. This CPT along with its interpretation are shown in figure a.11 and table a.1 respectively. The borings, the CPT and the legend for the borings were obtained from a soil investigation report along the southern part of the Enkele Wiericke.

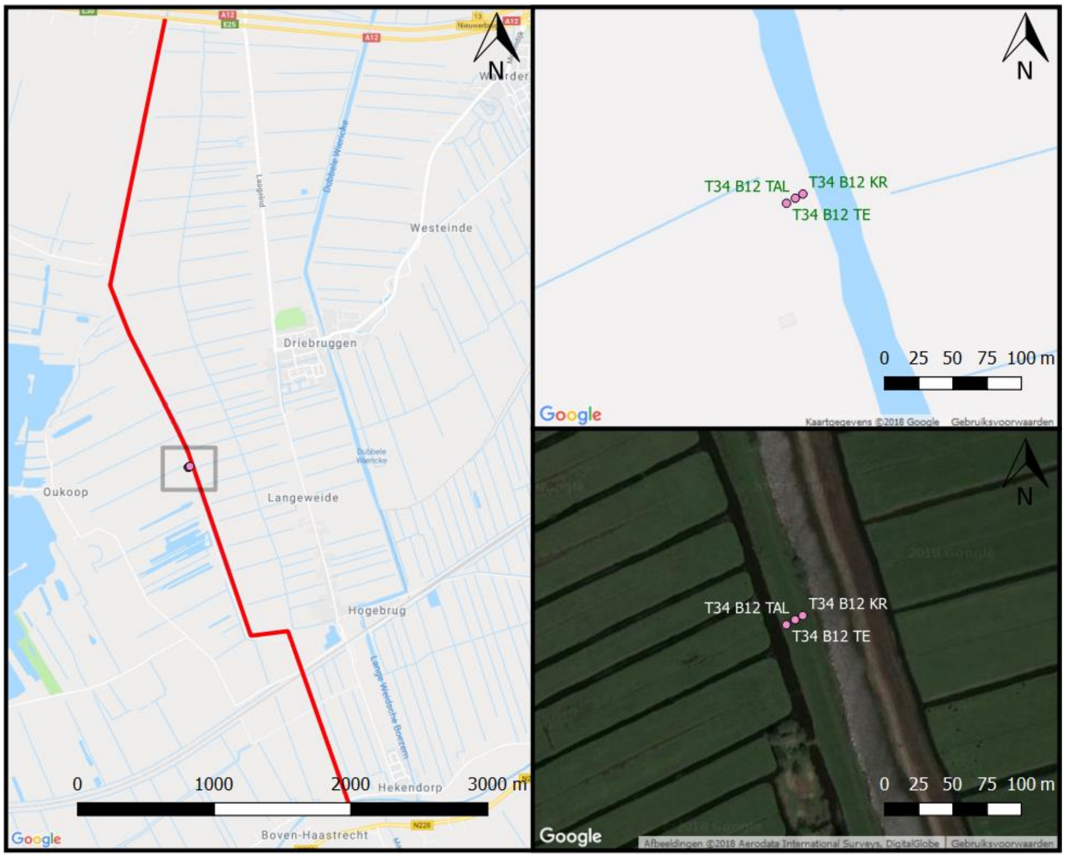


Figure A.6; Map of the location of the Enkele Wiericke (red line in left image) and the location of the used borings (right images) (Google, 2018b). The dots in the right images represent the borings that were made along the Enkele Wiericke that were used to make the model in D-GeoStability. The names of the borings are displayed in the right images as well.

Figure A.7; Boring T34 B12 KR as used to set up the soil profile for the 2D model in D-GeoStability. This boring was taken at the crest of the levee (de Vries, 2013).

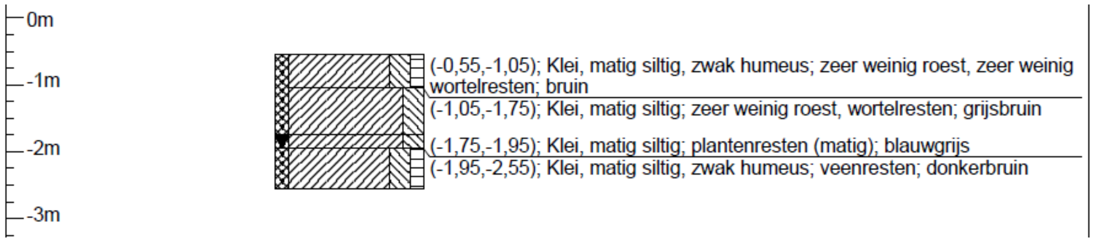


Figure A.8; Boring T34 B12 TAL as used to set up the soil profile for the 2D model in D-GeoStability. This boring was taken at the slope of the levee (de Vries, 2013).

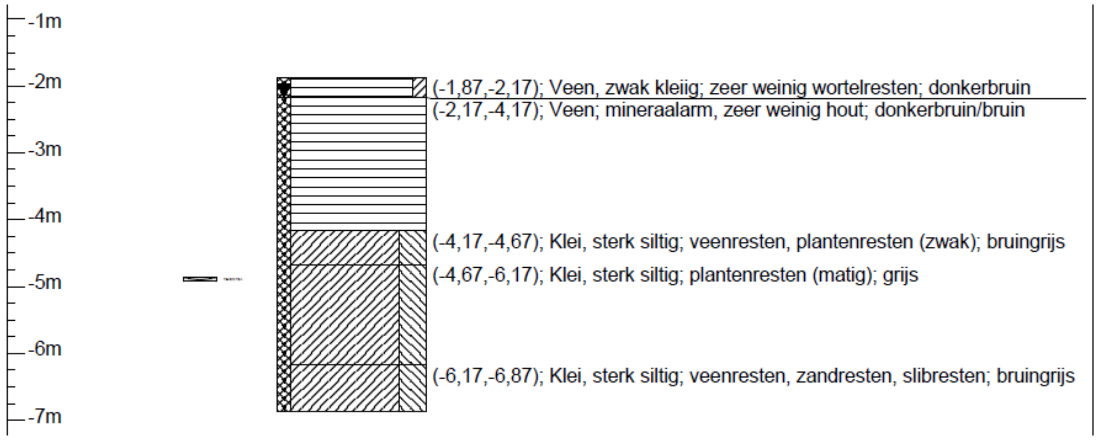


Figure A.9; Boring T34 B12 TE as used to set up the soil profile for the 2D model in D-GeoStability. This boring was taken at the toe of the levee (de Vries, 2013).

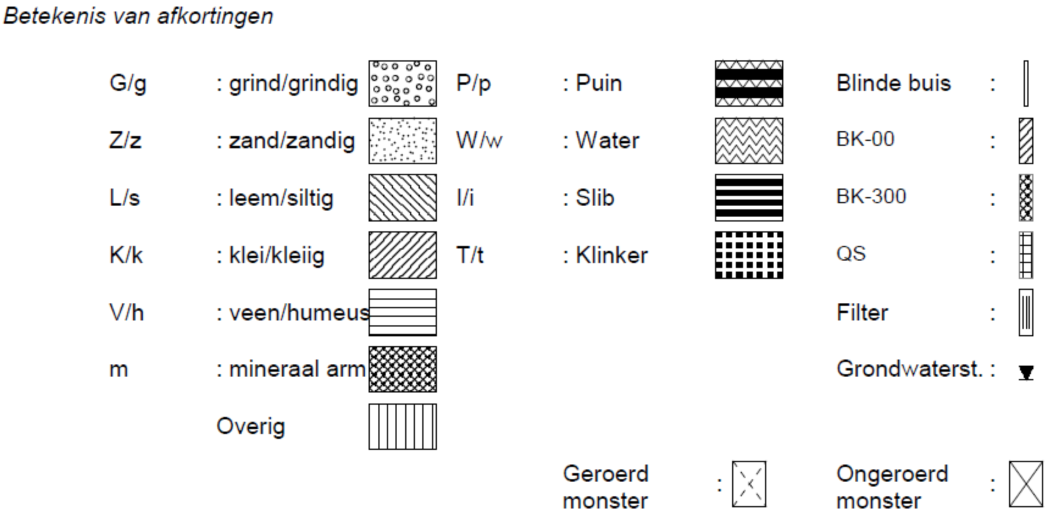


Figure A.10; Legend of the symbols from the borings as shown in figure a.7, figure a.8 and figure a.9 (de Vries, 2013).

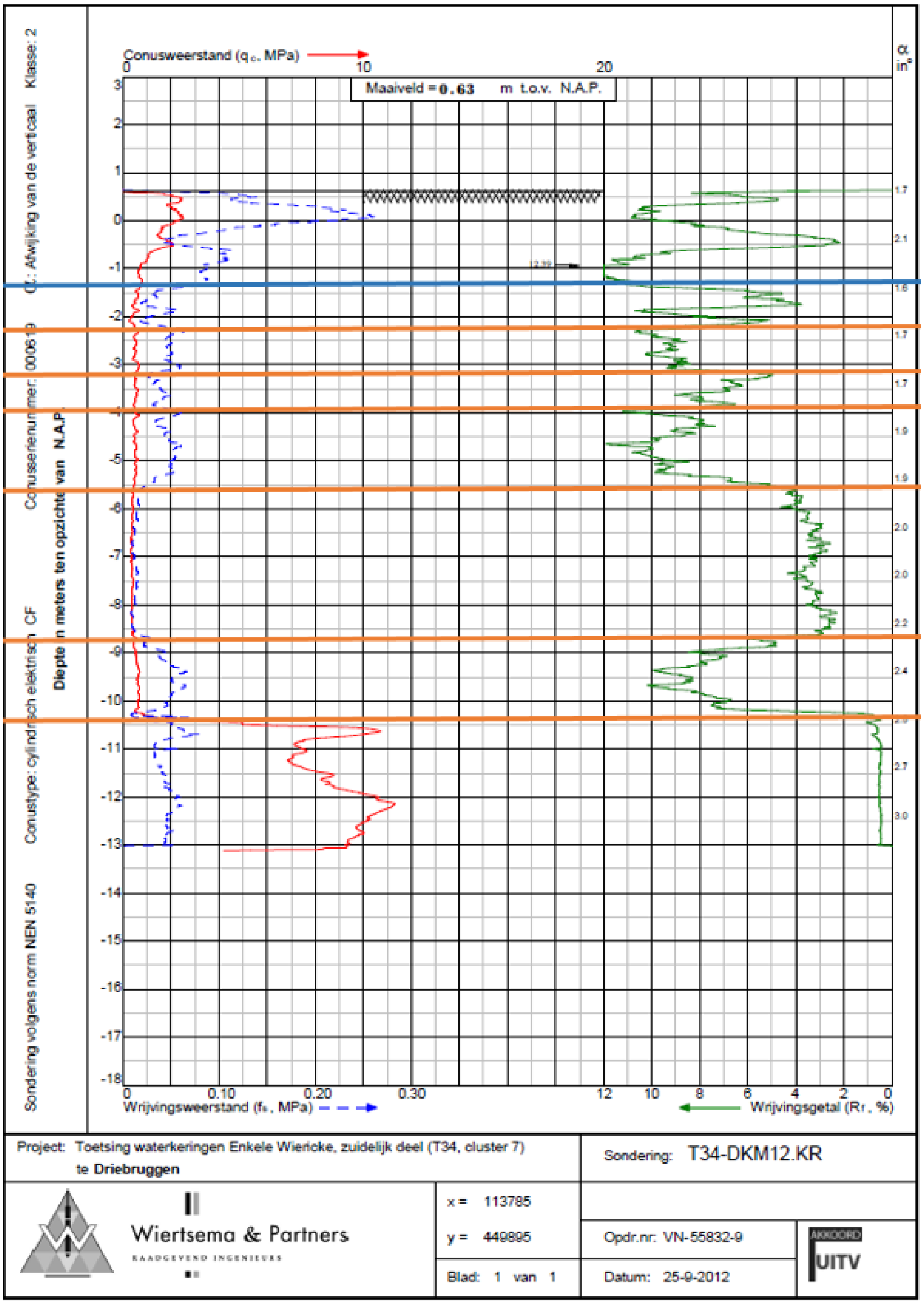


Figure A.11; CPT T34-DKM12.KR as used to set up the soil profile for the 2D model in D-GeoStability. This CPT was taken at the crest of the levee (same spot as boring T34 B12 KR) (de Vries, 2013).

Table A.1; Interpretation of the CPT from figure a.11.

Depth [m NAP]	Average q_c [MPa]	Average R_f [%]	Interpretation
At -1,2	-	-	Phreatic groundwater level
From -1,2 to -2,1	0,6	5,0	Silty clay
From -2,1 to -3,2	0,5	9,2	Peat
From -3,2 to -3,8	0,5	7,0	Organic clay
From -3,8 to -5,6	0,5	9,0	Peat
From -5,6 to -8,7	0,4	3,5	Silty clay
From -8,7 to -10,3	0,6	7,5	Peat
Below -10,3	8,5	0,5	Sand

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Appendix B - Principles and boundary conditions

B.1 Montfoortse Vaart

This section presents the principles and boundary conditions used to assess the inward macro-stability of the levee at the Montfoortse Vaart.

B.1.1 Soil profile

At the selected spot along the levee, the soil profile was derived using borings carried out at the site. The borings that were used for this derivation are presented in appendix A, section A.1. The soil layering that was derived is presented in table b.1.

Table B.1; Soil profile of the cross-section at the levee of the Montfoortse Vaart. The borings that were used to derive this soil profile are presented in appendix A, section A.1.

Soil layer	Lower boundary at the crest	Lower boundary at the toe
	[m NAP]	[m NAP]
Surface	+0,32	-1,07
Silty clay	-2,88	Does not exist at toe
Organic clay	Does not exist at toe	-2,37
Peat, poor in minerals	-5,77	-5,77
Sand	-7,07 (end of measurements)	-7,07 (end of measurements)

B.1.2 Soil parameters

Antea Group had commissioned laboratory tests on soil samples, which were obtained from borings near various regional flood defences within the region of water board *Hoogheemraadschap De Stichtse Rijnlanden* (HDSR). For all soil samples, the unit weight and the drained shear strength parameters were measured. In accordance with the current prevailing Dutch STOWA guideline for the assessment of the safety of regional flood defences, the drained shear strength parameters were determined at low strains of two or five percent (STOWA, 2015a). In the laboratory, the drained strength parameters of the peat soil samples were determined at 5% shear strain using DSS tests. For clay soil samples, the drained shear strength parameters were determined at 2% axial strain using consolidated, undrained triaxial compression tests. Using all measurements, a single dataset of soil parameters was made for every soil layer present in the region of Water Board HDSR. From this database, the 5% characteristic values of the soil parameters were determined. These values were used in this research for the soil layers at the levee of the Montfoortse Vaart and are presented in table b.2.

No soil samples had been taken from the sand layer. As such, no laboratory tests were carried out for the sand layer. Therefore, it was assumed that the sand layer was a moderately packed sand with characteristic values of the unit weight and the drained shear strength parameters based on table 2.b of Dutch standard NEN 9997-1 (Bardoel & Eshuis, 2017).

Table B.2; 5% characteristic values of the soil parameters of the various soil layers at the Montfoortse Vaart (Bardoel & Eshuis, 2017).

Soil layer	Bulk unit weight	Saturated unit weight	Effective angle of internal friction	Effective cohesion
	$\gamma_{bulk;k}$	$\gamma_{sat;k}$	ϕ'_k	c'_k
	[kN/m^3]	[kN/m^3]	[°]	[kPa]
Silty clay	15,95	15,95	30,7	2,41
Organic clay	12,80	12,80	34,8	1,00
Peat, poor in minerals	10,00	10,00	15,1	1,00
Sand	19,00	21,00	32,5	0,00

In order to determine the design values of the soil parameters, the characteristic values of the soil parameters need to be divided by a partial material factor. Module C of the Dutch STOWA guideline for the assessment of the safety of regional flood provides the partial material factors for the different soil layers. The module distinguishes between partial material factors for soil parameters determined by DSS test or consolidated, undrained triaxial tests. Therefore, different partial material factors are used for the clay and the peat layer. The partial material factors that were used for this case are presented in table b.3. The design values of the soil parameters were determined using these partial material factors and are presented in table b.4.

Table B.3; Material factors for the soil parameters of the various soil layers at the Montfoortse Vaart (STOWA, 2015c).

Soil layer	Material factor		
	$\gamma_{mat,\gamma}$	$\gamma_{mat,\tan(\phi')}$	$\gamma_{mat,c'}$
	[-]	[-]	[-]
Silty clay	1,0	1,15	1,2
Organic clay	1,0	1,15	1,2
Peat, poor in minerals	1,0	1,2	1,5
Sand	1,0	1,15	-

Table B.4; Design values of the soil parameters of the various soil layers at the Montfoortse Vaart.

Soil layer	Bulk unit weight	Saturated unit weight	Effective angle of internal friction	Effective cohesion
	$\gamma_{bulk;d}$	$\gamma_{sat;d}$	ϕ'_d	c'_d
	[kN/m³]	[kN/m³]	[°]	[kPa]
Silty clay	15,95	15,95	27,3	2,01
Organic clay	12,80	12,80	31,1	0,83
Peat, poor in minerals	10,00	10,00	12,7	0,67
Sand	19,00	21,00	29,0	0,00

B.1.3 Load

For the model, a temporary load of 5,0 kN/m² with a spread of 0° over a width of 2,5 metres was used. Additionally, the degree of consolidation for all (stabilised) cohesive layers was put at 50% for a load of 5,0 kN/m² (levee without roads). The degree of consolidation for the sand layer was put at 100%. Both the load and corresponding degree of consolidation were set in accordance with module B of the STOWA guideline for the assessment of the safety of regional flood defences (STOWA, 2015b).

B.1.4 (Ground)water level

The water levels relevant for this levee are shown in table b.5. The high water level that the levee should be able to hold is -0,05 m NAP (van Doorn, 2015). This is the water level of the Montfoortse Vaart that was tested for in the model.

Table B.5; Water levels in the Montfoortse Vaart (van Doorn, 2015) and groundwater levels in the subsurface (Hoogheemraadschap De Stichtse Rijnlanden (HDSR), 2011).

Normal water level in the Montfoortse Vaart	High water level in the Montfoortse Vaart	Polder water level	Water level sand layer
[m NAP]	[m NAP]	[m NAP]	[m NAP]
-0,47	-0,05	-1,87	-1,56

B.1.5 Phreatic groundwater level schematisation

The schematisation of the phreatic groundwater level (see figure b.1) inside the levee and the polder at high water conditions was based on the principles of water board HDSR (van Korlaar, 2015):

- A high water level of -0,05 m NAP at the outer side of the levee (van Doorn, 2015);
- At the outer crest of the levee, the phreatic water level is at the normal water level of the Montfoortse Vaart (-0,47 m NAP) (van Doorn, 2015);
- At the inner crest of the levee, the phreatic water level is 0,20 metres lower than the normal water level;
- The high polder groundwater level is -1,87 m NAP (Hoogheemraadschap De Stichtse Rijnlanden (HDSR), 2011).

A dry situation for this model was not considered.

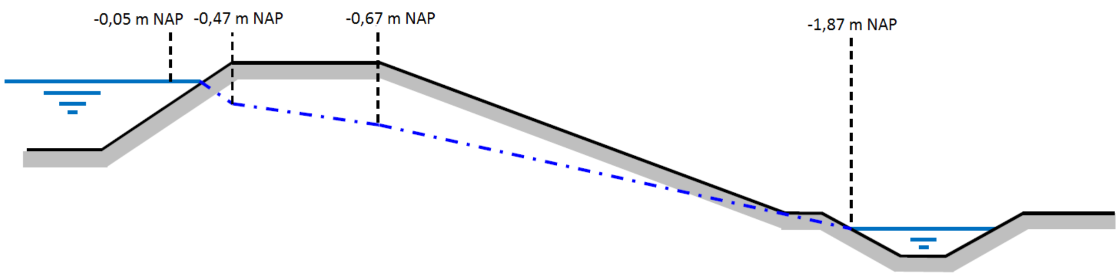

Figure B.1; Schematisation of the phreatic groundwater level at the levee of the Montvoortse Vaart at high water conditions.

B.1.6 Penetration length

The penetration length of the pore water pressure in the sand was put at 1,0 metre into the peat layer based on the principles of water board HDSR (van Korlaar, 2015).

B.1.7 Surface elevation

In order to determine the surface elevation of the levee and the surrounding polders, raster data from the *Actueel Hoogtebestand Nederland* (AHN) was used. The used data was 0,5 metre raster Digital Terrain Model (DTM) data from AHN version 3 (AHN3) and was obtained from the *Publieke Dienstverlening Op de Kaart* (PDOK, 2018).

The slope of the levee at the waterside was assumed to be 1:1,5 based on the information in the *Legger oppervlaktewater* of Water Board HDSR (Hoogheemraadschap De Stichtse Rijnlanden (HDSR), 2012). The depth of the Montfoortse Vaart in metres below NAP was not given in the *Legger oppervlaktewater*, but the water depth was mentioned to be 2,0 metres (Hoogheemraadschap De Stichtse Rijnlanden (HDSR), 2012). Therefore the maximum depth in the Montfoortse Vaart was assumed to be 2,0 metres below the normal water level of -0,47 m NAP, which equals -2,47 m NAP.

The combined information on the surface elevation and the principles and boundary conditions as presented in section B.1.1 through section B.1.6 were used to set up the model in D-GeoStability. The model as used to assess the safety of the levee at the Montfoortse Vaart is shown in figure b.2.

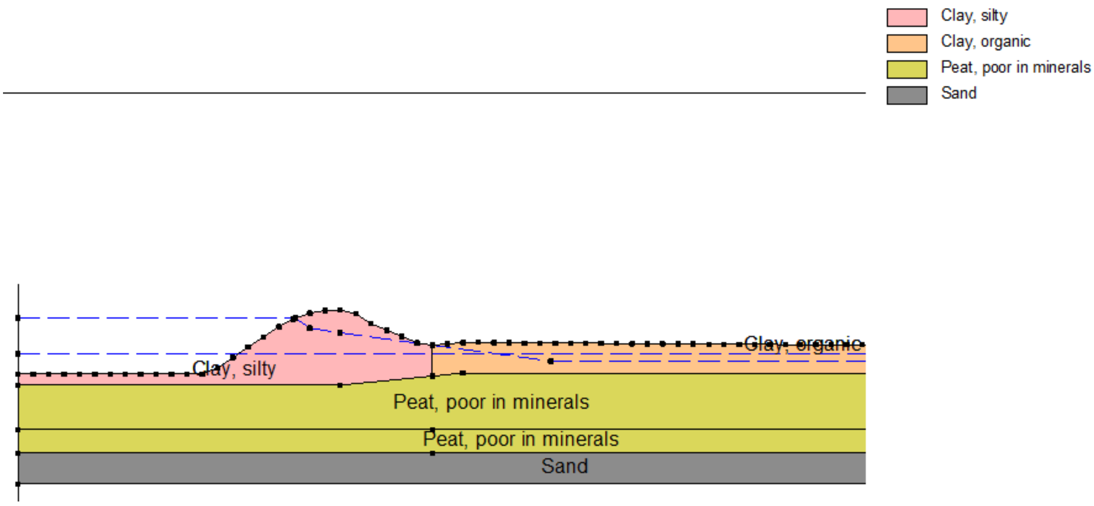

Figure B.2; D-GeoStability model of the selected cross-section at the levee of the Montfoortse Vaart.

B.1.8 Required factor of safety

The required factor of safety against inward macro-instability of regional flood defences needs to be determined with equation (B-1) according to the Dutch STOWA guideline for the assessment of the safety of regional flood defences (STOWA, 2015d).

$$FoS_{required} = \gamma_n \cdot \gamma_d \cdot \gamma_s \tag{B-1}$$

where:

$FoS_{required}$	*- required factor of safety against instability of the regional flood defence*	*[-]*
γ_n	*- damage factor*	*[-]*
γ_d	*- model factor*	*[-]*
γ_s	*- schematisation factor*	*[-]*

The levee at the Montfoortse Vaart is a regional flood defence of safety class II and therefore has a damage factor of 0,85 (STOWA, 2015c). For the schematisation factor a value of 1,2 was assumed (STOWA, 2015a). When making calculations with the Bishop calculation model in D-GeoStability, a model factor of 1,0 needs to be used, whereas for the Uplift Van calculation model a model factor of 1,05 needs to be used. When using the Spencer model, a model factor of 0,95 needs to be applied when uplift is not likely to occur or a model factor of 1,05 needs to be applied when uplift is likely to occur (STOWA, 2015c).

Using these values for the damage factor, the schematisation factor and the model factor, the required factors of safety were derived for each calculation model. These factors of safety are presented in table b.6.

Table B.6; The required factor of safety for the levee at the Montfoortse Vaart for various calculation models.

Model D-GeoStability	Required factor of safety
Spencer (no uplift)	0,97 $(= 0{,}85 \cdot 0{,}95 \cdot 1{,}2)$
Spencer (with uplift)	1,07 $(= 0{,}85 \cdot 1{,}05 \cdot 1{,}2)$
Bishop	1,02 $(= 0{,}85 \cdot 1{,}00 \cdot 1{,}2)$
Uplift Van	1,07 $(= 0{,}85 \cdot 1{,}05 \cdot 1{,}2)$

B.2 Enkele Wiericke (polder Oukoop)

This section presents the principles and boundary conditions used to assess the inward macro-stability of the levee at the Enkele Wiericke.

B.2.1 Soil profile

At the selected spot along the levee, the soil profile was derived using borings and a CPT carried out at the site. The borings and the CPT that were used for this derivation are presented in appendix A, section A.2. The soil layering that was derived is presented in table b.7.

Table B.7; Soil profile of the cross-section at the levee at the Enkele Wiericke. The borings and CPT that were used to derive this soil profile are presented in appendix A, section A.2.

Soil layer	**Lower boundary at the crest**	**Lower boundary at the slope**	**Lower boundary at the toe**
	[m NAP]	**[m NAP]**	**[m NAP]**
Surface	+0,50	-0,40	-1,90
Silty clay	-2,10	-2,60 (end of measurements)	Does not exist at toe
Peat, water content > 300%	-3,20	-	-4,15
Organic clay	-3,80	-	Does not exist at toe
Peat, water content > 300%	-5,60	-	Does not exist at toe
Silty clay	-8,70	-	-6,90 (end of measurements)
Peat, water content > 300%	-10,30	-	-
Pleistocene sand	-15,00 (end of measurements)	-	-

B.2.2 Soil parameters

For this case the soil parameters were taken from the *Nota Waterkeringen III* of Water Board *Hoogheemraadschap van Rijnland* (Rijnland). The document only specifies the design values of the soil parameters to be used for the various soil layers found within the region of Water Board Rijnland. As such, the characteristic and mean values of the soil parameters are not known. The design values of the soil parameters are presented in table b.8.

A peat layer surfaces at the toe of the levee. It was assumed that the first 30 cm of this peat layer at the surface has larger unit weight (van Joolingen, 2016).

Table B.8; Design values of the soil parameters of the various soil layers at the Enkele Wiericke (van Joolingen, 2016).

Soil layer	**Bulk unit weight**	**Saturated unit weight**	**Effective angle of internal friction**	**Effective cohesion**
	$\gamma_{bulk;d}$	$\gamma_{sat;d}$	ϕ'_d	c'_d
	[kN/m³]	**[kN/m³]**	**[°]**	**[kPa]**
Peat, surface (first 30 cm)	12,5	12,5	20,0	2,0
Peat, water content > 300%	10,3	10,3	20,0	2,0
Organic clay	13,3	13,3	25,5	1,4
Silty clay	15,4	15,4	26,8	2,8
Pleistocene sand	18,0	20,0	32,5	0,0

B.2.3 Load

For the model, a temporary load of 5,0 kN/m^2 with a spread of 0° over a width of 2,5 metres was used. Additionally, the degree of consolidation for all cohesive layers was put at 50% for a load of 5,0 kN/m^2 (levee without roads). The degree of consolidation for the sand layer was put at 100%. Both the load and corresponding degree of consolidation were set in accordance with module B of the Dutch STOWA guideline for the assessment of the safety of regional flood defences (STOWA, 2015b).

B.2.4 (Ground)water level

The water levels relevant for this levee are shown in table b.9. The high water level that the levee should be able to hold is -0,20 m NAP (van Joolingen, 2016). This is the water level of the Enkele Wiericke that was tested for in the model.

Table B.9; Water levels in the Enkele Wiericke and groundwater levels in the subsurface (van Joolingen, 2016).

Normal water level in the Enkele Wiericke	**High water level in the Enkele Wiericke**	**Polder water level**	**Water level sand layer**
[m NAP]	**[m NAP]**	**[m NAP]**	**[m NAP]**
-0,47	-0,20	-2,20	-2,10

B.2.5 Phreatic groundwater level schematisation

The schematisation of the phreatic groundwater level (see figure b.3) inside the levee and polder at high water was based on the starting points of Water Board Rijnland (Hoogheemraadschap van Rijnland, 2013):

- A high water level of -0,20 m NAP at the outer side of the levee;
- At the inner crest of the levee, the phreatic water level is at the normal water level of the Enkele Wiericke (-0,47 m NAP);
- The high polder groundwater level is -2,20 m NAP.

A dry situation for this model was not considered.

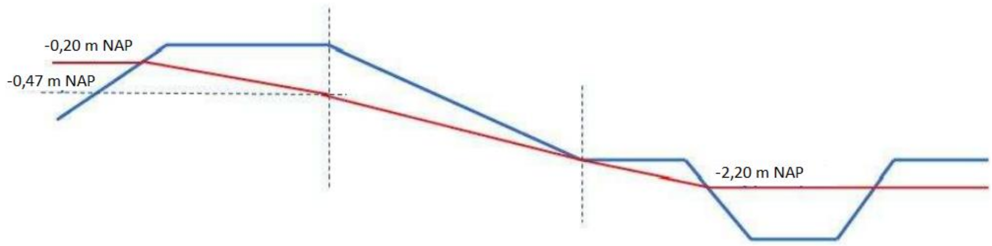

Figure B.3; Schematisation of the phreatic groundwater level at the levee of the Enkele Wiericke at high water conditions.

B.2.6 Pore water pressures

The pore water pressures in the soil layers underneath the upper silty clay layer, but above the Pleistocene sand layer, are interpolated between the pore water pressure at the bottom of the upper silty clay layer and the pore water pressure at the top of the sand layer. The pore water pressure at the bottom of the upper silty clay layer is determined using the phreatic groundwater level. The pore water pressure at the top of the sand layer is determined using the phreatic groundwater level as measured in the sand layer (van Joolingen, 2016).

B.2.7 Surface elevation

The surface elevation was measured on beforehand for the selected cross-section at the levee. Combining the information on the surface elevation and the principles and boundary conditions as presented in section B.2.1 through section B.2.7 were used to set up the model in D-GeoStability. The model as used to assess the safety of the levee at the Montfoortse Vaart is shown in figure b.4.

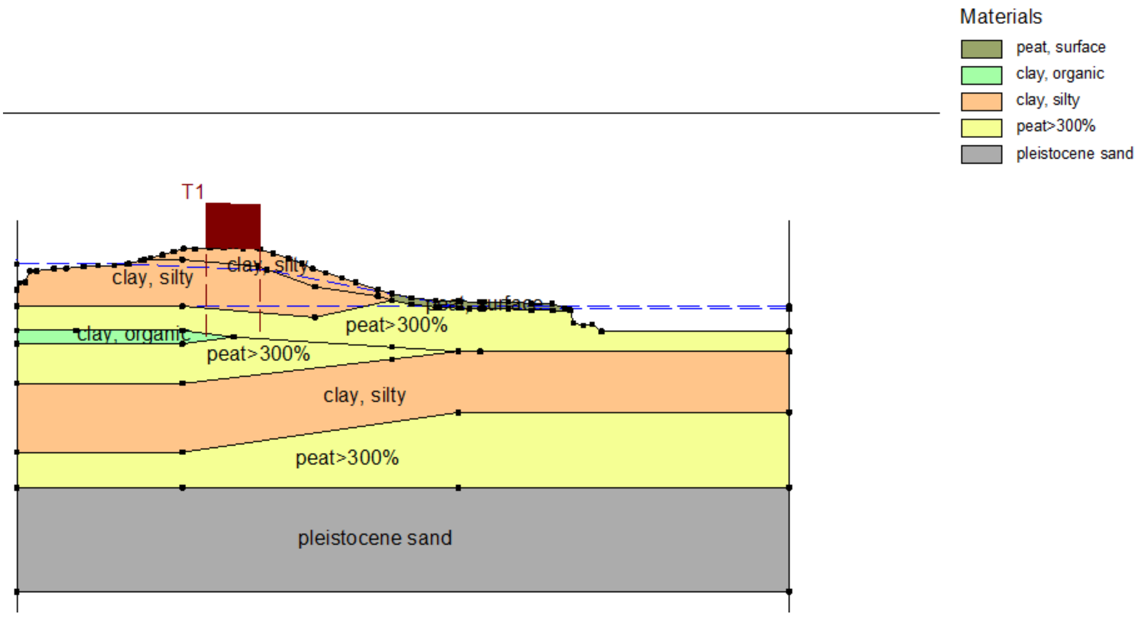

Figure B.4; D-GeoStability model of the selected cross-section at the levee of the Enkele Wiericke.

B.2.8 Required factor of safety

The required factor of safety for stability of regional flood defences needs to be determined with equation (B-2) according to the STOWA guideline for the assessment of the safety of regional flood defences (STOWA, 2015d).

$$FoS_{required} = \gamma_n \cdot \gamma_d \cdot \gamma_s \tag{B-2}$$

where:

$FoS_{required}$	*- required factor of safety against instability of the regional flood defence*	*[-]*
γ_n	*- damage factor*	*[-]*
γ_d	*- model factor*	*[-]*
γ_s	*- schematisation factor*	*[-]*

The Enkele Wiericke levee is a regional flood defence of safety class III and therefore has a damage factor of 0,90 (STOWA, 2015c). For the schematisation factor a value of 1,2 was assumed (STOWA, 2015a). When making calculations with the Bishop calculation model in D-GeoStability, a model factor of 1,0 needs to be used, whereas for the Uplift Van calculation model a model factor of 1,05 needs to be used. When using the Spencer model, a model factor of 0,95 needs to be applied when uplift is not likely to occur or a model factor of 1,05 needs to be applied when uplift is likely to occur (STOWA, 2015c).

Using these values for the damage factor, the schematisation factor and the model factor, the required factors of safety were derived for each calculation model. These factors of safety are presented in table b.10.

Table B.10; The required factor of safety for the levee at the Enkele Wiericke for various calculation models.

Model D-GeoStability	Required factor of safety
Spencer (no uplift)	1,03 $(= 0{,}90 \cdot 0{,}95 \cdot 1{,}2)$
Spencer (with uplift)	1,13 $(= 0{,}90 \cdot 1{,}05 \cdot 1{,}2)$
Bishop	1,08 $(= 0{,}90 \cdot 1{,}0 \cdot 1{,}2)$
Uplift Van	1,13 $(= 0{,}90 \cdot 1{,}05 \cdot 1{,}2)$

B.3 Implementation analysis Montfoortse Vaart

This section presents the principles and boundary conditions used to assess the inward macro-stability of the levee at the Montfoortse Vaart during the implementation analysis. Few changes in the principles and boundary conditions were made with respect to principles and boundary conditions presented in section B.1. The changes in the principles and boundary conditions that were made are presented in this section.

B.3.1 Load

For the model, a temporary load of 5,0 kN/m^2 with a spread of 0° over a width of 2,5 metres was applied on top of the crest of the levee. On top of the stabilised soil a temporary load of 8,0 kN/m^2 with a spread of 0° was applied (CUR-onderzoekscommissie D34, 2001). Additionally, the degree of consolidation for all (stabilised) cohesive layers was put at 50% for a load of 5,0 kN/m^2 on the levee (levee without roads). The degree of consolidation for the sand layer was put at 100%. Both the load on the levee and corresponding degree of consolidation for all layers were set in accordance with module B of the STOWA guideline for the assessment of the safety of regional flood defences (STOWA, 2015b).

B.3.2 Phreatic groundwater level schematisation

The schematisation of the phreatic groundwater level (see figure b.5) inside the levee and the polder at daily (normal) water conditions was made as follows:

- A normal water level of -0,47 m NAP at the outer side of the levee (van Doorn, 2015);
- An interpolation of the water level at the outer side of the levee through the inner toe to the high polder groundwater level of -1,87 m NAP (Hoogheemraadschap De Stichtse Rijnlanden (HDSR), 2011).

A dry situation for this model was not considered.

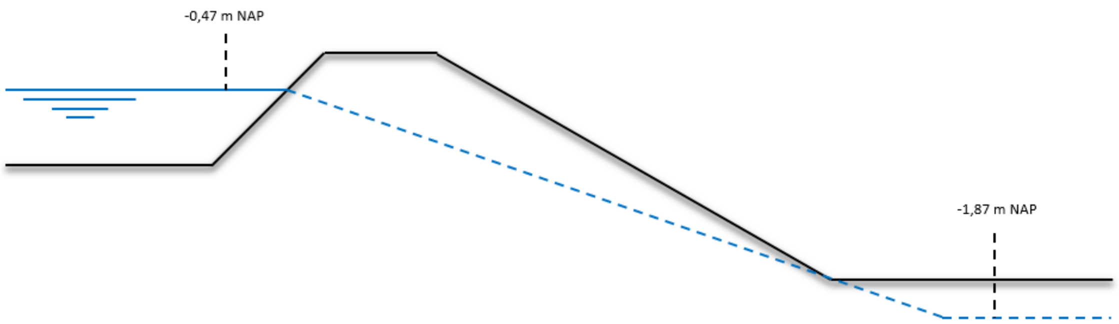

Figure B.5; Schematisation of the phreatic groundwater level at the levee of the Montvoortse Vaart at daily water conditions.

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Appendix C - Determination of the required strength

C.1 Desired strength of stabilised soil from selected case

In this section, the derivation of the minimum mean value of the unconfined compressive strength for all soils stabilised at the toe of the levee at the Montfoortse Vaart is presented. An indication on the required mean unconfined compressive strength is required for the laboratory research.

C.1.1 Determination required design values of effective strength parameters

In order to determine the minimum mean value of the unconfined compressive strength, first it needed to be determined which minimum design values of the effective strength parameters were required to reach the required Factor of Safety against inward macro-instability. For this case, a stabilisation of a peat layer and an organic clay layer at the toe of the levee were considered as shown in figure c.1. The current and required Factor of Safety for this levee are presented in table c.1. The derivation of the required Factor of Safety is presented in appendix B.

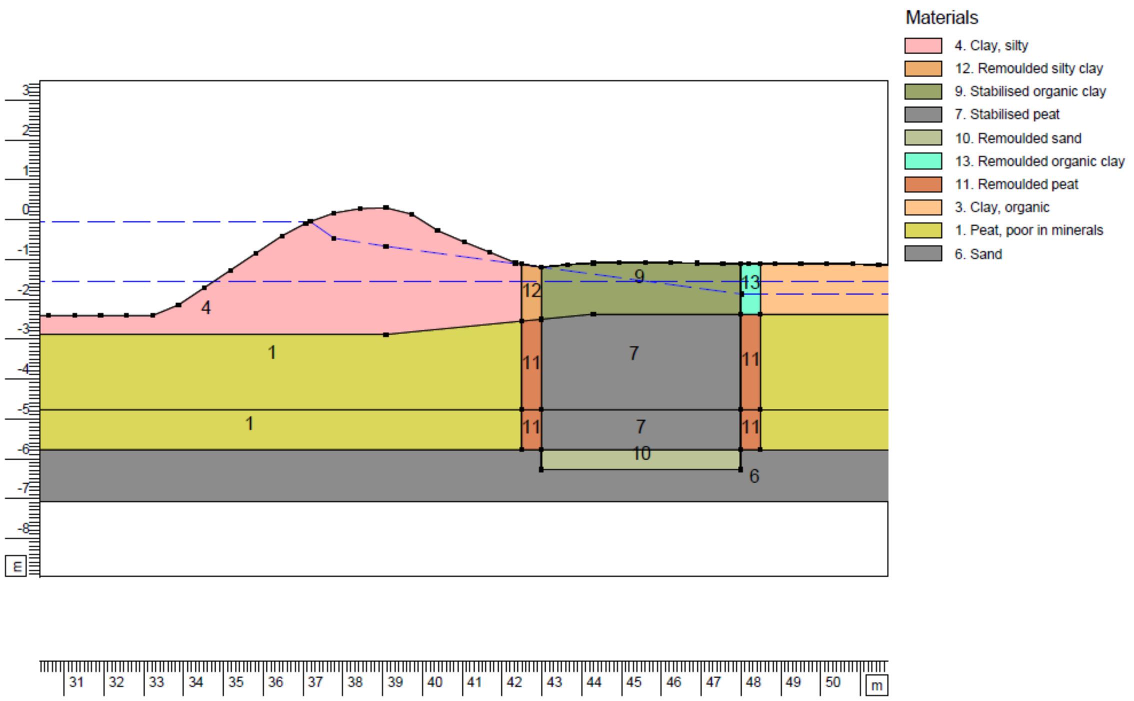

Figure C.1; Schematisation of the 5,0 metre wide and about 4,5 metre deep stabilised soil block at the toe of the levee at the Montfoortse Vaart.

Table C.1; The calculated current and the required Factors of Safety for the levee at the Montfoortse Vaart.

D-GeoStability model	Current Factor of Safety	Required Factor of Safety
Bishop	0,92	1,02
Uplift Van	0,91	1,07

In order to determine the minimum design values of the effective strength of both the stabilised organic clay and the stabilised peat, multiple stability calculations were carried out using both the Bishop and Uplift Van calculation model. The soil parameters that were used in these stability analyses are presented in table c.2. The soil parameters of the undisturbed soils were obtained from a database on undisturbed soil properties for soils in the region of Water Board *Hoogheemraadschap De Stichtse Rijnlanden* (HDSR) (Bardoel & Eshuis, 2017) as shown in appendix B, whereas for the remoulded soils 40% of the undisturbed soil strength was assumed on both the (c') and the tangent of the angle of internal friction ($\tan(\phi')$) based on the Dutch technical guideline for macro-stability (Zwanenburg, van Duinen, & Rozing, 2013).

In the stability calculations, a variation in the effective strength parameters was modelled:

- The design value of the effective cohesion was increased in steps of 1,0 kPa from 1,0 kPa to the maximum considered value of 5,0 kPa;
- The design value of the effective angle of internal friction was increased in steps of 5° from 15° to the maximum considered value of 35°.

Since the stability analyses from the main report with an effective cohesion of 5,0 kPa and an effective angle of internal friction of 35° of both stabilised soils resulted in a Factor of Safety exceeding the required Factor of Safety (see section 3.3), smaller values of both parameters were considered in these calculations. Additionally, a homogenous stabilisation was assumed, meaning that the strength of both the stabilised peat and the stabilised organic clay was kept equal in all stability calculations.

Furthermore, no changes in the unit weight upon stabilisation were assumed. From the main report it was shown that increases in the unit weight of the soils at the toe result in increases in the Factor of Safety (see section 3.3). However, since in the main report it was also mentioned that increases in the unit weight of the soils due to stabilisation can also be negligibly small (see section 2.4.2), it was decided that the strength of the stabilised soils should be sufficient enough that the levee can be reinforced without having to rely on increases in the unit weight to reach the required Factor of Safety against inward macro-instability.

The results from the stability analyses as carried out with the Bishop and Uplift Van calculation model are presented in respectively table c.3 and table c.4.

Table C.2; The applied design values of the soil parameters for the stability analyses of the levee at the Montfoortse Vaart.

Soil type (Montfoortse Vaart)	Wet unit weight	Saturated unit weight	Effective cohesion	Effective angle of internal friction
	$\gamma_{wet;d}$	$\gamma_{sat;d}$	c'_d	ϕ'_d
	[kN/m³]	[kN/m³]	[kPa]	[°]
Peat, poor in minerals	10,00	10,00	0,67	12,7
Organic clay	12,80	12,80	0,83	31,1
Silty clay	15,95	15,95	2,01	27,3
Sand	19,00	21,00	0,00	29,0
Remoulded peat	10,00	10,00	0,27	5,2
Remoulded organic clay	12,80	12,80	0,33	13,6
Remoulded silty clay	15,95	15,95	0,80	11,7
Remoulded sand	17,00	19,00	0,00	26,7
Stabilised peat	10,00	10,00	1,00 - 5,00	15,0 - 35,0
Stabilised organic clay	12,80	12,80	1,00 - 5,00	15,0 - 35,0
Stabilised silty clay	15,95	15,95	1,00 - 5,00	15,0 - 35,0

In order to determine which combinations of the effective strength parameters of the stabilised soils result in a Factor of Safety that meets the required Factors of Safety from table c.1, the results from table c.3 and table c.4 were combined. The combinations of the effective strength parameters of the stabilised soils at which the required Factor of Safety of both calculation models is reached are considered the minimum design values required. After combining the results, the four combinations of the effective strength parameters as presented in table c.5 were obtained.

Table C.3; The Factor of Safety for the different combinations of the effective strength parameters as calculated with the Bishop calculation model. Values in green indicate the required Factor of Safety for the Bishop calculation model is met, whereas values in red indicate the required Factor of Safety is not met.

Effective cohesion (c'_d) [kPa]	Effective angle of internal friction (ϕ'_d) [°]				
	Peat: 15,0 Org. clay: 15,0	Peat: 20,0 Org. clay: 20,0	Peat: 25,0 Org. clay: 25,0	Peat: 30,0 Org. clay: 30,0	Peat: 35,0 Org. clay: 35,0
Peat: 1,0 Org. clay: 1,0	0,89	0,98	1,06	1,13	1,19
Peat: 2,0 Org. clay: 2,0	0,99	1,07	1,15	1,21	1,27
Peat: 3,0 Org. clay: 3,0	1,09	1,17	1,23	1,29	1,30
Peat: 4,0 Org. clay: 4,0	1,18	1,25	1,30	1,30	1,30
Peat: 5,0 Org. clay: 5,0	1,26	1,30	1,30	1,30	1,30

Table C.4; The Factor of Safety for the different combinations of the effective strength parameters as calculated with the Uplift Van calculation model. Values in green indicate the required Factor of Safety for the Uplift Van calculation model is met, whereas values in red indicate the required Factor of Safety is not met.

Effective cohesion (c'_d) [kPa]	Effective angle of internal friction (ϕ'_d) [°]				
	Peat: 15,0 Org. clay: 15,0	Peat: 20,0 Org. clay: 20,0	Peat: 25,0 Org. clay: 25,0	Peat: 30,0 Org. clay: 30,0	Peat: 35,0 Org. clay: 35,0
Peat: 1,0 Org. clay: 1,0	0,83	0,92	1,00	1,08	1,10
Peat: 2,0 Org. clay: 2,0	0,94	1,02	1,10	1,10	1,10
Peat: 3,0 Org. clay: 3,0	1,04	1,10	1,10	1,10	1,10
Peat: 4,0 Org. clay: 4,0	1,10	1,10	1,10	1,10	1,10
Peat: 5,0 Org. clay: 5,0	1,10	1,10	1,10	1,10	1,10

Table C.5; Options with the required design values for the effective cohesion and the effective angle of internal friction of the stabilised soil layers at the toe of the levee at the Montfoortse Vaart.

Montfoortse Vaart	Option 1		Option 2		Option 3		Option 4	
	c'_d [kPa]	ϕ'_d [°]	c'_d [kPa]	ϕ'_d [°]	c'_d [kPa]	ϕ'_d [°]	c'_d [kPa]	ϕ'_d [°]
Stabilised peat	≥ 1,0	≥ 30,0	≥ 2,0	≥ 25,0	≥ 3,0	≥ 20,0	≥ 4,0	≥ 15,0
Stabilised organic clay	≥ 1,0	≥ 30,0	≥ 2,0	≥ 25,0	≥ 3,0	≥ 20,0	≥ 4,0	≥ 15,0

C.1.2 Determination required mean values of effective strength parameters

The obtained minimum design values of the effective strength parameters are required to be converted to mean values, as in the laboratory only mean values are recorded. So in order to determine whether the recorded mean values from the laboratory tests are sufficient, it is useful to know what the minimum required effective strength parameters as determined from the stability analyses are.

In order to convert the design values of the effective strength parameters to mean values, a coefficient of variation and a partial material factor is required for each of the two effective strength parameters. In this research, the coefficients of variation and the partial material factors from table c.6 were applied.

Table C.6; The applied coefficients of variation and partial material factors for the cohesion and the tangent of the internal friction for the stabilised soil layers at the toe of the levee at the Montfoortse Vaart.

Soil type	$COV_{c'}$	$COV_{tan(\phi')}$	$\gamma_{mat;c'}$	$\gamma_{mat;tan(\phi')}$
	[-]	[-]	[-]	[-]
Peat	0,4	0,03	1,5	1,2
Organic clay	0,4	0,09	1,5	1,2

For this case, the coefficient of variation of the effective cohesion was put equal to 0,4 for both stabilised soils. Although no indications on the coefficient of variation of the effective cohesion for stabilised soil was found during the literature study, it was found that the variability of the strength after stabilisation became two times larger than the variability of the strength of the undisturbed soils (see section 2.4.1.4). Even though the coefficients of variation of the effective cohesion of the undisturbed soils was known from a database maintained by Water Board HDSR (Bardoel & Eshuis, 2017), the values could not be used as they either exceeded 3,0 or were negative. Instead, it was decided to apply a value of twice the coefficient of variation of the effective cohesion of 0,2 as found in table 2.b from Dutch standard NEN 9997-1 (i.e. Eurocode 7 + Dutch national appendix) (Normcommissie 351 006 "Geotechniek", 2017). The factor two was applied to include the increased variability of the strength of the stabilised soils compared to undisturbed soils.

Additionally, it is expected that most of the variation of the drained strength will be in the effective cohesion and to a much lesser extent in the effective angle of internal friction. After all, if very large values of the coefficient of variation on the tangent of the effective angle of internal friction ($\tan(\phi')$) would be realised, then this would imply that it is possible to achieve very large angles of internal friction, which was not considered realistic. As such, the coefficient of variation on the tangent of the effective angle of internal friction for the stabilised soils was assumed equal to the coefficient of variation on the tangent of the effective angle of internal friction of the undisturbed soils. These values were obtained from the database on soil parameters of soils that were found in the management region of Water Board HDSR and were determined for the effective strength parameters evaluated at either 2% axial or 5% shear strain.

Furthermore, partial material factors for the effective strength parameters were chosen from Module C of the Dutch STOWA guideline for the assessment of the safety of regional flood defences (Stichting Toegepast Onderzoek Waterbeheer, 2015). These factors were selected rather high to include additional safety, since suitable values of the partial material factors were not known for mass stabilised soils.

Using these values of the partial material factors and the coefficients of variation, the design values of the effective strength parameters from table c.6 were converted to mean values using equation (C-1). Equation (C-1) assumes a normal distribution of the effective strength parameters with a t-student factor of 1,64. After conversion, the mean values of the effective strength parameters for each of the four options as presented in table c.7 are obtained.

$$X_d = \frac{X_m - 1{,}64 \cdot (COV_X \cdot X_m)}{\gamma_{mat(X)}} \quad \textbf{(C-1)}$$

where:

X_d	*- design value of soil parameter*	*[any]*
X_m	*- mean (measured) value of soil parameter*	*[any]*
COV_X	*- coefficient of variation of soil parameter*	*[-]*
$\gamma_{mat(X)}$	*- partial material factor for soil parameter*	*[-]*

Table C.7; The required mean values for the cohesion and the angle of internal friction of the stabilised soil layers at the toe of the levee at the Montfoortse Vaart for the four different options.

Soil parameters	Option 1		Option 2		Option 3		Option 4	
	Peat	Organic clay	Peat	Organic clay	Peat	Organic clay	Peat	Organic clay
c' **[kPa]**	≥ 4,4	≥ 4,4	≥ 8,7	≥ 8,7	≥ 13,1	≥ 13,1	≥ 17,4	≥ 17,4
ϕ' **[°]**	≥ 36,1	≥ 39,1	≥ 30,5	≥ 33,3	≥ 24,7	≥ 27,1	≥ 18,7	≥ 20,7

C.1.3 Determination of minimum unconfined compressive strength

For each of the options listed in table c.7, an indicative minimum unconfined compressive strength was derived using equation (C-2). The equation correlates the effective strength parameters determining the Mohr-Coulomb failure line to the major principle effective stress that can be acted on a sample without lateral effective stress. It should be noted though that the major principle effective stress here is assumed to be equal to the unconfined compressive strength. However, in actuality this is not true. The unconfined compressive strength is a strength parameter defined in terms of total stress, whereas the effective cohesion and angle of internal friction are defined in terms of effective stress. As a result, the effective strength parameters cannot be correlated to the unconfined compressive strength. Since no correlations between the unconfined compressive strength and the effective strength parameters (at specified strains) for stabilised soils were found, this assumption was made to obtain an indicative value of the unconfined compressive strength.

$$UCS = 2 \cdot c' \cdot \tan\left(45 + \frac{\phi'}{2}\right) \qquad \textbf{(C-2)}$$

where:

UCS	*- unconfined compressive strength*	*[kPa]*
c'	*- effective cohesion*	*[kPa]*
ϕ'	*- effective angle of internal friction*	*[°]*

Using equation (C-2), the unconfined compressive strength corresponding to the effective strength parameters from each of the four options shown in table c.7 was determined. These unconfined compressive strengths are presented in table c.8. From table c.8 can be seen that for each option different values of the unconfined compressive strength were obtained, mostly as a result of the increasing effective cohesion with each option. Since it is not known how the effective parameters of the soils will change due to stabilisation, it is important to ensure that regardless of the change in the effective strength parameters any of the four options will still be met. In order to do so, it was decided that both the stabilised peat and the stabilised organic clay should have an unconfined compressive strength of at least 50 kPa after curing has been completed. As a result, this value of the unconfined compressive strength was considered the minimum mean unconfined compressive strength that any stabilised soil produced in the laboratory should at least have after complete curing.

Table C.8; The required mean values for the cohesion, the angle of internal friction and the unconfined compressive strength (UCS) of the stabilised soil layers at the toe of the levee at the Montfoortse Vaart for the four different options.

Soil parameters	Option 1		Option 2		Option 3		Option 4	
	Peat	Organic clay	Peat	Organic clay	Peat	Organic clay	Peat	Organic clay
c' [kPa]	≥ 4,4	≥ 4,4	≥ 8,7	≥ 8,7	≥ 13,1	≥ 13,1	≥ 17,4	≥ 17,4
ϕ' [°]	≥ 36,1	≥ 39,1	≥ 30,5	≥ 33,3	≥ 24,7	≥ 27,1	≥ 18,7	≥ 20,7
UCS [kPa]	≥ 17	≥ 19	≥ 30	≥ 32	≥ 41	≥ 43	≥ 49	≥ 50

List of references

Bardoel, J., & Eshuis, F. (2017). *Proevenverzameling HDSR - Regionale waterkeringen.* Capelle aan den IJssel: Antea Group.

Normcommissie 351 006 "Geotechniek". (2017). *NEN 9997-1+C2 - Geotechnisch ontwerp van constructies - Deel 1: Algemene regels.* Delft: Nederlandse normalisatie-instituut.

Stichting Toegepast Onderzoek Waterbeheer. (2015). *Leidraad toetsen op veiligheid regionale waterkeringen - module C: sterkte.* Amersfoort: Stichting Toegepast Onderzoek Waterbeheer.

Zwanenburg, C., van Duinen, A., & Rozing, A. (2013). *Technisch Rapport Macrostabiliteit.* Delft: Deltares.

Appendix D - Laboratory soil stabilisation procedure

D.1 Sources used

The laboratory soil stabilisation procedure that was adhered to during the laboratory work was compiled by the author by collecting laboratory soil stabilisation procedures from various sources and combining them into a single procedure. Most of the sources provide a general stabilisation procedure, whereas some provide a more detailed stabilisation procedure.

The compiled laboratory stabilisation procedure that was produced is based on the following sources:

- EuroSoilStab Design Guide Soft Soil Stabilisation (2002):
    - Laboratory procedure for test samples (mass stabilisation applications);
- Dutch CUR 2001-10 *'Diepe grondstabilisatie in Nederland'* (2001):
    - *'Vervaardiging van met bindmiddelen gestabiliseerde grondmonsters'* (EN: Preparation of soil samples stabilised with binders);
- Dutch CUR 199 *'Handreiking toepassing No-Recess technieken'* (2001):
    - *'Laboratorium stabilisatie procedure – vervaardiging van met bindmiddelen gestabiliseerde grondmonsters (voor gestabiliseerde grondkolommen)'* (EN: Laboratory stabilisation procedure – preparation of samples stabilised with binders (for stabilised soil columns));
- The Deep Mixing Method – Principle, Design and Construction (2002):
    - Summary of the practice for making and curing stabilised soil specimens without compaction;
- United States of America Federal Highway Administration Design Manual (2013):
    - Laboratory procedure for mixing, curing and strength testing of treated soil specimens applicable to wet mixing;
- Fugro NL Land B.V. laboratory mass stabilisation manual (2018).

The laboratory stabilisation procedure was compiled with an emphasis on the Fugro NL Land B.V. mass stabilisation manual, because the laboratory work was carried out in the geotechnical laboratory of Fugro NL Land B.V. using the mass stabilisation test setup as developed by Fugro NL Land B.V..

D.2 Scope of the laboratory soil stabilisation procedure

In this procedure, the steps to be taken for the preparation of samples of soil stabilised in the laboratory for mass stabilisation applications is specified. This procedure is focussed on the stabilisation of soft soils, such as (organic) clay and peat. The binder material used for the stabilisation is case specific and is usually composed of cement or of a blend of cement and additives. Additives that are commonly used include ground granulated blast-furnace slag, gypsum, lime and/or fly ash. The stabilised soil samples prepared in the laboratory serve for research into the properties of the stabilised soil (over time). Properties of the stabilised soil may be determined by means of soil index tests, unconfined compression tests, triaxial tests, shearbox tests, direct simple shear tests, oedometer tests and/or permeability tests.

For other soil stabilisation applications, such as deep mixing, mixed-in-place or column stabilisation, different procedures may be required for the preparation of stabilised soil samples in the laboratory. In such an event it is advised to consult other relevant documents for the preparation of stabilised soil samples for the other soil stabilisation applications.

This laboratory soil stabilisation procedure was compiled in combination with:

- A form that has to be filled in during the laboratory soil stabilisation procedure;
- An excel file that is used to calculate the masses of the soil-binder mixture components that are required to obtain a soil stabilised with a certain quantity of binder material.

The form for the laboratory soil stabilisation procedure is presented in section D.7. The excel file on the other hand could not be presented in this report. Therefore, the steps required to calculate the masses of the soil-binder mixture components are presented instead.

D.3 General overview of laboratory soil stabilisation

The stabilisation of the soil samples from this research was carried out in the laboratory of Fugro NL Land B.V. in Arnhem (Netherlands). The soil stabilisation procedure was carried out using the mass stabilisation test setup as produced by Fugro NL Land B.V.. A photograph of the mass stabilisation test setup taken by the author is shown in figure d.1.

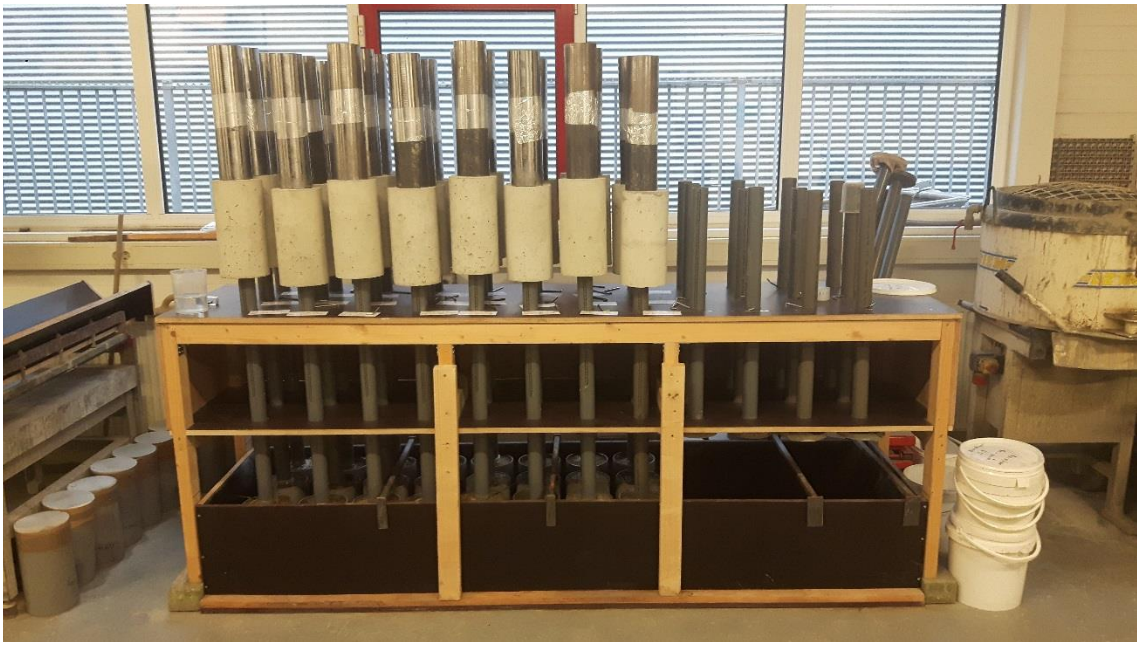

Figure D.1; The mass stabilisation test setup built by Fugro NL Land B.V. used during this research.

The laboratory soil stabilisation procedure can generally be divided into 7 consecutive steps as shown in figure d.2. The next subsections provide descriptions of the actions to take during each of these 7 steps.

Figure D.2; Flowchart showing the process of the laboratory soil stabilisation procedure.

D.3.1 Mixture component preparation

Before the laboratory soil stabilisation can start, a mixture has to be devised which has to be tested. This mixture typically consists of soil and binder, but sometimes it is also required to add water to the mixture. These three components are to be mixed in a specific mass ratio in order to create the desired mixture. When this mass ratio has been determined, the required amounts of soil, binder and possibly water can be weighed off. The amounts that should be weighed off should be large enough so that the required number of moulds can be filled with the mixture.

D.3.2 Mixing components

When the required amounts of soil, binder and possibly water had been weighed off, they are to be mixed using a mixer. In this research, a Varimixer BEAR type 94/AR60 as shown in figure d.3 was used for the mixing of the components.

First of all the soil was put in the mixing bowl. If no water were to be added for this mixture, then the soil was premixed by the mixer first. This was done to create a more homogeneous soil before it was stabilised with a binder. If water were to be added for this mixture, then the water was gradually added while the mixer was mixing only the soil. Of the mixed soil a sample was taken to determine the water content of.

Figure D.3; The Varimixer type 94/AR60 as used in this research for the laboratory soil stabilisation.

Once the mixing of soil (and water) was completed, then the binder was gradually added while the mixer mixed both the soil and the binder. The time that was required to mix the soil with the water and to mix the water-soil mixture with the binder to a visually homogeneous mixture were recorded. Once the mixing of the soil and the binder was completed, the mixture would be transported to the mass stabilisation setup where the moulds would be filled with the mixture.

D.3.3 Filling of moulds

With the mixture prepared, the moulds can be filled with this mixture to make stabilised soil samples. In this research PVC tubes were used as moulds which were closed off at the bottom end by a sock as shown in figure d.4. The PVC tubes were 150 mm in diameter and approximately 300 mm high. The sock on the mould was used to prevent the mixture from falling out of the mould when the mould was placed in the mass stabilisation setup. Also it allowed for water to pass through in order to keep the mixture wet.

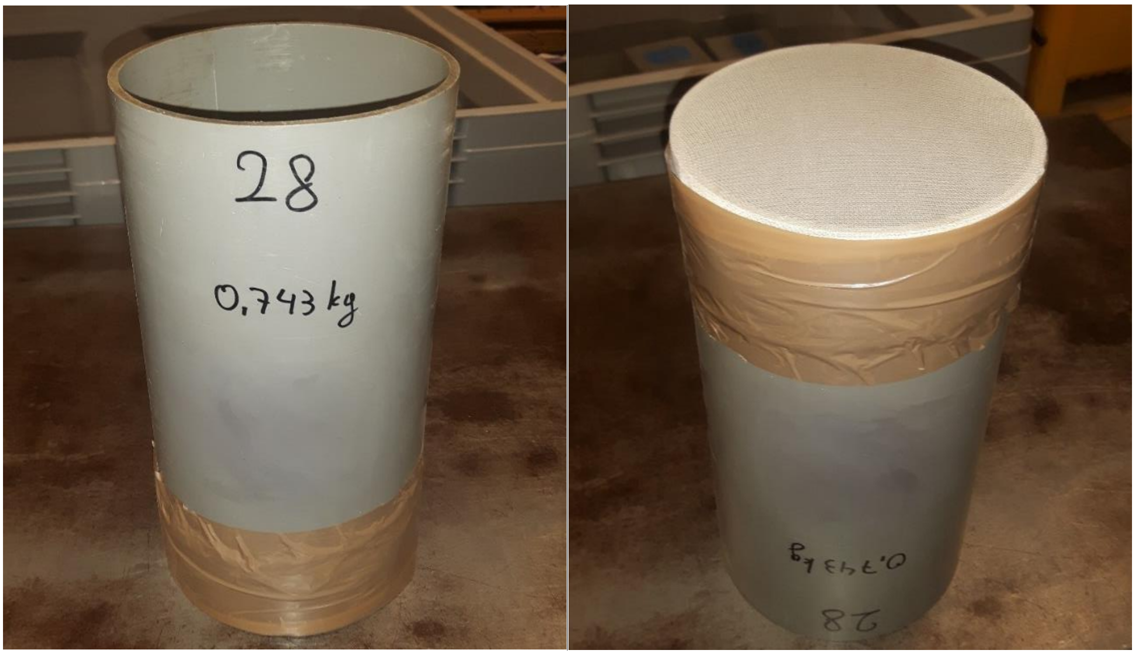

Figure D.4; The moulds as used in this research for the laboratory soil stabilisation.

The moulds were filled stepwise by applying 5 equally large layers of the mixture and then compacting them using a metal masher as shown in figure d.5. This was carried out by first weighing the empty moulds and lubricating the inner walls of the moulds. After this, a mass of the mixture is weighed that when compacted fills approximately 20% of the volume of the mould that is meant to be filled by the mixture. This mass of the mixture is then put in the mould, after which the mixture is equally spread over the entire inner diameter of the mould. The mixture can be compacted by applying a pressure with the fist. After this, the metal masher is placed in the mould on top of the mixture.

The metal masher was specifically designed for the compaction of these mixtures in the moulds. The metal masher has a diameter slightly less than 150 mm and has 5 so-called 'layer lines' engraved at the perimeter of the masher with an equal distance between the layer lines. These lines are used to determine whether the compacted mixture has filled approximately 20% of the volume of the mould that is meant to be filled by the mixture. Aside from this, the metal masher has two slots opposite of each other along the entire height of the masher. These slots were made to allow air to escape from the mixture upon compaction. If these slots were not made, compacting the mixture using the masher would cause a vacuum to be created. Once compaction was complete and the metal masher was attempted to be removed, it would cause the compacted mixture to be pulled out of the mould with the metal masher. However, with these slots in place this will be prevented.

Figure D.5; The metal masher that was used for the compaction of the mixtures in the moulds.

Once the metal masher is placed on the layer of mixture in the mould, the user's own weight would be used on the masher to apply a brief load on the layer of mixture causing it to compact. Subsequently, a recoil-free hammer was used to apply 10 hammer blows on top of the masher to cause the remaining air to escape the mixture.

After compaction, the metal masher is checked to see whether the layer line lines up with the end of the mould. If the layer line is (far) above the upper end of the mould, some of the mixture should be removed from the mould and the remaining mixture should be re-compacted. If the layer line is (far) below the upper end of the mould, then more mixture should be added to the mould to reach the required thickness of the layer. This mixture should then also be re-compacted. Once the layer line lines up with the upper end of the mould, a compacted layer of mixture of the right thickness has been created.

Before this process is repeated for the remaining layers of the mixture, the upper end of the compacted layer should be roughened up first. This was done by scratching the surface of the mixture using a fork. Roughening up the surface allows for a better binding of the different layers of the mixture. The effect of roughening up the surface on the binding of the different layers is presented in figure d.6.

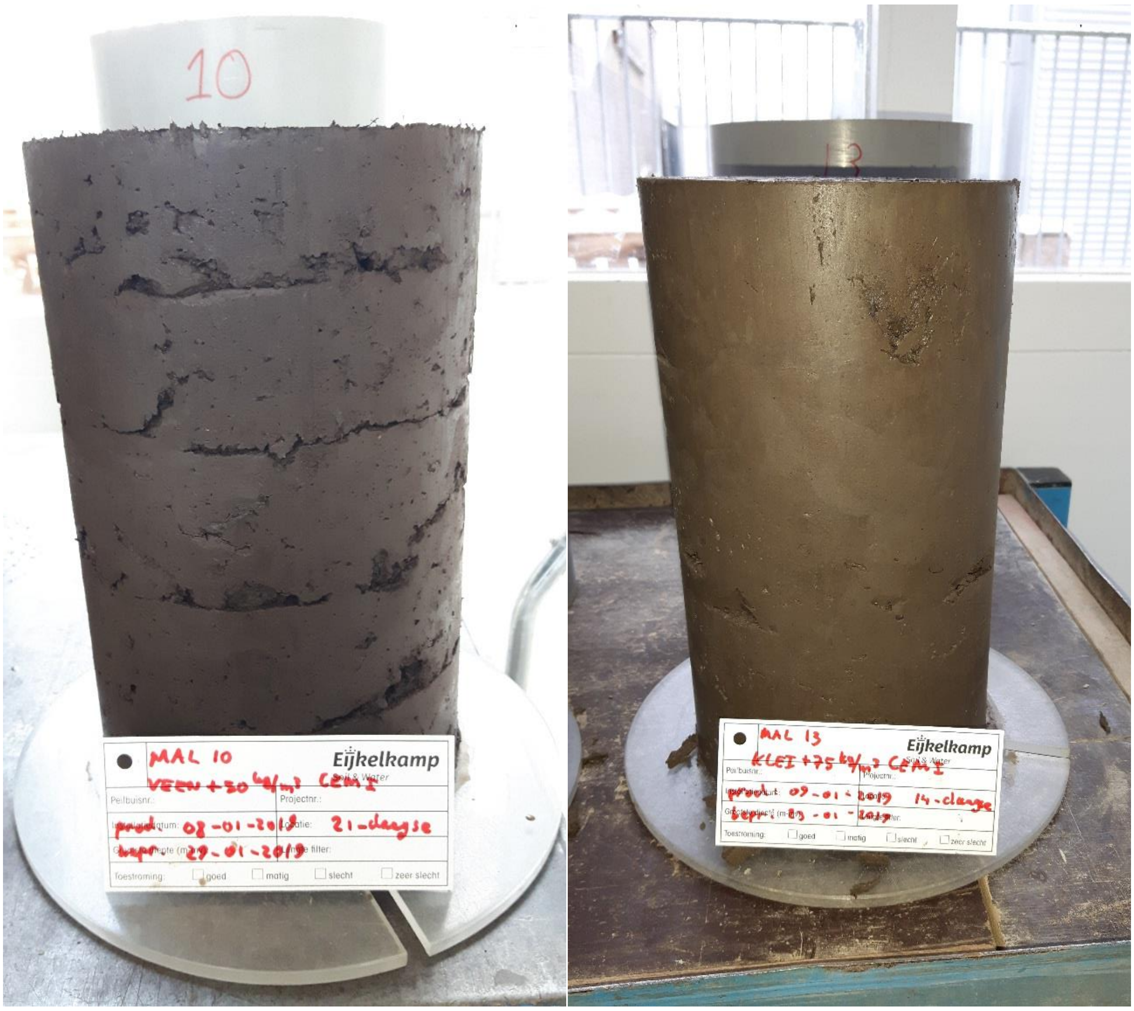

Figure D.6; The effect of roughening up the surface of each layer of mixture. The left image shows a badly-made sample since all the layers are visible. The right image shows a very well-made sample since the layers are hardly visible.

Once the mould is completely filled with 5 equally large layers of the mixture, the upper surface of the mixture is smoothened out using a small PVC masher similar to the masher shown in figure d.8. By wetting and subsequently placing the masher on top of the sample and rotating the masher slowly, any visible holes caused by trapped air can be removed. Once the upper end surface is smooth, the mixture in the mould is ready. The entire mould-filling process is repeated until the required amount of moulds are filled with the mixture.

D.3.4 Placing filled moulds in mass stabilisation setup

When the moulds were filled with the mixture, all moulds were weighed again and the masses of the moulds with the compacted mixtures was recorded. After this, the moulds were placed in the container of the mass stabilisation setup as shown in figure d.7. This container was filled with a small layer of water of about 2 cm to prevent the lower end of the mixture in the mould from drying out.

When the moulds had been placed in the mass stabilisation setup, the PVC mashers as shown in figure d.8 were lowered and placed on top of the mixtures in the moulds. On top of these mashers a small layer of water was applied to keep the upper end of the mixture wet. On these PVC mashers, a tape was pasted that acts as a measuring tape. This tape was read off when the mashers were placed on the mixture. This measurement is used as a zero measurement for the compression of the mixture upon loading.

Figure D.7; The container of the mass stabilisation setup filled with a few moulds.

D.3.5 Loading of mixtures

With the mashers placed on the mixtures in the moulds, weights were placed on top of the mashers to simulate the in-situ stresses. The weights that were used consist of a combination of steel rods, hollow concrete cylinders, partially hollow metal cylinders and metal pins to connect the concrete and metal cylinders. The order of placing the weights is important, as the steel rods fit in the long PVC tubes that make up the PVC mashers from figure d.8. The steel rods themselves have a pin extending from the top end allowing a concrete or metal cylinder to be put on top. If both the concrete and the metal cylinders are required to be put on top of the steel rod, the concrete cylinder will be placed first. Then a metal pin is placed in the cavity of the concrete cylinder, allowing the metal weight to be put on top of the concrete cylinder without risking it falling or sliding off the concrete cylinder. If all four weight objects were used, then one ends up in a situation as shown in figure d.1.

Figure D.8; The plastic mashers that are part of the mass stabilisation setup. These mashers have a measuring tape attached to measure the compression of the mixtures as indicated in the right part of the image.

After the mixtures were loaded with the required amount of weight, the measuring tape on the PVC mashers was read off. This was done to record the total compression of the mixture due to the applied load. The compression of the mixtures was measured at 5, 30 and 60 minutes after loading and after 1, 2, 7, 14 and 28 days after loading. In the event a sample was not to be left to cure for 28 days, the compression would only be recorded at these intervals up until the time the sample is to be removed from the mould. The final measurement of the compression would then occur right before the load is to be removed.

D.3.6 Removal of mould from mass stabilisation setup

Once the sample had cured under the load for a predetermined amount of time, the measuring tape on the PVC would be read off after which the load was removed. Once the load was removed, the PVC masher was lifted and the moulds with the compressed mixtures were removed from the container of the mass stabilisation setup. The exterior of the moulds was subsequently dried and the mass of the moulds with the compressed mixtures was recorded. The moulds were then taken towards the extrusion machine to extrude the samples from the moulds.

D.3.7 Sample extrusion

Once the samples were ready for extrusion, the socks were removed from the moulds to allow for sample extrusion. After this, the moulds were put in the holder of the extrusion machine as shown in figure d.9. A metal plate with a diameter slightly less than 150 mm was placed in the mould. This metal plate is required because the pressure plate of the extrusion machine has a diameter that is too small to push the sample out of the mould without damaging the sample. Furthermore, any disturbance of the stabilised soil sample during extrusion was minimised because the inner walls of the moulds were lubricated prior to filling.

Figure D.9; The extrusion machine that was used to extrude the stabilised soil samples from the moulds.

The extruded samples were weighed and the diameter and height of the samples were measured. This allowed for the determination of the unit weight after compression. The unit weight of the mixture prior to loading could be derived by combining the information of the weighed filled moulds prior to loading with the measured compression due to loading.

After these measurements, the extruded samples were inspected on visual irregularities. Once the inspection was finished, the stabilised soil samples were ready to be subjected to unconfined compression tests.

D.4 Extrusion of smaller stabilised soil samples

For the extrusion of stabilised soil samples with sizes smaller than the standard size of 150 mm diameter and a height of 300 mm, a method different from the method described in section D.3.7 was applied. In this research, smaller stabilised soil samples were required for triaxial tests, shearbox tests and oedometer tests. The applied method of extrusion of smaller samples was different for triaxial tests and for shearbox and oedometer tests. These methods are presented in the next subsections.

D.4.1 Triaxial tests

Instead of immediately pushing out the sample from the mould, three lubricated smaller 50 mm diameter tubes were placed on top of the sample first. Subsequently, the three tubes were pushed in the sample using a compression machine while the sample itself was still in the mould. This is shown in figure d.10. To prevent the cutting edge of the tubes to be damaged upon touching the table on which the mould was resting, the tubes were allowed to be pressed in the sample up to about 90% of the height of the sample. After the tubes were pushed in the sample, the mould with the sample and the tubes was put in the extrusion machine (figure d.9). Then the sample with the tubes in it was extruded from the mould. An example of such an extruded sample is also shown in figure d.10. The resulting material outside the three tubes was cut off.

Figure D.10; The pushing of three small 50 mm diameter tubes into the stabilised soil sample in the mould using a compression machine (left) and the resulted extruded sample with the smaller tubes pushed in the sample (right) (Koenders, 2018).

The stabilised soil in the three tubes was subsequently extruded using a different setup of the extrusion machine shown in figure d.9. The result was three stabilised soil samples of 50 mm in diameter, an example of which is shown in figure d.11. These samples may then be trimmed to the desired height, which for triaxial tests usually equals 100 mm.

Figure D.11; Extruded stabilised soil samples of 50 mm in diameter.

It should be noted that in this research 50 mm diameter samples were required for the triaxial tests that were carried out on the stabilised soil samples. Other sample diameters such as 67 mm diameter were also possible to extrude from the large stabilised sample using Ackermann tubes. However, then it would only have been possible to extrude one sample due to the large diameter of the tube compared to the diameter of the mould.

Additionally it was not desired to push the smaller tubes into the sample too close to the wall of the mould. As seen in figure d.6, it is still possible that air gaps were present at the outer perimeter of the stabilised soil sample. To prevent extruding smaller samples containing air gaps, it should be prevented to push the tubes in the sample too close to the wall of the mould.

D.4.2 Shearbox and oedometer tests

For shearbox and oedometer tests samples of similar diameters were required. However, for the shearbox and the oedometer test it is not desirable to prepare smaller stabilised soil samples using the method described in section D.4.1. For the shearbox and the oedometer test, the samples need to be prepared at exact dimensions using cylindrical cutters calibrated for these setups to prevent (large) gaps between the sample and the wall of the ring in which the samples are to be put. If a vertical load would be applied when there is a gap between the sample and the wall, the sample will compress vertically and extend in the lateral direction. This will result in too large vertical deformations during consolidation and will yield incorrect values of the stiffness parameters.

In order to combat this problem, the stabilised soil sample was extruded from the mould in its entirety. Subsequently, the sample was cut in multiple slices at the visible layer lines of the sample (see figure d.6), yielding approximately 5 equally thick slices. From these slices, samples at exact dimensions were taken using the calibrated cylindrical cutters for the specific test setups.

D.5 Compatibility of laboratory stabilisation procedure with practice

The applied laboratory soil stabilisation procedure for stabilisation of soils for mass stabilisation applications is not completely compatible with actual field stabilisations for a number of reasons:

- Premixing of the soil as done in the laboratory is not always carried out in the field, unless it would otherwise hinder the stabilisation in the field;
- The mixer used in the geotechnical laboratory of Fugro NL Land B.V. is not representative of an actual 'mixer' used in the field for field stabilisations;
- The dimensions of the stabilised soil samples are much smaller than the dimensions of field stabilised soil samples. This may result in differences in curing behaviour between laboratory and field stabilised soil samples;
- Soil samples stabilised in the laboratory have access to free-standing water during curing when placed in the container of the mass stabilisation setup. Soil samples stabilised in the field do not have this and are dependent on the permeability of the surrounding soil for access to water;
- Visual observation of the homogeneity of the soil during mixing in the laboratory allows for achieving rather homogenous stabilised soil samples. This is not possible in the field when soils at depth are mixed in-situ.

Premixing of soils in the field is not normally carried out unless very stiff or very dry soils require stabilisation (Forsman, Jyrävä, Lahtinen, Niemelin, & Hyvönen, 2015). In the event of very stiff soil it may benefit the stabilisation if the very stiff soils are loosened first by mixing the soil prior to the stabilisation. In the event of very dry soils it will benefit the stabilisation if the soil is mixed with water first, since the binder will not cause cementation in the absence of water. However, in all other cases premixing of the soil will usually not be carried out, while premixing of the soil in the laboratory is always recommended for more representative measurements of the water content of the homogeneously mixed soil. However, it is unknown to what extent the premixing of the soil in the laboratory will affect the properties of the stabilised soil compared to field stabilisations without premixing. After all, the degree of homogeneity of the stabilised soil is expected to depend mostly, if not solely, on the efficiency of the mixing of the soil with the binder and to a lesser extent, if at all, on the degree of homogeneity of the untreated soil prior to stabilisation.

The mixer used in the geotechnical laboratory of Fugro NL Land B.V. is not representative of the mass stabilisation mixing equipment as used in field stabilisations. The differences are found in both the build-up of the mixers and the power with which is mixed. It is therefore not possible to simulate the field mixing in the laboratory with small mixers. This is a disadvantage of stabilising soils in the laboratory. This does not necessarily have to be a problem though, since the objective in both the field and the laboratory is to obtain stabilised soil samples that are as homogeneous as possible. However, the results may still be different as one method may be able to better mix the soil and the binder to a homogeneous mass than another.

The dimensions of the stabilised soil samples right after filling the moulds are approximately 280 mm in height and 150 mm in diameter (i.e. about 5 L), whereas the dimensions of volumes of soil stabilised in one go in the field could be as much as 5,0 by 5,0 by 5,0 metres (Allu Finland Oy, 2007). Due to the mismatch in dimensions it may be possible for the curing behaviour (i.e. strength and stiffness development in time) to be different for the laboratory stabilised soil samples than for soils stabilised in the field, possibly leading to unrepresentative results.

Another difference between the curing conditions of laboratory and field stabilised soil samples is the access to water during water. The stabilised soil samples have access to free-standing water on the top and bottom when placed in the container of the mass stabilisation setup. Stabilised soils in the field typically do not have access to free-standing water, but are instead dependent on the permeability of the surrounding soils for their access to water. Additionally, stabilised soils in the field could have access to water on all sides during curing, whereas the stabilised soil samples in the laboratory only have access to water on the top and the bottom. However, it is currently unknown how this difference between the laboratory and the field may cause differences in the quality and properties of soils stabilised in the laboratory and soils stabilised in the field.

Lastly, it is possible to visually observe whether the mixture is homogeneous or not during mixing in the laboratory, allowing one to better control the stabilisation. Visual observation of the homogeneity of the mixed soil at depth during stabilisation is not possible. Therefore it is not possible in the field to visually assess whether the soil is well-mixed or not. This is why the mixing equipment in the field have a special system that helps the contractor keep track how well each block is expectedly stabilised. Yet it is likely that the mixtures produced in the laboratory are much more homogeneous than in-situ mixed soils. This may yield significant differences in the properties of the stabilised soils and the variation therein when comparing laboratory and field test results.

Besides the aforementioned incompatibilities between the laboratory and the field stabilisation, there are also a couple of drawbacks of the applied laboratory soil stabilisation procedure:

- The degree of compaction of the mixtures after filling the moulds and the subsequent recorded compression of the mixtures in the moulds upon loading are dependent on the laboratory technician that produced the mixtures;
- The dimensions of the used moulds did not keep the compression of the mixtures in the moulds as a result of loading into account, often resulting in cured samples with height-to-diameter ratios (h/d) of less than 1,8.

Once the moulds were filled with a layer of mixture, the metal masher was used to compact the mixture in the mould in order to remove air present in the mixture. However, this required the laboratory technician to push on the metal masher. The amount of force applied on the metal masher is different per laboratory technician and may yield significant differences in the degree of compaction of the mixture. Additionally, the compression of the mixtures in the moulds after loading will be different as a result of the different degrees of compaction. If the laboratory technician applied a small force, more air will be in the mixture and more compression will be measured upon loading and vice versa.

Besides this, the moulds used for the preparation of the stabilised soil samples were of insufficient height. Once the moulds were filled with mixture, about 280 mm out of the 300 mm of the mould was filled with mixture. The remaining 20 mm would be taken up by the PVC masher. However, after compression the moulds were usually only filled with mixture up to 270 mm or less. The result is that the compressed mixtures mostly had a height-to-diameter ratio of less than 1,8 after extrusion from the mould. This was undesired, because the applied Dutch standard NEN-EN-ISO 17892-7 for unconfined compression tests require the height-to-diameter ratio of cylindrical soil specimens to be at least 1,8.

Both these drawbacks can be overcome when higher moulds are used and a single laboratory technician compacts the mixtures. Yet most of the incompatibilities between the laboratory stabilisation procedure and an actual field stabilisation cannot be overcome. Therefore the results obtained for stabilised soils produced in accordance with this laboratory soil stabilisation procedure only serve as an indication on the possible behaviour of the examined stabilised soils. As a result it always recommended to carry out trial stabilisations in the field prior to the actual stabilisation, as well as carrying out field tests during the actual stabilisation to observe the behaviour of the stabilised soils.

D.6 Step-by-step plan of the laboratory soil stabilisation procedure

The detailed step-by-step plan of the laboratory soil stabilisation procedure consists of 11 consecutive steps, one of which is optional. In this step-by-step plan, the following steps are distinguished:

- Phase 0 – General stabilisation information;
- Phase 1 – Measurements before stabilisation;
- Phase 2 – Homogenisation of the soil;
- Phase 3 – Preparation of the binder;
- Phase 4 – Mixing of the soil and binder;
- Phase 5 – Compaction of the mixture in the mould;
- Phase 6 – Storage and loading;
- Phase 7 – Removal of the stabilised soil sample from the mould;
- Phase 8 – Extrusion of smaller stabilised soil samples (OPTIONAL);
- Phase 9 – Preparation of sample ends (if applicable);
- Phase 10 – Geotechnical laboratory testing.

In the next subsections, the actions that need to be taken during each step are presented. In all steps direct references to the form to fill in during the laboratory stabilisation procedure as presented in section D.7 are made. It is advised to read through the next subsections with a separate copy of the form.

D.6.1 Phase 0 – General stabilisation information

Prior to starting the stabilisation procedure, it needs to be decided which soil-binder mixture is to be produced and at which date it is to be tested. This requires information on a number of soil index properties of the soil to be stabilised, such as the natural density, the natural water content and the particle density of the soil. Additionally, a decision needs to be made on the binder materials that will be used and in which quantities they will be applied.

In step 0, the next steps must be taken in sequence:

1. Fill in the table *"GENERAL INFORMATION"* on the form.
2. Determine which binder materials will be used for the stabilisation of the soil samples and fill in the table *"MATERIAL INFORMATION"*.
3. Determine the binder quantity of the binder components that will be used for stabilisation and fill in the *"MIXTURE INFORMATION"* table on the form. Should it be desired that the water content be increased to above the current water content of the soil to be mixed, for example because the soil used for mixing is drier than the undisturbed soil samples on which the mixtures are based, the following additional steps must be taken:
   3.1. Determine the required mass of the soil containing only naturally present water per cubic metre of soil with the required water content. This is done using the following equation:

$$\rho_{req} = \frac{\rho_{des}}{(100 + w_{des})} \cdot (100 + w_{nat})$$

where:

ρ_{req}	*- required mass of soil that, when mixed with a specific amount of water, results in a soil sample with the desired water content*	*[kg drier soil/m³ undisturbed soil]*
ρ_{des}	*- density of the soil at the desired water content (e.g. measured from undisturbed soil samples)*	*[kg soil/m³ undisturbed soil]*
w_{des}	*- desired water content of the soil (e.g. measured from undisturbed soil samples)*	*[%]*
w_{nat}	*- natural water content of the soil*	*[%]*

3.2. Determine the required mass of the water that needs to be added to the soil to reach the desired water content of the soil. This is done using the following equation:

$$\alpha_{water} = \rho_{des} - \rho_{req}$$

<u>*where:*</u>

α_{water}	*- required mass of water that, when mixed with a specific amount of drier soil, results in a soil sample with the desired water content*	*[kg water/m³ undisturbed soil]*
ρ_{req}	*- required mass of drier soil that, when mixed with a specific amount of water, results in a soil sample with the desired water content*	*[kg drier soil/m³ undisturbed soil]*
ρ_{des}	*- density of the soil at the desired water content (e.g. measured from undisturbed soil samples)*	*[kg soil/m³ undisturbed soil]*

4. Fill in the table *"DENSITY INFORMATION"* on the form. The mass of soil one cubic metre of undisturbed soil has in the field to which a specific amount of binder is added should be simulated in the laboratory to get the most representative mixture. Depending on whether the soil that is used for mixing is drier than desired, water should be added to create a sample with a certain water content representative of the field. Then the total mass of material that is required for the mixture is calculated using the following equation:

$$\rho_{stab} = \rho_{req} + \alpha_{binder} + \alpha_{water} = \rho_{des} + \alpha_{binder}$$

<u>*where:*</u>

ρ_{stab}	*- total mass of material in the mixture when one cubic metre of undisturbed soil is stabilised with a specific mass of binder*	*[kg material/m³ undisturbed soil]*
ρ_{req}	*- required mass of drier soil that, when mixed with a specific amount of water, results in a soil sample with the desired water content*	*[kg drier soil/m³ undisturbed soil]*
ρ_{des}	*- density of the soil at the desired water content (e.g. measured from undisturbed soil samples)*	*[kg soil/m³ undisturbed soil]*
α_{binder}	*- summation of the dosages of all binder components used for the mixture*	*[kg binder/m³ undisturbed soil]*
α_{water}	*- required mass of water that, when mixed with a specific amount of drier soil, results in a soil sample with the desired water content*	*[kg water/m³ undisturbed soil]*

Additionally, determine the theoretical density of the chosen mixture if the natural unit weight, the water content and the particle density of the undisturbed soil are known. This requires one to make a soil phase diagram of the undisturbed soil first. The theoretical density can then be derived using the following equation:

$$\rho_{theoretical} = \frac{m_s + m_w + m_b}{V_w + V_s + V_b}$$

<u>*where:*</u>

$\rho_{theoretical}$	*- theoretical maximum density of the mixture*	*[kg material/m³ stabilised soil]*
m_s	*- mass of solids in undisturbed soil sample*	*[kg]*
m_w	*- mass of water in undisturbed soil sample*	*[kg]*
m_b	*- mass of binder to be added*	*[kg]*
V_s	*- volume of solids in undisturbed soil sample*	*[m³]*
V_w	*- volume of water in undisturbed soil sample*	*[m³]*
V_b	*- volume of binder to be added*	*[m³]*

The theoretical density can be used to determine how much soil, water and binder will be required to fill the required amounts of moulds.

D.6.2 Phase 1 – Measurements before stabilisation

Before the soils can be stabilised, it is required that the masses of the mixture components are determined. This requires information on the moulds that are used.

In phase 1, the next steps must be taken in sequence:

1. Look up the binder material particle densities of the binder materials used and write them down in the table *"BINDER INFORMATION"* on the form.
2. Determine the number of samples that should be produced and write this number down in the table *"SAMPLE PRODUCTION"* on the form.
3. Fill in the table *"MOULD MASS"* and the table *"MOULD VOLUME"* on the form. Take the following steps to obtain the required information:
   3.1. Take a mould and label the mould, possibly by using stickers. Write the label down in both tables.
   3.2. Measure the inner diameter and the height of the mould and write these down. Keep in mind that the height of the mould that we write down is the height of the mould that will be filled with soil. For the moulds in the laboratory of Fugro, the inner diameter was 150 mm and the height of mould was 300 mm, but the height filled with soil was 270 mm.
   3.3. Attach the sock to the mould and weigh the mould with the sock. Write the mass down in the table.
   3.4. Repeat steps 3.1 through 3.4 for the required number of moulds.
4. Fill in the remaining tables on the form for this phase.
   4.1. Determine the mass of the mixture components, including a 25% additional mass, based on the mixture and density information.
      4.1.1. First, calculate the mass ratio of the different components when assuming 1,0 m^3 of undisturbed soil. This is done using the following equation:

$$mass\ ratio\ of\ component = \frac{\alpha_{binder}}{\rho_{stab}} \cdot 100\%$$

where:

$mass\ ratio\ of\ component$	*- mass ratio of the mixture component*	*[%]*
α_{binder}	*- mass of binder (component) added to the mass of soil that one cubic metre of undisturbed soil has*	*[kg binder/m^3 undisturbed soil]*
ρ_{stab}	*- total mass of material in the mixture when one cubic metre of undisturbed soil is stabilised with a specific mass of binder*	*[kg material/m^3 undisturbed soil]*

The same equation holds for the mass ratio of the soil density and, if applicable, for the mass ratio of the added water. Fill in the mass ratios in the table *"MASS OF MIXTURE COMPONENTS"* on the form.

4.1.2. Determine the minimum mass per stabilised soil sample (including an additional 25% of mass to ensure one ends up with sufficient material to fill the moulds) using the volume of the moulds. This can be done using the following equation:

$$m_{soil\ min.} = \rho_{guide} \cdot V_{mould} \cdot 1{,}25$$

<u>*where:*</u>

$m_{soil\ min.}$	*- minimum mass of material that is required to fill one mould (including an additional 25% of mass)*	*[g]*
ρ_{guide}	*- guiding density (i.e. expected amount of material required for making a sufficient amount of mixture to fill the desired number of moulds with this mixture)*	*[kg material/m³ stabilised soil]*
V_{mould}	*- volume of the mould*	*[L]*

The guiding density is an important parameter for determining the required quantities of soil, water and binder as it keeps the compaction of the mixture into account (see phase 5) such that hardly any air, that may have been present in the untreated soil, is present in the mixture. Keep in mind that the guiding density[2] can only be determined experimentally in the laboratory. If the value is not known from experience or experiments, set the guiding density equal to the theoretical density calculated earlier.

4.1.3. Determine the mass of the mixture components (including 25% additional mass) for a single sample. This is done using the following equation:

$$m_{component} = \frac{mass\ ratio\ of\ component}{100} \cdot m_{soil\ min.}$$

<u>*where:*</u>

$m_{component}$	*- required mass of the mixture component for the mixture*	*[g]*
$mass\ ratio\ of\ component$	*- mass ratio of the mixture component*	*[%]*
$m_{soil\ min.}$	*- minimum mass of material that is required to fill the mould (including an additional 25% of mass)*	*[g]*

Fill in the masses of the mixture components in the table *"MASS OF MIXTURE COMPONENTS"* on the form. For the masses for multiple samples, the required masses should be multiplied by the amount of samples.

4.1.4. Determine the mass per layer based on 5 layers. This can be done using the following equation:

$$m_{\ layer} = \frac{\rho_{guide} \cdot V_{mould}}{5}$$

<u>*where:*</u>

$m_{min.\ layer}$	*- (expected) mass of material required for creating 1 layer of compacted mixture in the mould*	*[g]*
ρ_{guide}	*- guiding density (i.e. expected amount of material required for making a sufficient amount of mixture to fill the desired number of moulds with this mixture)*	*[kg material/m³ stabilised soil]*
V_{mould}	*- volume of the mould*	*[L]*

Fill in the mass per layer based on 5 layers in the table *"MASS OF MIXTURE COMPONENTS"* on the form.

[2] The guiding density is the maximum achievable density in practice. It is a parameter that helps determine how much material is required to fill the desired number of moulds with compacted mixture.

D.6.3 Phase 2 – Homogenisation of the soil

With all the information required for the soil stabilisation known, the required mass of soil that will be stabilised is taken and mixed in a mixing bowl to a homogeneous mass.

In phase 2, the following steps must be taken in sequence:

1. Fill in the *"MASS REQUIRED AND WEIGHED"* table on the form:
   1.1. Copy the required mass of soil and, if applicable, the required mass of water for the required amount of samples from the table *"MASS OF MIXTURE COMPONENTS"* of phase 1 to the table *"MASS REQUIRED AND WEIGHED"* of phase 2 on the form.
   1.2. Put a box or bucket that will hold the mixture component on the scale, set it to 0, and fill the box or bucket with the required mass of soil. Do the same for the other mixture components. Write the down the actual weighed masses in the *"MASS REQUIRED AND WEIGHED"* on the form.
2. Take the box or bucket with the soil and, if applicable, the water to the mixer. Place the mixing bowl in the mixer and attach the mixing blade to the mixer.
3. Record the specifications of the mixer that will be used for mixing:
   3.1. Check the mixer brand and type and write this down in the table *"MIXER INFORMATION"* on the form.
   3.2. Look up or determine the mixer speed setting at which the soil will be mixed. Write the speed setting and the corresponding rotation and revolution of the mixing tool down in the table *"MIXER INFORMATION"* on the form. Typically, the lowest mixing setting is used.
4. Put the soil in the mixing bowl and start mixing the soil at the selected setting. If necessary, change the speed setting of the mixer to a faster setting and write down the used speed settings and the corresponding rotation and revolution of the mixing tool down in the table *"MIXER INFORMATION"* on the form.
5. Mix the soil to a visually homogeneous mass. Some sources prescribe a mixing time:
   5.1. Federal Highway Administration Design Manual: mix the soil for about 3 minutes.
   Dutch CUR 199: mix the soil for at least 5 minutes.
   5.2. The mixing times mentioned are guide numbers. It is always best to keep mixing the soil until it is visually homogeneous. Write down the time used for mixing of the soil in the table *"SOIL MIXING TIME"* on the form.
6. Take a small sample from the mixed soil and determine the water content:
   6.1. Take a small cup or plate and weigh it. Write down its mass in the table *"MIXED SOIL PROPERTIES"* on the form.
   6.2. Take a sample from the mixed soil in the mixing bowl and put it in the cup. Write down the combined mass of the cup and the wet soil down in the table *"MIXED SOIL PROPERTIES"* on the form. Also write down the current time. Keep in mind that, depending on the grain diameter, a minimum mass of soil should be oven dried as shown in table 1 in Dutch standard NEN-EN-ISO 17892-1.
   6.3. Put the sample in the oven at e.g. 60 °C when the soil contains (NEN-EN-ISO 17892-1):
      - Gypsum or other minerals having a significant amount of chemically-bonded water;
      - A significant amount of organic material.

   For such soils, the mass change due to drying may not be due to just loss of free water. These samples will then need to be dried until a mass change of less than 0,1% is measured for a further period of at least 1 hour.

   For soils not meeting these conditions, oven drying at 105 to 110 °C will be sufficient (NEN-EN-ISO 17892-1). Typically, fine soils take about 16 hours to fully dry. It may, however, be wise to stick with 60 °C, as oven-drying of stabilised soil samples, which may be required for some laboratory tests after those are finished, should also be done at such a temperature as it the stabilised soil will contain chemically-bonded water.
   6.4. At a later time, measure the combined mass of the cup and the dry soil sample and determine the water content. Write the combined mass, the water content and the current time down in the table *"MIXED SOIL PROPERTIES"* on the form. In the meantime, continue the stabilisation procedure.

D.6.4 Phase 3 – Preparation of the binder

With the soil mixed to a homogeneous material, the binder has to be prepared. The binder components will be taken and mixed to a homogeneous mass. This step is only required if a multi-component binder is to be used.

In phase 3, the next steps must be taken in sequence:

1. Fill in the *“MASS REQUIRED AND WEIGHED”* table on the form:
   1.1. Copy the required mass of binder materials for the required amount of samples from the table *“MASS OF MIXTURE COMPONENTS”* of phase 1 to the table *“MASS REQUIRED AND WEIGHED”* of phase 3 on the form.
   1.2. Put a box or bucket that will hold the mixture component on the scale, set it to 0, and fill the box or bucket with the required mass of soil. Do the same for the other mixture components. Write the down the actual weighed masses in the *“MASS REQUIRED AND WEIGHED”* on the form.
2. Take the box or bucket with the binder materials to the mixer. Place the bowl in the mixer and attach the mixing blade to the mixer.
3. Record the specifications of the mixer that will be used for mixing:
   3.1. Look up the mixer brand and type and write this down in the table *“MIXER INFORMATION”* on the form.
   3.2. Look up or determine the mixer speed setting at which the binder will be mixed. Write the speed setting and the corresponding rotation and revolution of the mixing tool down in the table *“MIXER INFORMATION”* on the form. Typically, the lowest mixing setting is used.
4. Put the mixing bowl in the mixer and start mixing the binder components at the selected setting. If necessary, change the speed setting of the mixer to a faster setting and write down the used speed settings and the corresponding rotation and revolution of the mixing tool down in the table *“MIXER INFORMATION”* on the form.
5. Mix the binder components to a visually homogeneous mass. Write down the time used for mixing of the binder components in the table *“BINDER MIXING TIME”* on the form.

D.6.5 Phase 4 – Mixing of the soil and binder

With the binder and the soil prepared, the binder and the soil will be mixed to a visually homogeneous mixture.

In phase 4, the next steps must be taken in sequence:

1. Turn on the mixer with the mixing bowl containing the mixed soil.
2. During mixing, gradually add the binder and mix the soil and binder to a visually homogeneous mass. Write down the time and mixer speed setting used for mixing the soil and the binder in the table *“SOIL AND BINDER MIXING TIME”* on the form.

D.6.6 Phase 5 – Compaction of the mixture in the mould

With the soil-binder mixture prepared, the moulds will be filled with the mixture. The mixture is put in the mould in 5 layers, with each layer compacted before the next one is put in the mould.

In phase 5, the next steps must be taken in sequence:

1. Put a mass of the mixture in the mould that equals the mass per layer based on 5 layers from the table *“MASS OF MIXTURE COMPONENTS”* of phase 1.
2. Spread the mass equally over the inner diameter of the mould. The mixture may be compacted using the fist.
3. Use the masher to compact the mixture:
   3.1. Place the masher on the mixture in the mould and push on the handles with the user’s own body weight to compact the mixture.
   3.2. Then use a recoil-free hammer to hit the masher 10 times on top.

3.3. Check whether the layer line of the masher[3] meets the top of the mould to make sure a single layer is compacted sufficiently. If not, add a little more mass and repeat steps 2, 3.1, 3.2 and 3.3 until the layer line is met.
3.4. Scratch the upper end (and especially the part close to the wall of the mould) of the stabilised soil sample loose. This is to enhance the bonding between the different layers and to try to reduce the visible layering of the sample.
3.5. Repeat steps 1 through 3.5 until all 5 layers have been made, making the full stabilised soil sample.

4. Assign a sample code to the compacted stabilised soil sample in mould 1. Write the sample code down in table *"SAMPLE CODE OF STABILISED SOIL SAMPLES"* on the form.
5. Put the mould with the compacted mixture on the scale and record its mass. Write the mass down in the table *"MOULD FILLING WITH MASS OF STABILISED SOIL"*.
6. Preferably store mould 1 in a conditioned room until all moulds are filled.
7. Repeat steps 1 through 6 for the remaining moulds.

D.6.7 Phase 6 – Storage and loading

With the moulds filled with the compacted mixture, the moulds are put in the mass stabilisation test setup. Then, the mixtures are loaded with a preload for a predetermined curing time.

In phase 6, the next steps must be taken in sequence:

1. Fill the container of the mass stabilisation setup with a small layer of water.
2. Take all moulds filled with compacted stabilised soil and put it in the large container of the mass stabilisation test setup that is filled with a layer of water. Cover the setup-specific mashers with a cloth and place them on the compacted mixtures inside the moulds. The mashers should not be loaded yet. Write down the labels of the moulds in the table *"COMPRESSION OF STABILISED SOIL SAMPLES"* on the form.
3. Read off the millimetre sticker on all mashers to the nearest integer and write down these measurements in table *"COMPRESSION OF STABILISED SOIL SAMPLES"* on the form. These measurements are the *'Before loading'* measurement.
4. Place the required weights matching the desired in-situ stresses (including preload) on the PVC tube that connects to the masher. Write down the applied mass and the corresponding preload in the table *"PRELOADING"* on the form.
5. Start the stopwatch for the 5 minute measurement. Note down the time at the start of the loading.
6. After 5 minutes (300 ± 30 seconds) the millimetre sticker is read off again. Write the 5-minute settlement of the sample in the table *"COMPRESSION OF STABILISED SOIL SAMPLES"* on the form. Write down the current time in the table *"TIMES AT COMPRESSION MEASUREMENTS OF STABILISED SOIL SAMPLES"* on the form.
7. Record the storage conditions of the moulds in the laboratory in the table *"STORAGE CONDITIONS"* on the form.
8. After a certain amount of time, depending on the curing time allowed for all samples in this series, the millimetre sticker is read off again. These measurements of the compression are written down in the table *"COMPRESSION OF STABILISED SOIL SAMPLES"* on the form. Also note the times at which the measurements were done in the table *"TIMES AT COMPRESSION MEASUREMENTS OF STABILISED SOIL SAMPLES"* on the form. The other curing times at which a measurement of the compression should be done are:
   - After 30 minutes (± 1 minute);
   - After 60 minutes (± 1 minutes);
   - After 24 hours (± 15 minutes);
   - After 48 hours (± 30 minutes);
   - After 7 days (± 2 hours);
   - After 14 days (± 4 hours);

[3] The masher used in the laboratory of Fugro was marked with 'layer lines'. The height of the masher was made to match the total height of the mould. The layer lines represented the point when 1/5 of the mould was filled with soil, such that it was easy to check whether about 1/5 of the mould was filled. Please note that the layer lines represent 1/5 of the mould height that would be filled with soil and not 1/5 of the total height of the mould.

- After 28 days (± 8 hours);
- After the allowed curing period if it is not at any of the times mentioned above.

Record any deviations in the storage conditions if they occurred.

D.6.8 Phase 7 - Removal of the stabilised soil sample from the mould

After the specified curing time, the stabilised soil samples are extruded from the moulds. However, if samples smaller than the extruded stabilised soil samples are required, follow phase 8 instead.

In phase 7, the next steps must be taken in sequence:

1. Record any deviations of the storage conditions if they occurred in the table *"DEVIATIONS IN STORAGE CONDITIONS"* table on the form.
2. Remove the weights and the masher off of all moulds and take the moulds from the water-filled container of the mass stabilisation setup. Read off the labels of all moulds and write down the label in all tables of phase 7 on the form.
3. Place the mould on a flat surface and examine the roughness of the upper end of the compressed stabilised soil in the mould. Record the roughness (i.e. smooth or rough) in the table *"ROUGHNESS OF END SURFACE OF SAMPLES BEFORE EXTRUSION"* on the form.
4. Weigh the mould with the sock and the stabilised soil. Note the mass of the mould with sock and the stabilised soil in the table *"MASS OF STABILISED SOIL SAMPLES"* on the form.
5. Extrude the stabilised soil from the mould using an extrusion machine. Record any difficulties with the extrusion of the sample from its mould in the table *"STABILISED SOIL SAMPLE INSPECTION"* on the form.
6. Inspect the extruded stabilised soil sample. Record any irregularities, such as visible holes, large voids or the bottom end not being entirely flat and perpendicular to the length axis of the sample. Write down any irregularity found in the table *"STABILISED SOIL SAMPLE INSPECTION"* on the form.
7. Measure the mass of the extruded stabilised soil sample. Write down the mass of the extruded soil sample in the table *"MASS OF STABILISED SOIL SAMPLES"* on the form.
8. Measure the diameter and height of the extruded soil sample and write these down in the table *"VOLUME OF EXTRUDED SOIL SAMPLES"* on the form.
9. Store the extruded stabilised soil sample in a conditioned room and repeat steps 2 through 9 for the remaining moulds of this series.
10. Determine the guiding density for the samples and write these down in the table *"GUIDING DENSITY FOR THE MIXTURE"*. The guiding density can be determined using the following equation:

$$\rho_{guide} = \frac{m_{mould+compacted\ mixture} - m_{mould}}{\frac{\pi}{4} \cdot D_{extruded}^2 \cdot h_{extruded} + \frac{\pi}{4} \cdot D_{mould}^2 \cdot compression} \cdot 1.000.000$$

<u>*where:*</u>

ρ_{guide}	*- mass ratio of the mixture component*	*[kg material/m³ stabilised soil]*
$m_{mould+compacted\ mixture}$	*- mass of the mould when filled with the mixture*	*[g]*
m_{mould}	*- mass of the (empty) mould*	*[g]*
$D_{extruded}$	*- diameter of the extruded sample*	*[mm]*
D_{mould}	*- inner diameter of the mould*	*[mm]*
$h_{extruded}$	*- height of the extruded sample*	*[mm]*
$compression$	*- measured compression of the sample*	*[mm]*

D.6.9 Phase 8 – Extrusion of smaller samples (optional)

This phase is optional and should only be carried out when stabilised soil samples with a diameter smaller than the diameter of the otherwise extruded stabilised soil samples are desired.

In phase 8, the next steps must be taken in sequence:

1. Record any deviations of the storage conditions if they occurred in the table *"DEVIATIONS IN STORAGE CONDITIONS"* table on the form.
2. Remove the weights and the masher off of all moulds and take the moulds from the water-filled container of the mass stabilisation setup. Read off the labels of all moulds and write down the labels in all tables of phase 8 on the form.
3. Place the mould on a flat surface and examine the roughness of the upper end of the compressed stabilised soil in the mould. Record the roughness (i.e. smooth or rough) in the table *"ROUGHNESS OF END SURFACE OF SAMPLES BEFORE EXTRUSION"* on the form.
4. Weigh the mould with the sock and the stabilised soil. Note the mass of the mould with sock and the stabilised soil in the table *"MASS OF STABILISED SOIL SAMPLES"* on the form.
5. Place mould 1 filled with the stabilised soil in the compression machine.
6. Take a number of tubes with an inner diameter that is smaller than the inner diameter of the mould. Spray the inside of the tubes with lubricant (e.g. Teflon spray).
7. Attach the smaller tubes (e.g. 3 tubes with an inner diameter of 50 mm) together. Subsequently place the attached tubes on top of the stabilised soil in the mould.
8. Place a metal plate on top of the smaller tubes and connect the masher to the compression machine. The metal plate is required for equal compression of the tubes into the sample.
9. Mark the tubes at the point that would indicate that the tubes would have been almost completely pressed into the stabilised soil. A good reference measure is 2 cm from the bottom of the mould.
10. Start the compression machine and press the tubes into the sample up to the mark.
11. Stop the compression machine and remove the masher.
12. Place the mould with the tubes pressed into the stabilised soil sample in the larger compression machine. Place a metal plate between the masher and the tubes to allow for equal compression of the stabilised soil from the mould.
13. Start the sample extrusion.
14. Record any difficulties with the extrusion of the sample from its mould in the table *"STABILISED SOIL SAMPLE INSPECTION"* on the form.
15. After extrusion of the large stabilised soil sample from the mould, cut off the stabilised soil around the tubes with a knife or screwdriver.
16. Then, extrude the stabilised soil from the tubes using a similar, but more precise procedure in the bigger compression machine.
17. Record any difficulties with the extrusion of the sample from its mould in the table *"STABILISED SOIL SAMPLE INSPECTION"* on the form.
18. Trim the extruded smaller samples to the desired height.
19. Inspect the extruded stabilised soil samples. Record any irregularities, such as visible holes, large voids or the bottom end not being entirely flat and perpendicular to the length axis of the sample. Write down any irregularity found in the table *"STABILISED SOIL SAMPLE INSPECTION"* on the form.
20. Measure the mass, height and diameter of the smaller samples and write this down in the table *"EXTRUDED SMALLER SOIL SAMPLES FROM THE LARGE STABILISED SOIL SAMPLE"* on the form. Additionally, label the smaller samples and write the labels down in the table *"LABELS FOR EXTRUDED SMALLER SOIL SAMPLES FROM THE LARGE STABILISED SOIL SAMPLE"* on the form.
21. Repeat steps 1 through 20 for the remaining moulds from which smaller stabilised soil samples need to be obtained.

D.6.10 Phase 9 – Preparation of the sample ends (if applicable)

This phase is optional and should only be carried out if the upper end of the large stabilised soil samples are rough or not perpendicular to the length axis of the sample. If smaller samples were extruded from the large stabilised soil samples, this step can be skipped for that mould.

In phase 9, the following steps must be taken in sequence:

1. If the sample is to be tested with a test in which the permeability of the sample ends is not of importance, such as for a unconfined compression test (UCS test) or for a unconsolidated, undrained triaxial test (UU triaxial test), the upper surface of the sample is smoothened with a

thin layer of gypsum. Record the height of the adjusted sample and write it down in the table *"TREATMENT OF STABILISED SOIL SAMPLES"* on the form. Also write down that a treatment was applied and that gypsum was used.

2. If the sample is to be tested with a test in which the permeability of the sample ends is of importance, such as for a consolidated, undrained triaxial test (CU triaxial test), a small slice is cut from the upper end of the sample to obtain a flat surface perpendicular to its length axis. Record the height of the adjusted sample and write it down in the table *"TREATMENT OF STABILISED SOIL SAMPLES"* on the form. Also write down that a treatment was applied and that the sample was cut.

D.6.11 Phase 10 – Geotechnical laboratory testing

In this step, the (smaller) stabilised soil samples are tested in the laboratory to determine the properties of the stabilised soil samples. The test should be carried out according to local standards.

D.7 Form to fill in during the laboratory soil stabilisation procedure

Phase 0: General stabilisation information

GENERAL INFORMATION	
Phase of research	
Date of sample preparation	
Hardening period (curing time)	days
Date of sample testing	
Number of samples prepared	

MATERIAL INFORMATION	
Material	**Name**
Soil type tested	
Cement type used	
Water type used	
Ground-granulated blast-furnace slag (GGBS) type used	
Lime type used	
Gypsum type used	

MIXTURE INFORMATION					
Material	**Ratio of components**			**Water content of untreated soil**	
Soil	$\rho_{natural\ soil}$		kg/m^3		%
Cement	α_{cement}		kg cement/m^3 soil		
Water	α_{water}		kg water/m^3 soil		
GGBS	α_{GGBS}		kg GGBS/m^3 soil		
Lime	α_{lime}		kg lime/m^3 soil		
Gypsum	α_{gypsum}		kg gypsum/m^3 soil		

DENSITY INFORMATION		
Type of density	**Value**	**Unit**
Theoretical density		kg/m^3
Guiding density		kg/m^3

Phase 1: Measurements before stabilisation

BINDER INFORMATION		
Particle density of cement used		kg/m^3
Particle density of GGBS used		kg/m^3
Particle density of lime used		kg/m^3
Particle density of gypsum used		kg/m^3

SAMPLE PRODUCTION		
Number of samples that will be produced		-

MOULD MASS						
Mould number	**Label (sticker)**	**Table number**	**Mass (without sock)**		**Mass (with sock)**	
1				g		g
2				g		g
3				g		g
4				g		g
5				g		g

MOULD VOLUME							
Mould number	**Label (sticker)**	**Inner diameter**		**Height**		**Volume**	
1			m		m		m^3
2			m		m		m^3
3			m		m		m^3
4			m		m		m^3
5			m		m		m^3

MASS OF MIXTURE COMPONENTS, INCLUDING 25% ADDITIONAL MASS												
Material	**1 sample**		**2 samples**		**3 samples**		**4 samples**		**5 samples**		**Mass ratio**	
Soil		g		g		g		g		g		%
Cement		g		g		g		g		g		%
Water		g		g		g		g		g		%
GGBS		g		g		g		g		g		%
Lime		g		g		g		g		g		%
Gypsum		g		g		g		g		g		%
Total		g		g		g		g		g		%

Mass per layer based on 5 layers		g

Phase 2: Homogenisation of the soil

MASS REQUIRED AND WEIGHED						
Material	**Mass required**		**Mass weighed**		**Mass ratio of weighed masses**	
Soil		g		g		%
Water		g		g		%

MIXER INFORMATION		
Brand of mixer		-
Type of mixer		-

SOIL MIXING TIME		
Time used for mixing of the soil (until visually homogeneous)		min
Binder mixed at speed setting		-
Approximate rotation of mixing tool at this speed setting		RPM
Revolution of the mixing tool around the bowl in the planetary mixing action at this speed setting		RPM

MIXED SOIL PROPERTIES		
Mass of empty plate		g
Mass of plate and wet mass of mixed soil		g
Date and time at start of oven drying		-
Date and time at end of oven drying		-
Mass of plate and dry mass of mixed soil (mass of soil solids)		g
Water content of mixed soil		-

Phase 3: Preparation of the binder

MASS REQUIRED AND WEIGHED						
Material	**Mass required**		**Mass weighed**		**Mass ratio of weighed masses**	
Cement		g		g		%
GGBS		g		g		%
Lime		g		g		%
Gypsum		g		g		%
Total		g		g		%

MIXER INFORMATION		
Brand of mixer		-
Type of mixer		-

BINDER MIXING TIME		
Time used for mixing of the binder (until visually homogeneous)		min
Binder mixed at speed setting		-
Approximate rotation of mixing tool at this speed setting		RPM
Revolution of the mixing tool around the bowl in the planetary mixing action at this speed setting		RPM

Phase 4: Mixing of the soil and binder

SOIL AND BINDER MIXING TIME		
Time used for mixing of the soil with the binder (until visually homogeneous)		min
Binder and soil mixed together at speed setting		-

Phase 5: Compaction of the mixture in the mould

MOULD FILLING WITH MASS OF STABILISED SOIL										
Mass of mould with sock with	**Mould 1**		**Mould 2**		**Mould 3**		**Mould 4**		**Mould 5**	
Label										
5 layers		g		g		g		g		g

Phase 6: Storage and loading

STORAGE CONDITIONS		
Storage temperature of air		°C
Water temperature		°C
Expected deviations from the storage temperature during curing		°C
Expected deviations from the water temperature during curing		°C

PRELOADING		
Applied mass		kg
Corresponding preload		kN/m^2

COMPRESSION OF STABILISED SOIL SAMPLES										
Settlement at	**Mould 1**		**Mould 2**		**Mould 3**		**Mould 4**		**Mould 5**	
Label										
Before loading		mm		mm		mm		mm		mm
5 minutes		mm		mm		mm		mm		mm
30 minutes		mm		mm		mm		mm		mm
60 minutes		mm		mm		mm		mm		mm
24 hours		mm		mm		mm		mm		mm
48 hours		mm		mm		mm		mm		mm
7 days		mm		mm		mm		mm		mm
14 days		mm		mm		mm		mm		mm
28 days		mm		mm		mm		mm		mm

TIMES AT COMPRESSION MEASUREMENTS OF STABILISED SOIL SAMPLES					
Time at compression measurement at	**Mould 1**	**Mould 2**	**Mould 3**	**Mould 4**	**Mould 5**
Label					
Before loading					
5 minutes					
30 minutes					
60 minutes					
24 hours					
48 hours					
7 days					
14 days					
28 days					

Phase 7: Removal of the stabilised soil sample from the mould

DEVIATIONS IN STORAGE CONDITIONS		
Occurred deviations in storage temperature of air, if applicable		°C
Occurred deviations in water temperature, if applicable		°C

ROUGHNESS OF END SURFACE OF SAMPLES BEFORE EXTRUSION					
	Mould 1	**Mould 2**	**Mould 3**	**Mould 4**	**Mould 5**
Label					
Roughness of end surface					

MASS OF STABILISED SOIL SAMPLES										
	Mould 1		Mould 2		Mould 3		Mould 4		Mould 5	
Label										
Mass of sample with mould and sock		g		g		g		g		g
Mass of extruded sample		g		g		g		g		g

VOLUME OF EXTRUDED STABILISED SOIL SAMPLES										
	Mould 1		Mould 2		Mould 3		Mould 4		Mould 5	
Label										
Height		m		m		m		m		m
Diameter		m		m		m		m		m

GUIDING DENSITY FOR THE MIXTURE										
	Mould 1		Mould 2		Mould 3		Mould 4		Mould 5	
Label										
Guiding density		kg/m^3		kg/m^3		kg/m^3		kg/m^3		kg/m^3

STABILISED SOIL SAMPLE INSPECTION					
	Mould 1	Mould 2	Mould 3	Mould 4	Mould 5
Label					
Difficulties with sample extrusion, if applicable					
Irregularities of the extruded sample, if applicable					

Phase 8 - Extrusion of smaller stabilised soil samples (if applicable)

DEVIATIONS IN STORAGE CONDITIONS		
Occurred deviations in storage temperature of air, if applicable		°C
Occurred deviations in water temperature, if applicable		°C

ROUGHNESS OF END SURFACE OF SAMPLES BEFORE EXTRUSION					
	Mould 1	Mould 2	Mould 3	Mould 4	Mould 5
Label					
Roughness of end surface					

MASS OF STABILISED SOIL SAMPLES										
	Mould 1		**Mould 2**		**Mould 3**		**Mould 4**		**Mould 5**	
Label										
Mass of sample with mould and sock		g		g		g		g		g

LABELS FOR EXTRUDED SMALLER SOIL SAMPLES FROM THE LARGE STABILISED SOIL SAMPLE					
	Mould 1 label	**Mould 2 label**	**Mould 3 label**	**Mould 4 label**	**Mould 5 label**
Label mould					
Sample 1					
Sample 2					
Sample 3					
Sample 4					
Sample 5					
Sample 6					

EXTRUDED SMALLER SOIL SAMPLES FROM THE LARGE STABILISED SOIL SAMPLE										
	Mould 1		**Mould 2**		**Mould 3**		**Mould 4**		**Mould 5**	
Label mould										
Height 1		m		m		m		m		m
Height 2		m		m		m		m		m
Height 3		m		m		m		m		m
Height 4		m		m		m		m		m
Height 5		m		m		m		m		m
Height 6		m		m		m		m		m
Diameter 1		m		m		m		m		m
Diameter 2		m		m		m		m		m
Diameter 3		m		m		m		m		m
Diameter 4		m		m		m		m		m
Diameter 5		m		m		m		m		m
Diameter 6		g		g		g		g		g
Mass 1		g		g		g		g		g
Mass 2		g		g		g		g		g
Mass 3		g		g		g		g		g
Mass 4		g		g		g		g		g
Mass 5		g		g		g		g		g
Mass 6		g		g		g		g		g

STABILISED SOIL SAMPLE INSPECTION					
	Mould 1	**Mould 2**	**Mould 3**	**Mould 4**	**Mould 5**
Label(s)					
Difficulties with sample extrusion, if applicable					
Irregularities of the extruded sample, if applicable					

<u>Phase 9 - Preparation of sample ends (if applicable)</u>

TREATMENT OF STABILISED SOIL SAMPLES										
	Mould 1		**Mould 2**		**Mould 3**		**Mould 4**		**Mould 5**	
Label										
Treatment of upper end prior to further testing	Yes/No		Yes/No		Yes/No		Yes/No		Yes/No	
Applied treatment if applicable										
Height of adjusted sample if applicable		m		m		m		m		m

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Appendix E - Soil sampling in the field

E.1 Soil profile at the cross-section of the levee

For the design calculations in D-GeoStability, the soil profile had to be known. Using the borings in appendix A, section A.1, the soil profile at the selected cross-section of the levee at the Montfoortse Vaart was derived. The derived soil profile at the cross-section is presented in table b.1, which is schematically shown in figure e.1.

Table E.1; Soil profile of the cross-section at the levee of the Montfoortse Vaart. The borings that were used to derive this soil profile are presented in appendix A, section A.1.

Soil layer	Lower boundary at the crest [m NAP]	Lower boundary at the toe [m NAP]
Surface	+0,32	-1,07
Silty clay	-2,88	Does not exist at toe
Organic clay	Does not exist at toe	-2,37
Peat, poor in minerals	-5,77	-5,77
Sand	-7,07 (end of measurements)	-7,07 (end of measurements)

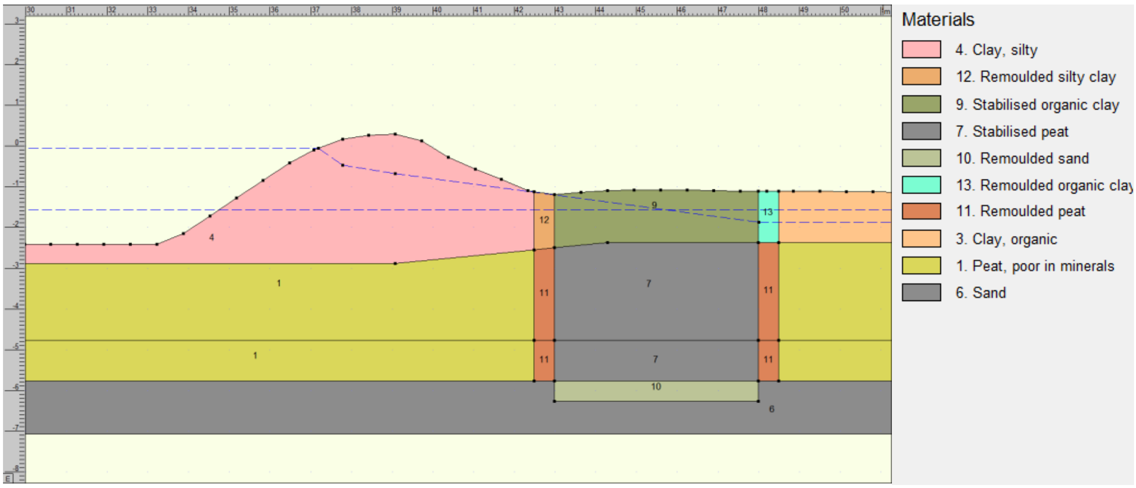


Figure E.1; D-GeoStability model of the design of the levee at the Montfoortse Vaart with stabilised soil.

Mostly due to time restrictions only the stabilisation of the soils at the toe of the levee (see figure e.1) was examined in the geotechnical laboratories of Fugro NL Land B.V. in Arnhem (Netherlands) and Delft University of Technology (Netherlands). Additionally, the clay the levee consists of is different than the clay at the toe of the levee. Since only the soil at the of the levee was sampled for a number of practical reasons, the results obtained in the laboratory could not be applied to stabilised soils within the levee due to compositional differences.

E.2 Required amount of soil

In order to examine the feasibility of the design shown in figure e.1 by means of laboratory testing, both organic clay and peat was required to be sampled. In order to carry out the extensive laboratory research programme outlined in the main report (see section 4.2), it was estimated that about 1,0 m^3 of organic clay and 1,0 m^3 of peat would be required. For the 2,0 m^3 of soil it did not matter whether the soil was remoulded or not, because the soil would be reconstituted in the laboratory anyway.

Aside from the 1,0 m^3 of remoulded peat and remoulded organic clay, a number of undisturbed samples of both the peat and the organic clay were also required. These undisturbed soil samples were required to determine the properties of the soil in the field on which the soil-binder mixtures would be based. In order to be able to determine the properties of the undisturbed soil samples in the laboratories, it was estimated that 5 thin-walled open-ended Ackermann tubes of both peat and organic clay were required.

E.3 Soil profile in the field

The 2,0 m^3 of remoulded soil and the 10 tubes with undisturbed soil samples were required to be taken as close to the examined cross-section of the levee as possible. In order to be able to do so, it was attempted to convince the owner of the meadow east of the examined cross-section to allow for the author to make an excavation. The land owner did not allow it as the owner feared uncontrolled settlement as a result of the excavation. Subsequently the municipality of Montfoort was contacted. The municipality owned and maintained a parcel of land directly north of the examined cross-section. This parcel of land is known locally as the Ecopark of Linschoten and is highlighted in orange in figure e.2. The Municipality of Montfoort was enthusiastic about the project and was willing to allow the excavation and the sampling of the soil.

However, before the excavation could take place, the soil profile in the Ecopark had to be determined because it was unknown. Therefore two manual borings were carried out by the author in the Ecopark on Wednesday 22 August. The locations of these two borings are highlighted in figure e.2. The soil profiles as determined from visual inspection of the soil obtained from the borings is presented in table e.2.

The groundwater level was found at about 1,1 metre below ground surface at both boring locations. This is very close to the boundary between the organic clay and peat layer at the location of boring 2.

Table E.2; Soil profile at the location of the two boring carried out in the Ecopark.

Soil layer	Lower boundary at boring 1 [m NAP]	Lower boundary at boring 2 [m NAP]
Surface	0,0	0,0
Organic clay	-2,3	-1,5
Peat	-3,9 (end of boring)	-2,8
Silty clay	Not found	-4,0 (end of boring)

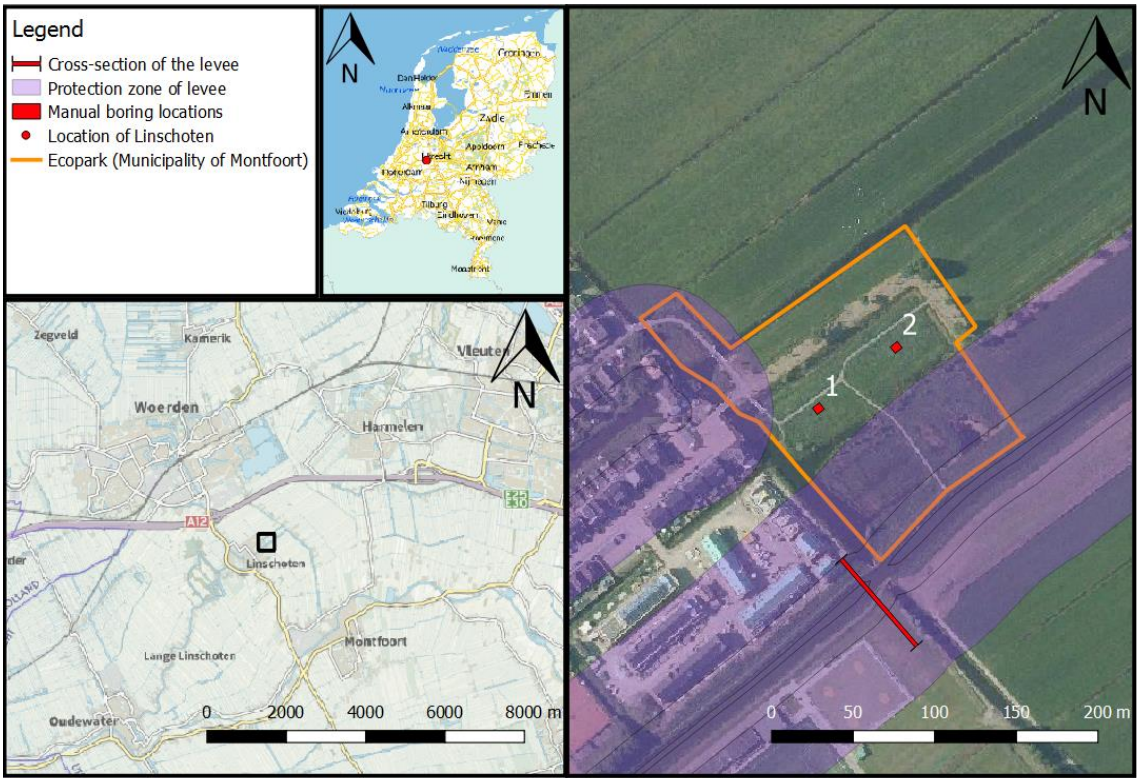


Figure E.2; The location of the manual borings that were carried out in the Ecopark of Linschoten. The numbers of the manual borings are indicated in the right image and correspond to the boring numbers in table e.2.

E.4 Execution of the work

With the soil profile in the Ecopark known, the soil sampling in the Ecopark was planned. It was decided that a trench would be dug at the location of boring 2 due to the shallow depth of the peat layer. This enabled the digging of a shallower trench. However, this simultaneously presented a problem. Since the groundwater table was found very close to the boundary between the organic clay and the peat layer, a large area had to be excavated to obtain the required 1,0 m^3 of wet organic clay.

On Tuesday, October 9th the soil sampling in the Ecopark took place. In the early morning, the mini-excavator and four pallet boxes for the storage of the excavated soil were transported to Linschoten. Once there, the truck transporting the mini-excavator and the pallet boxes was unloaded. The mini-excavator then transported itself across the bicycle bridge to the site (see figure e.3), whereas a separate car with trailer was used to transport the pallet boxes across the bicycle bridge. Once on site, the mini-excavator dug the trench at the pre-determined spot (see figure e.3). Each time the excavator dug up some soil, the excavated soil was assessed. If the excavated soil was assessed sufficiently wet, the soil was put in a pallet box. During the excavation, care was taken to make sure the organic clay and the peat layer were excavated separately and put in separate pallet boxes.

After the trench was dug, 5 Ackermann tubes were used to sample the peat layer vertically at the bottom of the trench, whereas another 5 Ackermann tubes were used to sample the organic clay layer horizontally in the wall of the trench.

After the soil sampling was completed, the mini-excavator filled the trench with the soil it had previously excavated. After compaction of the soil by the mini-excavator, a layer of garden soil was applied to compensate for the excavated 2,0 m^3 of soil. The garden soil was sown with grass seed and subsequently ploughed to prevent birds from eating the grass seed. Then, the filled pallet boxes and the mini-excavator were loaded unto the truck and subsequently transported away, leaving a site restored to its original state.

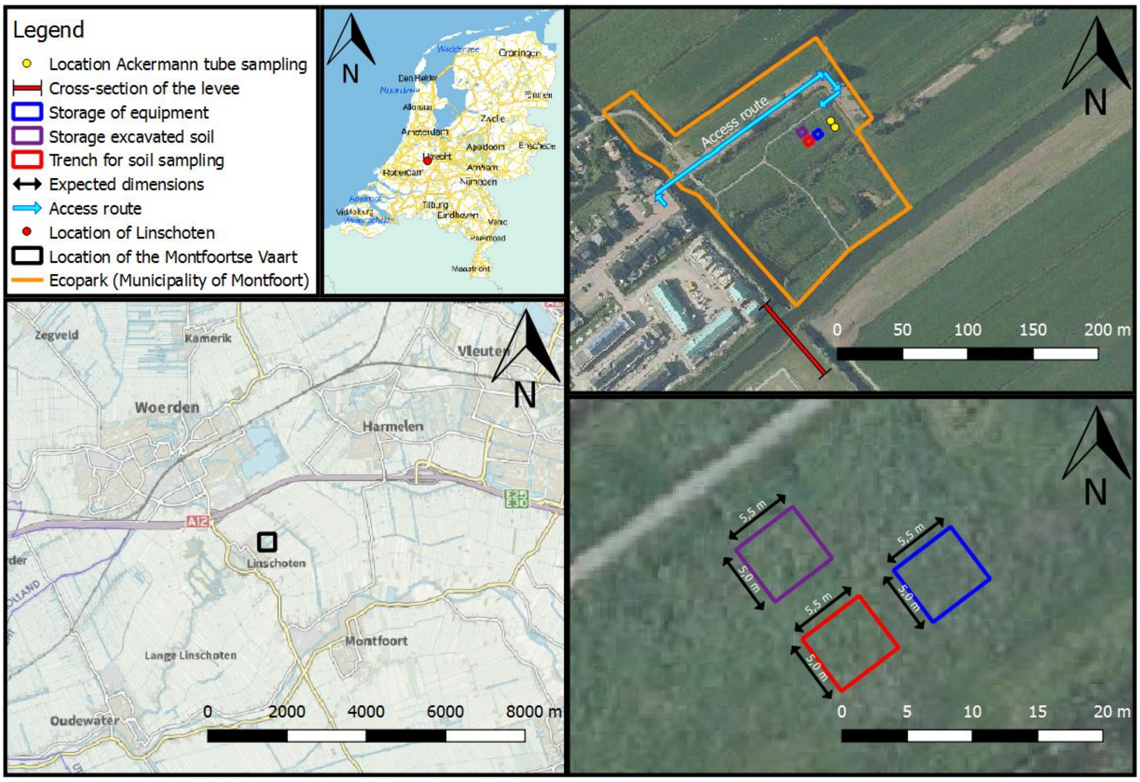


Figure E.3; The location of the dug trench, along with expected dimensions of the trench and the storage areas for the equipment and excavated soil.

Appendix F - Laboratory test results

F.1 Phase 1 – Soil parameters of undisturbed soil samples

F.1.1 Soil classification

At the laboratory of Fugro NL Land B.V. the undisturbed soil samples were extruded from the 10 Ackermann tubes. A photograph of some of the 10 extruded undisturbed soil samples are shown in figure f.1. By analysing the contents of the tubes it was found that some of the tubes containing peat were only filled for about 80%, whereas all other tubes were found to be much fuller with soil. Regardless of some tubes not being fully filled, the samples came out of the tubes very easily and did not show any cracks or signs of disturbance.

Figure F.1; The undisturbed soil samples extruded from the Ackermann tubes. The contents of tubes 8, 9, 10, 1, 4 and 5 are shown from left to right containing peat (the three most left tubes) and organic clay (the three most right tubes).

A full classification of the contents of all tubes was not done, but the laboratory technician who carried out an oedometer test on some of these samples gave a description of the material. This description is presented in table f.1. In addition to the description for the peat, it was noted that all undisturbed peat samples were relatively rich in wood.

Table F.1; Visual description of the undisturbed peat and organic clay soil samples as carried out by the laboratory technician of Fugro NL Land B.V. prior to the oedometer test.

Soil type	Visual description
Peat	Peat, poor in minerals, brown
Organic clay	Clay, strongly silty, moderately organic, grey

F.1.2 Bulk density

Table F.2; Bulk unit weight measurements on the undisturbed peat and organic clay samples.

Soil type	ρ_{bulk} sample 1	Sample 1 taken from tube	ρ_{bulk} sample 2	Sample 2 taken from tube	ρ_{bulk} sample 3	Sample 3 taken from tube	Mean value ρ_{bulk}	Standard deviation ρ_{bulk}
	[kN/m^3]	[-]	[kN/m^3]	[-]	[kN/m^3]	[-]	[kN/m^3]	[kN/m^3]
Peat	9,98	8	9,44	9	9,69	10	9,70	0,272
Organic clay	12,47	1	12,84	4	12,78	5	12,70	0,197

F.1.3 Dry density and water content

Table F.3; Dry unit weight measurements on the undisturbed soil samples.

Soil type	γ_{dry} sample 1	Sample 1 taken from tube	γ_{dry} sample 2	Sample 2 taken from tube	γ_{dry} sample 3	Sample 3 taken from tube	Mean value γ_{dry}	Standard deviation γ_{dry}
	[kN/m³]	[-]	[kN/m³]	[-]	[kN/m³]	[-]	[kN/m³]	[kN/m³]
Peat	2,31	8	2,06	9	1,98	10	2,12	0,17
Organic clay	5,58	1	5,76	4	5,67	5	5,67	0,09

Table F.4; Water content measurements on the undisturbed soil samples.

Soil type	w_{nat} sample 1	Sample 1 taken from tube	w_{nat} sample 2	Sample 2 taken from tube	w_{nat} sample 3	Sample 3 taken from tube	Mean value w_{nat}	Standard deviation w_{nat}
	[%]	[-]	[%]	[-]	[%]	[-]	[%]	[%]
Peat	331	8	358	9	389	10	359	29
Organic clay	124	1	123	4	125	5	124	1

F.1.4 Particle density

Table F.5; Particle density measurements on the undisturbed organic clay samples.

Soil type	ρ_s sample 1	Sample 1 taken from tube	ρ_s sample 2	Sample 2 taken from tube	ρ_s sample 3	Sample 3 taken from tube	Mean value ρ_s	Standard deviation ρ_s
	[Mg/m³]	[-]	[Mg/m³]	[-]	[Mg/m³]	[-]	[Mg/m³]	[Mg/m³]
Organic clay	2,50	1	2,51	4	2,51	5	2,50	0,01
	2,49	1	2,48	4	2,49	5		

Table F.6; Particle density measurements on the disturbed peat samples from the palletbox.

Soil type	Volume of pycno-meter	Oven drying tempe-rature	ρ_s sample 1	ρ_s sample 2	ρ_s sample 3	Mean value ρ_s	Standard deviation ρ_s
	[mL]	[°C]	[Mg/m³]	[Mg/m³]	[Mg/m³]	[Mg/m³]	[Mg/m³]
Peat	50	60	1,74	1,76	-	1,76	0,02
			1,78	1,76	-		
	50	110	1,77	-	-	1,78	0,01
			1,79	-	-		
	100	60	1,76	-	-	1,77	0,00
			1,77	-	-		
	100	110	1,79	-	-	1,78	0,02
			1,77	-	-		

For the particle density of the peat no tube numbers were recorded, because remoulded peat was used as an initial test with the oven-dried undisturbed peat failed. For the tests with the remoulded peat, multiple pycnometer volumes and soil drying temperatures were applied as a result of an argument between the author and the laboratory technicians on the procedure of the pycnometer tests. Although the results show that there are hardly any differences between the measurements, the measurements on the particle density of the remoulded peat dried at 60 °C using 100 mL pycnometer were used for further analyses.

F.1.5 Unconfined compression tests (triaxial cell)

Table F.7; Properties of the undisturbed peat and organic clay sample as measured during the unconfined compression test.

Soil type	d	h	h/d	ρ_{bulk}	ρ_{dry}	w_{nat}	UCS	ε_f	Test rate
[-]	[mm]	[mm]	[-]	[kg/m³]	[kg/m³]	[%]	[kPa]	[%]	[%/min]
Peat	64	129,5	2,02	1,136	0,208	447	16	10,3	0,77
Org. clay	66	134,4	2,04	1,381	0,621	122	24	11,5	0,74

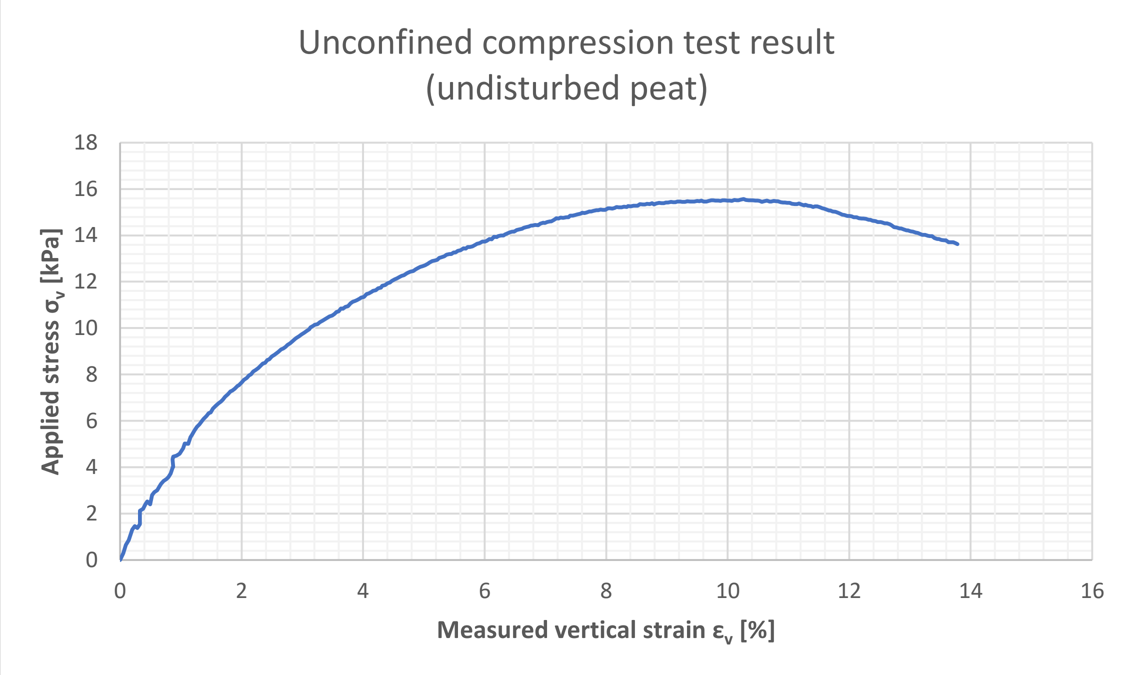


Figure F.2; The stress-strain diagram of the unconfined compression test on the undisturbed peat sample.

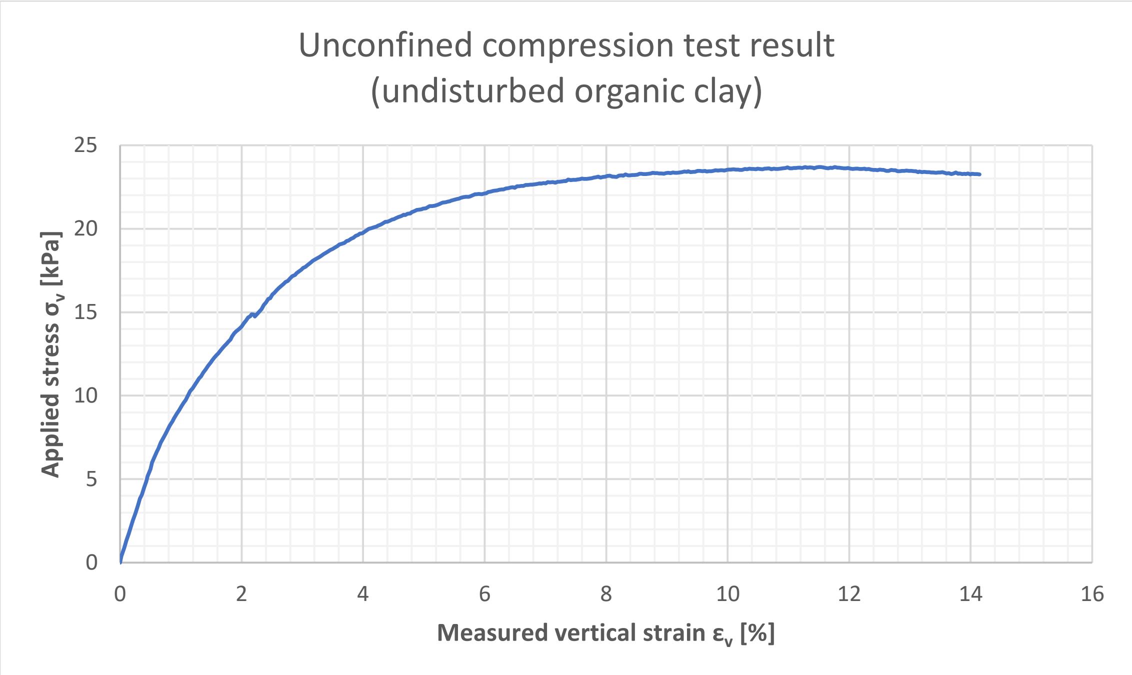


Figure F.3; The stress-strain diagram of the unconfined compression test on the undisturbed organic clay sample.

F.1.6 Shear box tests

Table F.8; Properties of the undisturbed peat samples as measured for the shear box tests. NM = not measured.

Soil type	Applied normal stress	Taken from tube	d	h	$\gamma_{bulk;i}$	$\gamma_{dry;i}$	$w_{nat;i}$	ρ_s (measured)
	[kPa]	[-]	[mm]	[mm]	[kN/m³]	[kN/m³]	[%]	[Mg/m³]
Peat	16,0	7	20,95	62,80	9,99	1,94	416	1,77
	60,0	6	20,95	62,80	8,70	2,09	317	1,77
	120	7	20,95	62,80	10,10	NM	NM	1,77
	135	7	20,95	62,80	10,24	2,31	344	1,77

Table F.9; Information about the shear box tests on the undisturbed peat samples.

Soil type	Applied normal stress	Sheared	Test rate	Strain reversal
	[kPa]	[-]	[mm/min]	[-]
Peat	16,0	Submerged	0,013	No
	60,0	Submerged	0,013	No
	120	Submerged	0,013	No
	135	Submerged	0,013	No

Table F.10; Data from the shear box test at failure of the undisturbed peat samples.

Soil type	Applied normal stress	Adopted failure criterion	$\tau_{\max}$	u_f	γ_f	c'	ϕ'
	[kPa]	[-]	[kPa]	[mm]	[%]	[kPa]	[°]
Peat	16,0	Max. τ	19,1	15,4	77,2	9,49	39,9
	60,0	Max. τ	68,5	15,3	99,9		
	120	Max. τ	95,8	12,7	97,4		
	135	Max. τ	131,4	15,1	109,2		

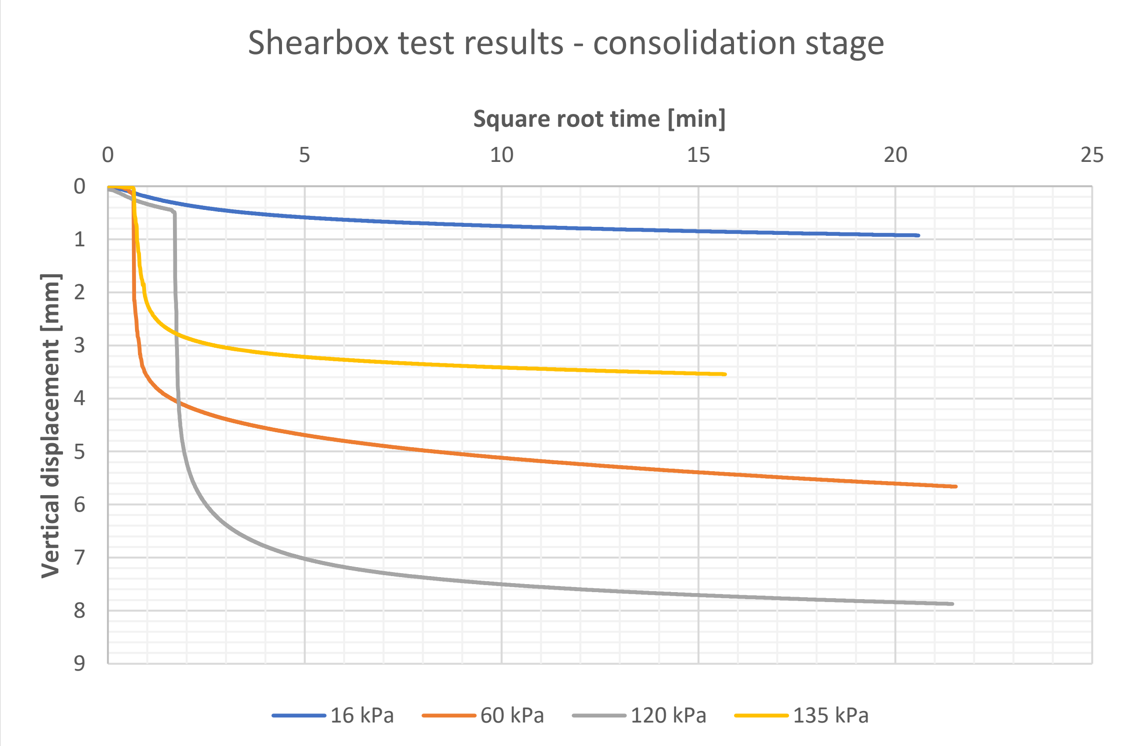


Figure F.4; The measured settlement of the undisturbed peat samples during the consolidation stage of the shear box tests at four different consolidation pressures.

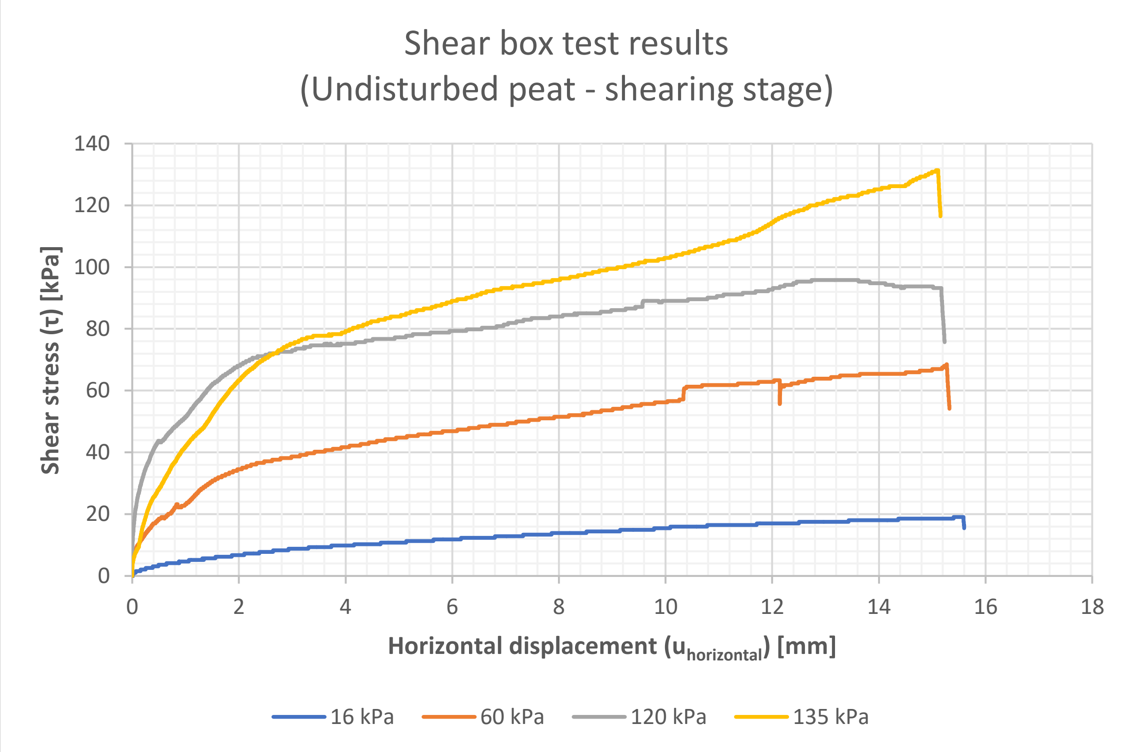


Figure F.5; The shear stress-displacement response of the undisturbed peat samples during the shearing stage of the shear box tests at four different consolidation pressures.

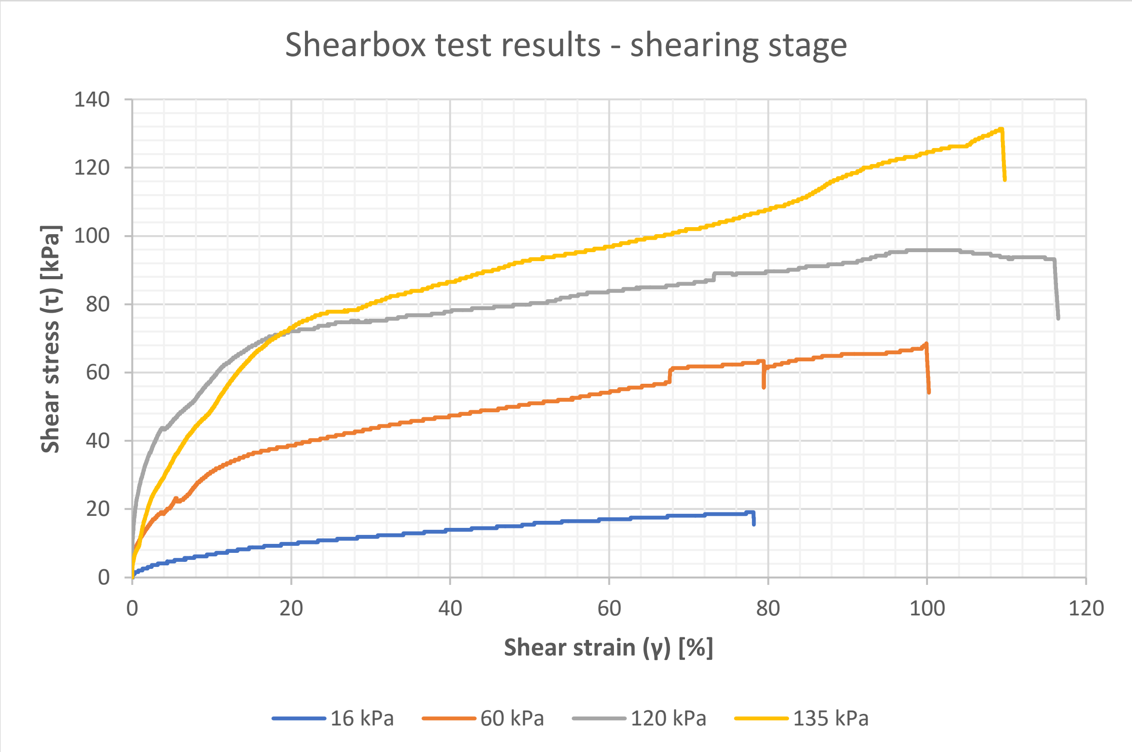


Figure F.6; The shear stress-shear strain response of the undisturbed peat samples during the shearing stage of the shear box tests at four different consolidation pressures.

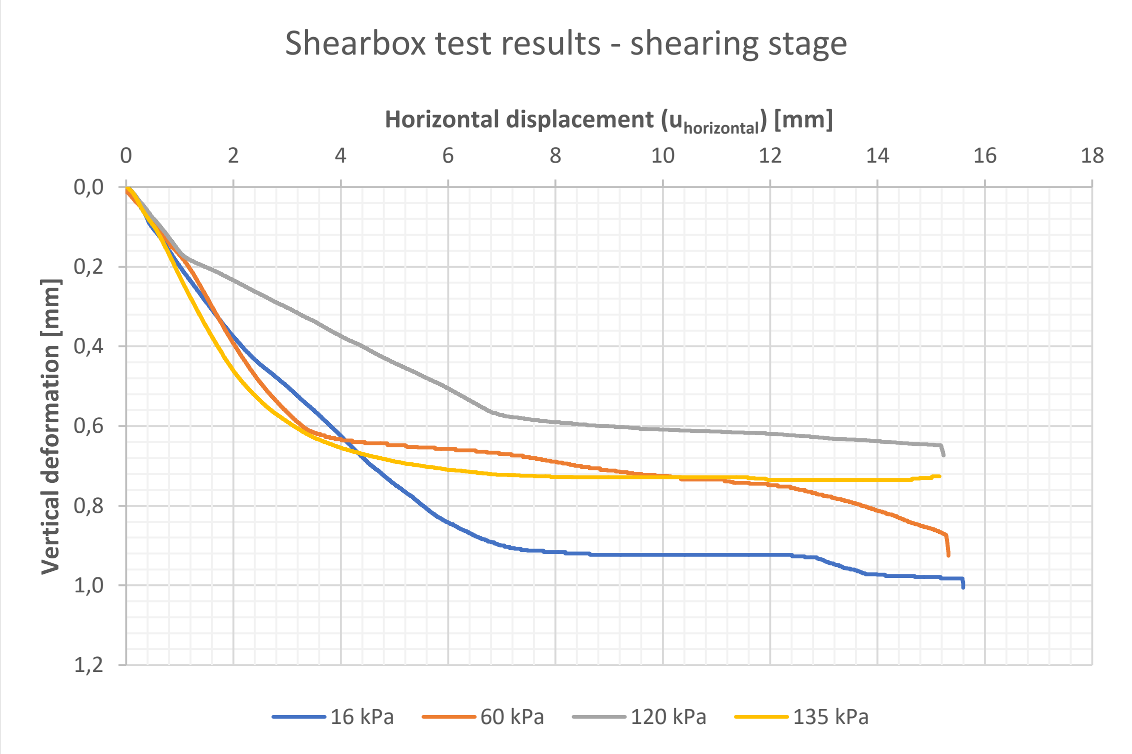


Figure F.7; The measured vertical and horizontal displacements of the undisturbed peat samples during the shearing stage of the shear box tests at four different consolidation pressures.

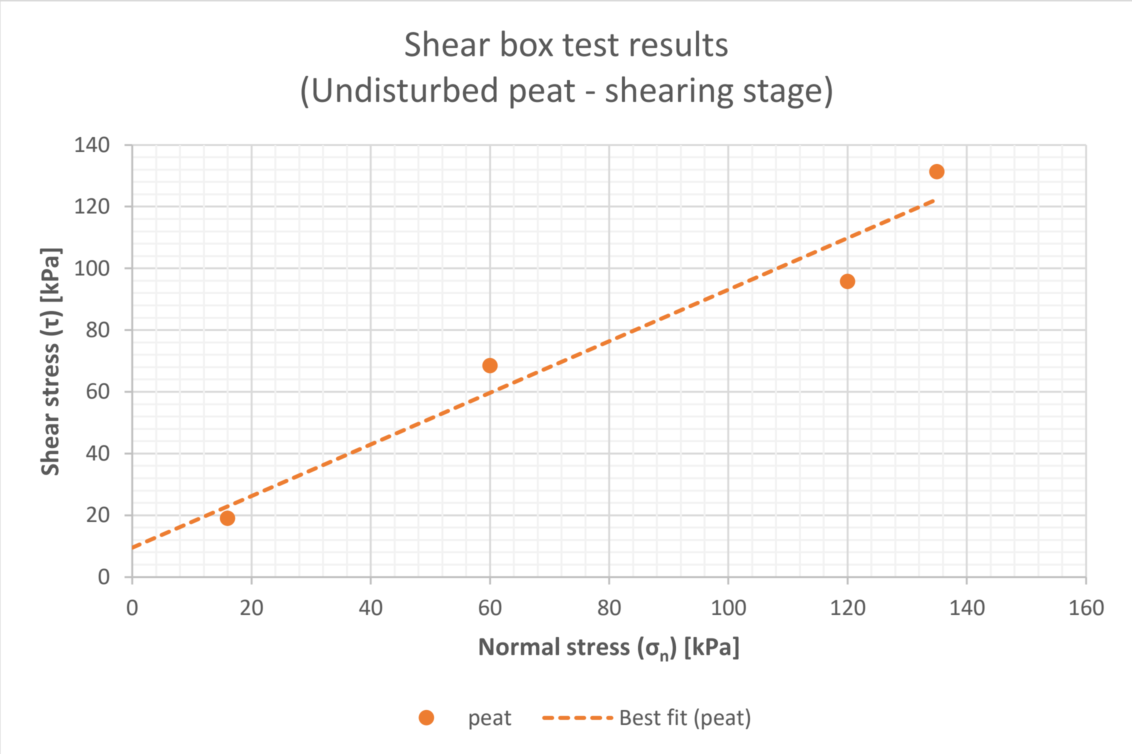


Figure F.8; The Mohr-Coulomb failure lines for the sheared undisturbed peat samples at peak stress.

Table F.11; The derived values of the drained shear strength parameters of the undisturbed peat samples at various strain levels using the measured stresses during the shear box test. N/A = not applicable.

Applied normal stress [kPa]	Parameter	Unit	Value of parameter at shear strain (γ [%])								
			0,2	**0,5**	**1,0**	**2,0**	**5,0**	**10,0**	**15,0**	**20,0**	**25,0**
	ϕ'	[°]	1,48	1,85	1,91	1,66	1,92	3,64	3,45	3,13	3,96
	c'	[kPa]	4,14	5,75	7,72	11,53	16,40	21,47	26,20	28,69	29,61
16,0	τ	[kPa]	1,03	1,55	2,06	3,09	4,64	6,70	8,76	9,79	10,82
60,0	τ	[kPa]	7,21	9,27	11,33	14,94	21,12	30,91	36,06	38,63	41,21
120	τ	[kPa]	15,97	22,15	27,81	35,54	45,33	58,20	67,48	72,11	74,17
135	τ	[kPa]	5,67	7,73	11,33	20,60	34,00	48,93	64,39	73,14	77,78

Applied normal stress [kPa]	Parameter	Unit	Value of parameter at shear strain (γ [%])								
			30,0	**35,0**	**40,0**	**45,0**	**50,0**	**55,0**	**60,0**	**65,0**	**70,0**
	ϕ'	[°]	5,11	6,10	7,14	7,49	8,53	9,24	9,97	10,68	13,04
	c'	[kPa]	29,89	30,37	30,77	31,37	31,76	32,17	32,64	33,05	33,09
16,0	τ	[kPa]	11,85	12,88	13,91	14,42	15,45	16,48	17,00	17,51	18,03
60,0	τ	[kPa]	43,27	45,33	47,39	48,93	50,99	52,02	54,08	56,14	61,81
120	τ	[kPa]	75,20	76,75	77,78	78,81	79,84	81,90	83,96	84,99	86,02
135	τ	[kPa]	80,35	83,44	86,54	89,63	92,72	94,78	96,84	99,41	101,99

Applied normal stress [kPa]	Parameter	Unit	Value of parameter at shear strain (γ [%])									
			75,0	**80,0**	**85,0**	**90,0**	**95,0**	**100,0**	**105,0**	**110,0**	**115,0**	**120,0**
	ϕ'	[°]	12,86	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
	c'	[kPa]	33,98	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
16,0	τ	[kPa]	18,54	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
60,0	τ	[kPa]	62,33	61,81	63,87	65,42	65,93	63,87	N/A	N/A	N/A	N/A
120	τ	[kPa]	89,11	89,63	91,17	92,20	94,78	95,81	95,29	93,75	93,23	N/A
135	τ	[kPa]	104,56	107,65	111,77	117,96	121,56	124,65	126,20	N/A	N/A	N/A

F.1.7 CIU Triaxial tests

Table F.12; Visual description of the undisturbed organic clay soil samples as carried out by the laboratory technician of Fugro NL Land B.V. prior to the isotropically consolidated undrained triaxial test.

Soil type	Visual description
Organic clay	Clay, slightly silty, slightly organic, remnants of reed and peat, grey

Table F.13; Properties of the undisturbed organic clay samples as measured during the CIU triaxial test. NM = not measured.

Soil type	Load step	Taken from tube	$\gamma_{bulk;i}$	$\gamma_{dry;i}$	$w_{nat;i}$	$w_{nat;f}$
		[-]	[kN/m³]	[kN/m³]	[%]	[%]
Organic clay	1	NM	12,9	5,7	127	120
	2	NM	13,0	5,5	136	104
	3	NM	13,1	5,9	123	80,9

Table F.14; Data from the isotropically consolidated undrained triaxial test on the undisturbed organic clay samples after the consolidation stage was finished.

Soil type	Load step	σ_{1C}	σ_{3C}	B	ε_{1C}	$\varepsilon_{vol;C}$	Strain rate
	[kPa]	[kPa]	[kPa]	[-]	[%]	[%]	[%/h]
Stabilised organic clay	1	15,0	15,0	0,98	2,20	4,28	3,0
	2	60,0	60,0	0,98	7,90	19,40	3,0
	3	120	120	0,98	11,40	26,43	2,4

Table F.15; Measured strain and stress parameters from the isotropically consolidated undrained triaxial test at failure of the undisturbed organic clay samples.

Soil type	Load step	Adopted failure criterion	q	p'	s'	t	Δu	σ'_1	σ'_3	ε_f
		[-]	[kPa]	[kPa]	[kPa]	[kPa]	[kPa]	[kPa]	[kPa]	[%]
Stab. organic clay	1	Max. q	23,02	10,30	14,14	11,51	12,32	25,65	2,63	17,99
	2	Max. q	52,62	24,95	36,72	26,31	49,44	63,03	10,41	16,86
	3	Max. q	85,67	63,79	78,07	42,84	84,85	120,9	35,23	10,16

Table F.16; Measured strength parameters from the isotropically consolidated undrained triaxial test at failure of the undisturbed organic clay samples.

Soil type	Load step	Adopted failure criterion	c'	ϕ'	S_u	$E_{u;50}$
			[kPa]	[°]	[kPa]	[MPa]
Stabilised organic clay	1	Max. q	7,18	28,61	11,5	0,6
	2	Max. q			26,3	3,5
	3	Max. q			42,8	4,7

Table F.17; The derived values of the drained shear strength parameters of the undisturbed organic clay samples at various strain levels using the measured stresses during the isotropically consolidated undrained triaxial test.

Load step	Parameter	Unit	Value of parameter at axial strain (ε_{axial} [%])									
			0,2	**0,5**	**1,0**	**2,0**	**3,0**	**4,0**	**5,0**	**6,0**	**7,0**	**8,0**
	ϕ'	[°]	0,00	1,45	1,86	2,34	2,34	2,73	2,91	3,25	3,55	3,98
	c'	[kPa]	0,00	6,40	10,60	14,74	19,91	22,81	25,10	26,60	27,73	28,43
1	s'	[kPa]	15,00	13,79	13,75	13,72	13,40	13,49	13,60	13,79	14,08	14,20
	t	[kPa]	0,00	1,91	3,38	4,78	5,86	6,87	7,60	8,29	8,91	9,45
	q	[kPa]	0,00	3,83	6,77	9,56	11,73	13,74	15,21	16,59	17,82	18,89
	p'	[kPa]	15,00	13,15	12,63	12,13	11,45	11,20	11,07	11,03	11,11	11,05
2	s'	[kPa]	60,00	52,00	51,32	49,93	47,58	46,01	44,81	43,74	42,86	42,10
	t	[kPa]	0,00	9,10	12,97	16,67	19,95	21,86	23,02	23,83	24,41	24,92
	q	[kPa]	0,00	18,21	25,94	33,34	39,91	43,72	46,04	47,67	48,82	49,83
	p'	[kPa]	60,00	48,97	47,00	44,37	40,93	38,72	37,14	35,80	34,72	33,80
3	s'	[kPa]	120,00	103,77	102,02	99,09	94,25	90,66	88,02	85,82	84,03	82,57
	t	[kPa]	0,00	12,23	19,87	26,77	33,63	37,03	39,39	40,77	41,70	42,25
	q	[kPa]	0,00	24,46	39,74	53,53	67,27	74,05	78,79	81,54	83,39	84,50
	p'	[kPa]	120,00	99,69	95,39	90,17	83,04	78,32	74,89	72,23	70,13	68,49

Load step	Parameter	Unit	Value of parameter at axial strain (ε_{axial} [%])									
			9,0	**10,0**	**11,0**	**12,0**	**13,0**	**14,0**	**15,0**	**16,0**	**17,0**	**18,0**
	ϕ'	[°]	4,65	4,80	5,02	5,14	5,26	5,54	5,80	6,05	6,36	NM
	c'	[kPa]	29,47	30,08	30,49	30,92	31,20	31,22	31,09	30,90	30,65	NM
1	s'	[kPa]	14,51	14,51	14,59	14,38	14,17	14,20	14,23	14,14	14,20	14,12
	t	[kPa]	10,34	10,57	10,85	10,87	10,82	11,02	11,15	11,21	11,45	11,50
	q	[kPa]	20,67	21,13	21,71	21,74	21,64	22,04	22,31	22,43	22,91	23,00
	p'	[kPa]	11,06	10,99	10,98	10,76	10,56	10,53	10,52	10,40	10,39	10,29
2	s'	[kPa]	40,82	40,23	39,65	39,02	38,42	37,89	37,36	36,94	36,57	NM
	t	[kPa]	25,56	25,75	25,91	26,00	26,11	26,18	26,20	26,25	26,26	NM
	q	[kPa]	51,12	51,49	51,82	52,01	52,22	52,36	52,41	52,49	52,51	NM
	p'	[kPa]	32,30	31,64	31,02	30,35	29,72	29,17	28,62	28,19	27,82	NM
3	s'	[kPa]	79,75	78,22	76,79	75,42	74,06	72,79	71,34	70,06	68,45	NM
	t	[kPa]	42,71	42,78	42,69	42,53	42,17	41,73	41,01	40,31	39,48	NM
	q	[kPa]	85,42	85,57	85,39	85,06	84,34	83,47	82,03	80,63	78,97	NM
	p'	[kPa]	65,52	63,96	62,55	61,24	60,00	58,88	57,66	56,62	55,29	NM

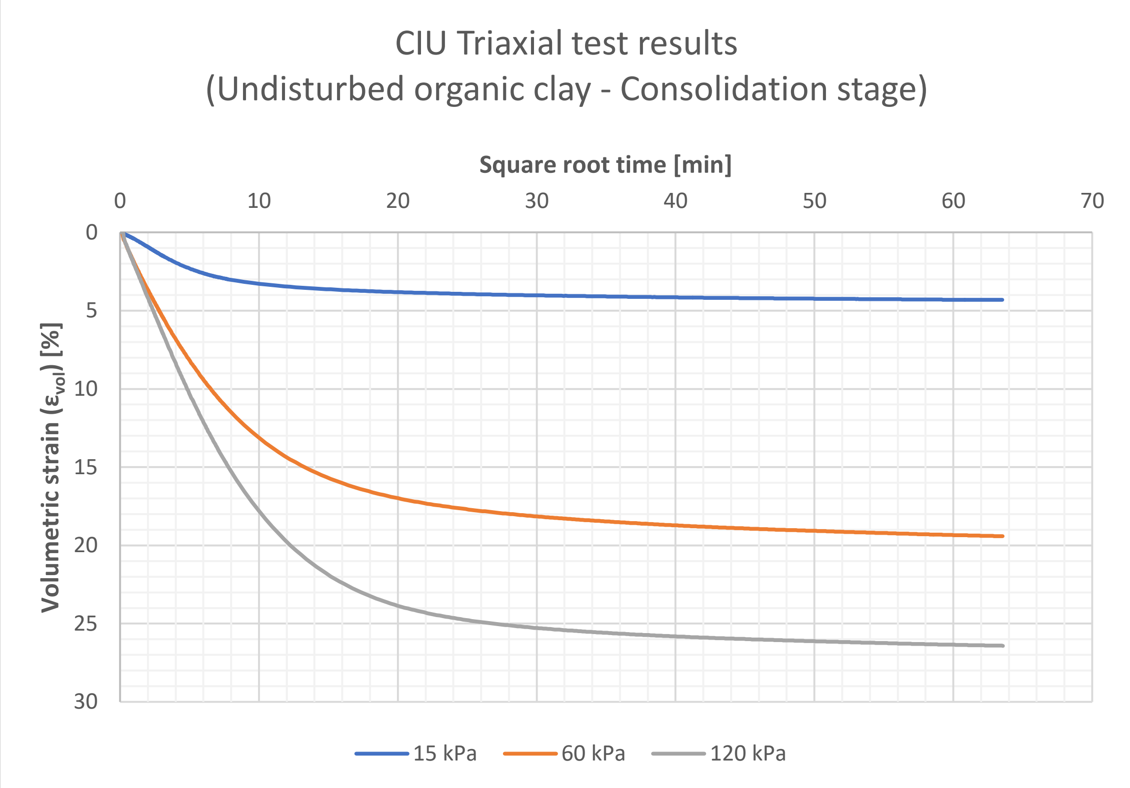


Figure F.9; The measured volumetric strain of the undisturbed organic clay samples during the consolidation stage of the isotropically consolidated undrained triaxial test for three different consolidation stresses.

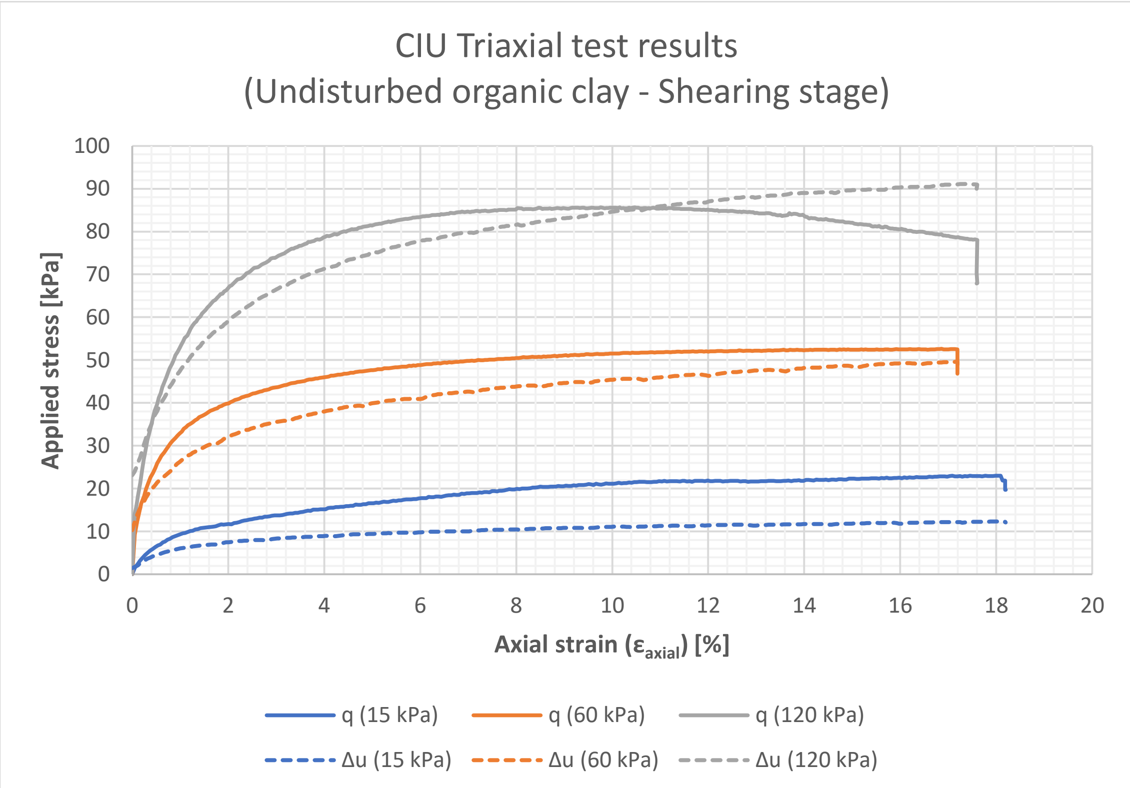


Figure F.10; The measured stress-strain responses of the undisturbed organic clay samples during the shearing stage of the isotropically consolidated undrained triaxial test at three different consolidation stresses. The deviator stress (q) and the change in pore pressure (Δu) during the shearing stage are indicated in the graph.

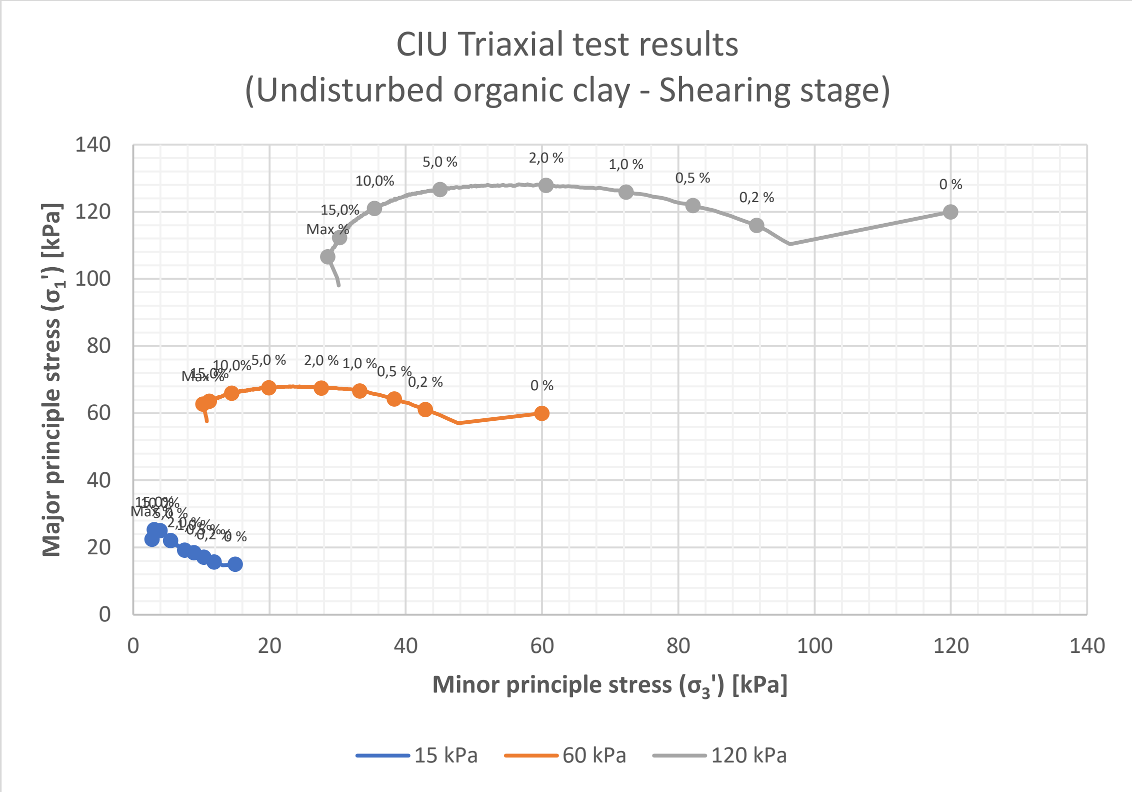


Figure F.11; The measured stress paths of the undisturbed organic clay samples during the shearing stage of the isotropically consolidated undrained triaxial test at three different consolidation stresses. The axial strains during shearing are indicated in the graph.

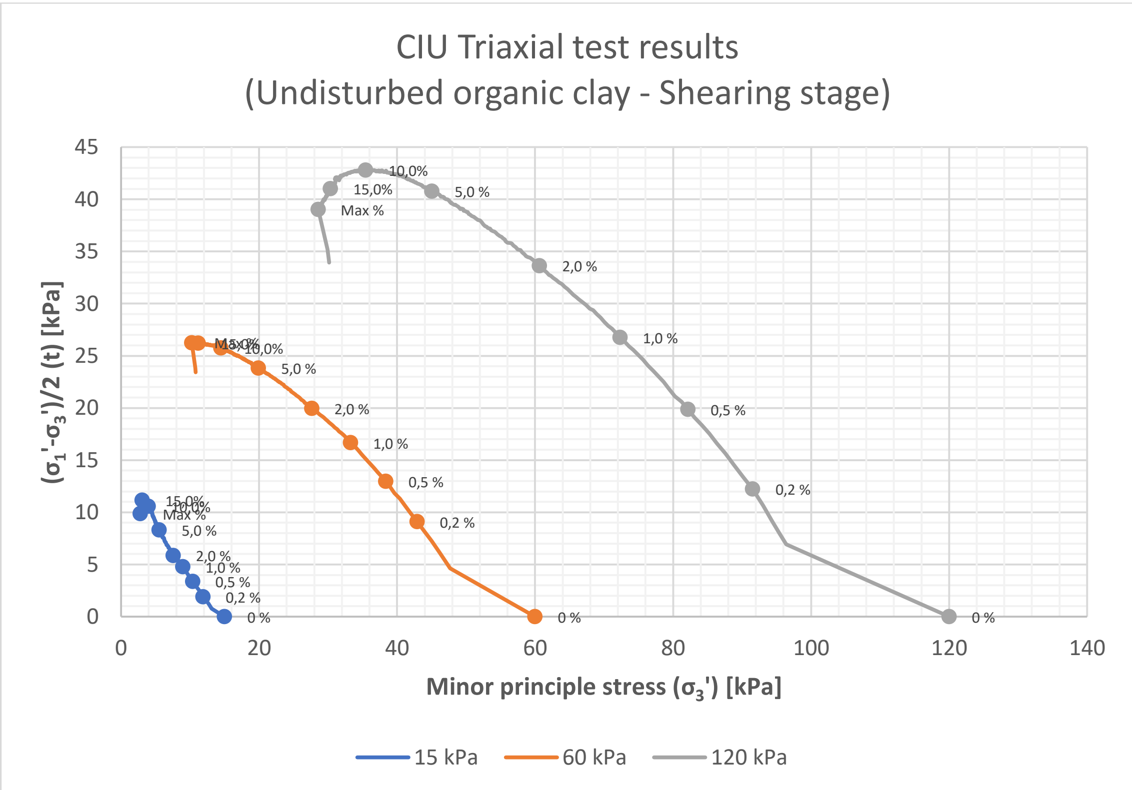


Figure F.12; The measured stress paths of the undisturbed organic clay samples during the shearing stage of the isotropically consolidated undrained triaxial test at three different consolidation stresses. The axial strains during shearing are indicated in the graph.

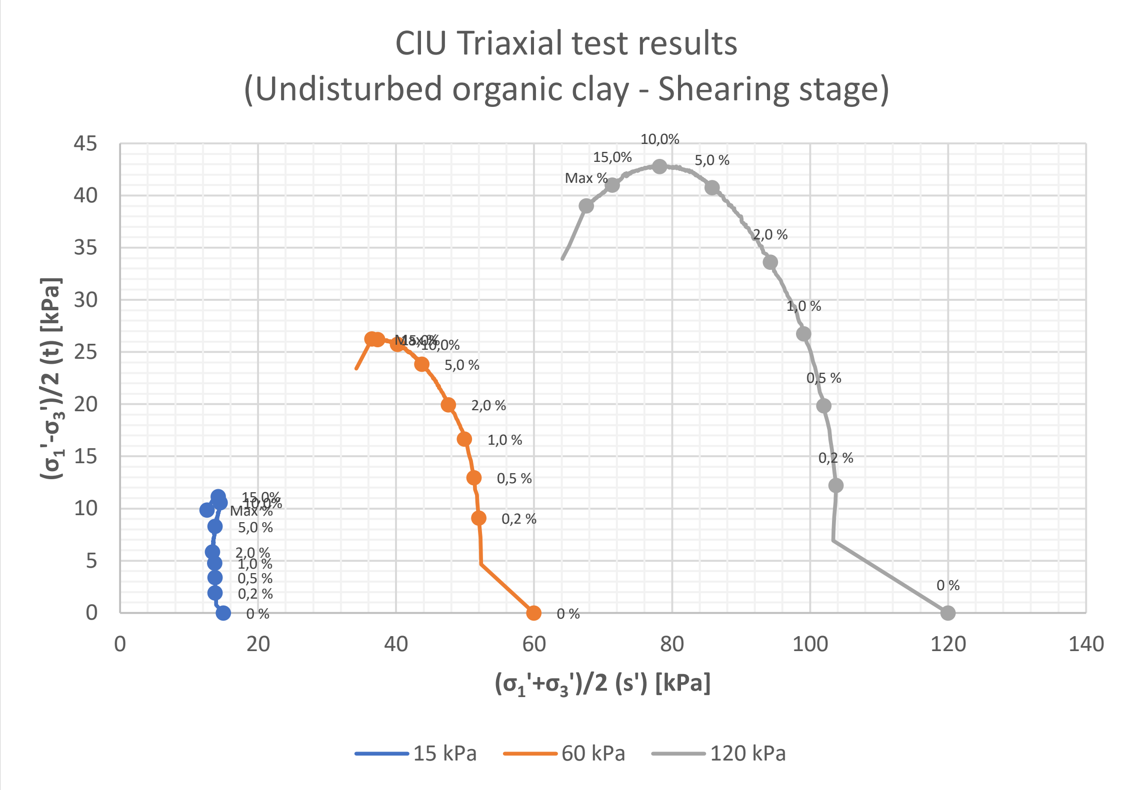


Figure F.13; The measured stress paths of the undisturbed organic clay samples during the shearing stage of the isotropically consolidated undrained triaxial test at three different consolidation stresses. The axial strains during shearing are indicated in the graph.

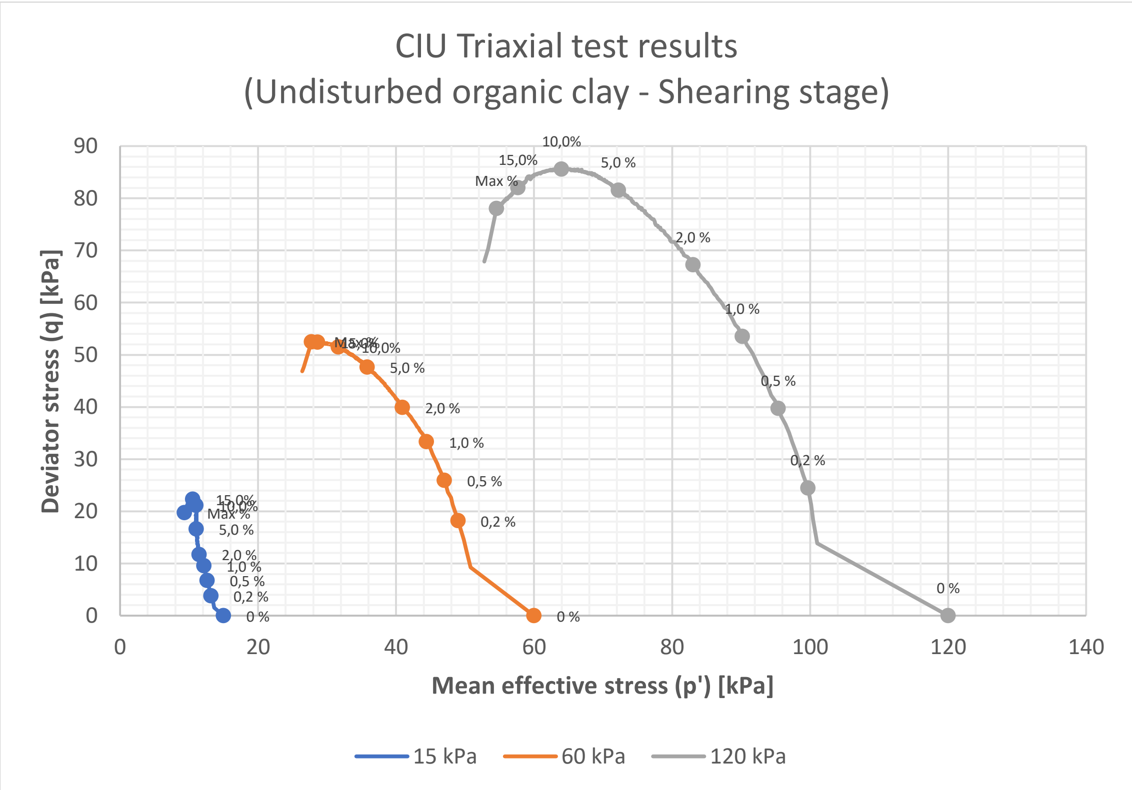


Figure F.14; The measured stress paths of the undisturbed organic clay samples during the shearing stage of the isotropically consolidated undrained triaxial test at three different consolidation stresses. The axial strains during shearing are indicated in the graph.

Applied normal stress [kPa]		
15,0		
60,0		
120		

Figure F.15; Photographs of the undisturbed organic clay samples after failure in the isotropically consolidated undrained triaxial test at three different consolidation stresses.

F.1.8 Oedometer tests

Table F.18; Visual description of the undisturbed peat and organic clay soil samples as carried out by the laboratory technician of Fugro NL Land B.V. prior to the oedometer test.

Soil type	Visual description
Peat	Peat, poor in minerals, brown
Organic clay	Clay, strongly silty, moderately organic, grey

Table F.19; Initial properties of the undisturbed peat and organic clay samples as measured before the oedometer tests. NM = not measured.

Soil type	Sample code	Taken from tube	d	h	$\gamma_{bulk;i}$	$\gamma_{dry;i}$	$w_{nat;i}$	e_0	ρ_s (measured)
	[-]	[-]	[mm]	[mm]	[kN/m^3]	[kN/m^3]	[%]	[-]	[Mg/m^3]
Peat	M1	NM	50,0	18,7	9,0	2,0	354	7,750	1,77
	M2	NM	50,1	19,0	10,0	2,0	396	7,600	1,77
	M3	NM	50,1	18,5	9,2	1,7	434	9,100	1,77
Organic clay	M1	NM	50,1	19,0	13,0	5,6	133	3,392	2,50
	M2	NM	50,1	18,7	12,9	5,7	127	3,321	2,50
	M3	NM	50,1	19,1	13,2	5,9	123	3,156	2,50

Table F.20; Applied loading steps in the oedometer tests on the undisturbed peat and organic clay samples.

Soil type	Loading steps [kPa]								
	1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**
Peat	4	7	15	30	60	120	60	120	240
Org. clay	4	7	15	30	60	120	60	120	240

Table F.21; Derived compression parameters from the results of the oedometer tests on the undisturbed peat and organic clay samples.

Applied method	Value of parameter						
	Soil type	**Sample code**	$CR\ (<\sigma'_p)$	$CR\ (>\sigma'_p)$	SR **(step 6-7)**	RR **(step 7-8)**	σ'_p
	[-]	**[-]**	**[-]**	**[-]**	**[-]**	**[-]**	**[kPa]**
Anglo-Saxon method (linear strain)	**Peat**	**M1**	0,1017	0,3816	0,0237	0,0625	21
		M2	0,1127	0,3776	0,0193	0,0570	22
		M3	0,1273	0,4191	0,0218	0,0651	20
	Organic clay	**M1**	0,0196	0,3517	0,0224	0,0456	25
		M2	0,0283	0,3405	0,0214	0,0435	23
		M3	0,0321	0,4203	0,0214	0,0497	30
	Soil type	**Sample code**	$C_c\ (<\sigma'_p)$	$C_c\ (>\sigma'_p)$	C_{sw} **(step 6-7)**	C_r **(step 7-8)**	σ'_p
	[-]	**[-]**	**[-]**	**[-]**	**[-]**	**[-]**	**[kPa]**
Anglo-Saxon method (void ratio)	**Peat**	**M1**	0,8893	3,3382	0,2072	0,5467	21
		M2	0,9687	3,2472	0,1658	0,4898	22
		M3	1,2859	4,2342	0,2199	0,6579	20
	Organic clay	**M1**	0,0862	1,5447	0,0985	0,2001	25
		M2	0,1223	1,4711	0,0924	0,1880	23
		M3	0,1336	1,7469	0,0893	0,2067	30
	Soil type	**Sample code**	C_p	C_s	C'_p	C'_s	σ'_p
	[-]	**[-]**	**[-]**	**[-]**	**[-]**	**[-]**	**[kPa]**
Koppejan method	**Peat**	**M1**	19,7	54,3	6,6	27,7	22
		M2	19,9	53,2	6,7	36,3	24
		M3	16,4	44,2	5,9	25,0	21
	Organic clay	**M1**	56,1	258,3	7,3	41,4	22
		M2	44,7	232,0	7,3	37,7	22
		M3	42,7	176,8	6,7	37,7	24

Table F.22; Derived parameters from the settlement analyses of the oedometer test on undisturbed organic clay sample M1.

Load step	Load	Taylor method			Casagrande method				
		$c_{v;10}$	m_v	$k_{v;10}$	$c_{v;10}$	m_v	$k_{v;10}$	$C_{\alpha;NEN}$	$C_{\alpha;HEAD}$
[-]	**[kPa]**	**[m²/s]**	**[m²/ MN]**	**[m/s]**	**[m²/s]**	**[m²/ MN]**	**[m/s]**	**[-]**	**[-]**
1	**4**	-	-	-	-	-	-	-	-
2	**7**	-	-	-	-	-	-	-	-
3	**15**	$1,4\cdot10^{-7}$	$2,7\cdot10^{0}$	$3,7\cdot10^{-9}$	N/A	$2,7\cdot10^{0}$	N/A	$5,4\cdot10^{-3}$	$5,3\cdot10^{-3}$
4	**30**	$6,5\cdot10^{-8}$	$3,5\cdot10^{0}$	$2,3\cdot10^{-9}$	$4,8\cdot10^{-8}$	$3,5\cdot10^{0}$	$1,7\cdot10^{-9}$	$1,2\cdot10^{-2}$	$1,2\cdot10^{-2}$
5	**60**	$3,2\cdot10^{-8}$	$3,2\cdot10^{0}$	$1,0\cdot10^{-9}$	$2,1\cdot10^{-8}$	$3,2\cdot10^{0}$	$6,9\cdot10^{-10}$	$2,0\cdot10^{-2}$	$1,8\cdot10^{-2}$
6	**120**	$2,0\cdot10^{-8}$	$2,1\cdot10^{0}$	$4,2\cdot10^{-10}$	$1,5\cdot10^{-8}$	$2,1\cdot10^{0}$	$3,2\cdot10^{-10}$	$2,2\cdot10^{-2}$	$1,8\cdot10^{-2}$
7	**60**	-	-	-	-	-	-	-	-
8	**120**	-	-	-	-	-	-	-	-
9	**240**	$1,4\cdot10^{-8}$	$1,0\cdot10^{0}$	$1,5\cdot10^{-10}$	$1,1\cdot10^{-8}$	$1,0\cdot10^{0}$	$1,1\cdot10^{-10}$	$2,1\cdot10^{-2}$	$1,5\cdot10^{-2}$

Table F.23; Derived parameters from the settlement analyses of the oedometer test on *undisturbed organic clay sample M2.*

Load step	Load	Taylor method			Casagrande method				
		$c_{v;10}$	m_v	$k_{v;10}$	$c_{v;10}$	m_v	$k_{v;10}$	$C_{\alpha;NEN}$	$C_{\alpha;HEAD}$
[-]	[kPa]	[m²/s]	[m²/MN]	[m/s]	[m²/s]	[m²/MN]	[m/s]	[-]	[-]
1	4	-	-	-	-	-	-	-	-
2	7	-	-	-	-	-	-	-	-
3	15	$1,2\cdot10^{-7}$	$3,2\cdot10^{0}$	$3,7\cdot10^{-9}$	N/A	$3,2\cdot10^{0}$	N/A	$5,6\cdot10^{-3}$	$5,5\cdot10^{-3}$
4	30	$4,7\cdot10^{-8}$	$3,7\cdot10^{0}$	$1,7\cdot10^{-9}$	$2,6\cdot10^{-8}$	$3,7\cdot10^{0}$	$9,4\cdot10^{-10}$	$1,3\cdot10^{-2}$	$1,3\cdot10^{-2}$
5	60	$2,2\cdot10^{-8}$	$3,3\cdot10^{0}$	$7,5\cdot10^{-10}$	$1,4\cdot10^{-8}$	$3,3\cdot10^{0}$	$4,8\cdot10^{-10}$	$2,1\cdot10^{-2}$	$1,9\cdot10^{-2}$
6	120	$1,0\cdot10^{-8}$	$2,1\cdot10^{0}$	$2,1\cdot10^{-10}$	$8,1\cdot10^{-9}$	$2,1\cdot10^{0}$	$1,7\cdot10^{-10}$	$2,1\cdot10^{-2}$	$1,7\cdot10^{-2}$
7	60	-	-	-	-	-	-	-	-
8	120	-	-	-	-	-	-	-	-
9	240	$8,0\cdot10^{-9}$	$9,8\cdot10^{-1}$	$7,9\cdot10^{-11}$	$6,1\cdot10^{-9}$	$9,8\cdot10^{-1}$	$6,0\cdot10^{-11}$	$2,1\cdot10^{-2}$	$1,5\cdot10^{-2}$

Table F.24; Derived parameters from the settlement analyses of the oedometer test on *undisturbed organic clay sample M3.*

Load step	Load	Taylor method			Casagrande method				
		$c_{v;10}$	m_v	$k_{v;10}$	$c_{v;10}$	m_v	$k_{v;10}$	$C_{\alpha;NEN}$	$C_{\alpha;HEAD}$
[-]	[kPa]	[m²/s]	[m²/MN]	[m/s]	[m²/s]	[m²/MN]	[m/s]	[-]	[-]
1	4	-	-	-	-	-	-	-	-
2	7	-	-	-	-	-	-	-	-
3	15	$7,6\cdot10^{-8}$	$3,3\cdot10^{0}$	$2,5\cdot10^{-9}$	N/A	$3,3\cdot10^{0}$	N/A	$6,8\cdot10^{-3}$	$6,7\cdot10^{-3}$
4	30	$3,5\cdot10^{-8}$	$3,5\cdot10^{0}$	$1,2\cdot10^{-9}$	$2,5\cdot10^{-8}$	$3,5\cdot10^{0}$	$8,7\cdot10^{-10}$	$1,3\cdot10^{-2}$	$1,3\cdot10^{-2}$
5	60	$1,6\cdot10^{-8}$	$3,1\cdot10^{0}$	$4,8\cdot10^{-10}$	$1,1\cdot10^{-8}$	$3,1\cdot10^{0}$	$3,4\cdot10^{-10}$	$2,1\cdot10^{-2}$	$1,9\cdot10^{-2}$
6	120	$6,9\cdot10^{-9}$	$2,5\cdot10^{0}$	$1,8\cdot10^{-10}$	$7,5\cdot10^{-9}$	$2,5\cdot10^{0}$	$1,9\cdot10^{-10}$	$2,1\cdot10^{-2}$	$1,8\cdot10^{-2}$
7	60	-	-	-	-	-	-	-	-
8	120	-	-	-	-	-	-	-	-
9	240	$6,0\cdot10^{-9}$	$9,7\cdot10^{-1}$	$5,8\cdot10^{-11}$	$4,8\cdot10^{-9}$	$9,7\cdot10^{-1}$	$4,6\cdot10^{-11}$	$2,1\cdot10^{-2}$	$1,4\cdot10^{-2}$

Table F.25; Derived parameters from the settlement analyses of the oedometer test on *undisturbed peat sample M1.*

Load step	Load	Taylor method			Casagrande method				
		$c_{v;10}$	m_v	$k_{v;10}$	$c_{v;10}$	m_v	$k_{v;10}$	$C_{\alpha;NEN}$	$C_{\alpha;HEAD}$
[-]	[kPa]	[m²/s]	[m²/MN]	[m/s]	[m²/s]	[m²/MN]	[m/s]	[-]	[-]
1	4	-	-	-	-	-	-	-	-
2	7	-	-	-	-	-	-	-	-
3	15	$5,0\cdot10^{-7}$	$6,9\cdot10^{0}$	$3,5\cdot10^{-8}$	N/A	$6,9\cdot10^{0}$	N/A	$2,6\cdot10^{-2}$	$2,5\cdot10^{-2}$
4	30	$4,0\cdot10^{-7}$	$5,2\cdot10^{0}$	$2,1\cdot10^{-8}$	N/A	$5,2\cdot10^{0}$	N/A	$3,8\cdot10^{-2}$	$3,4\cdot10^{-2}$
5	60	$3,0\cdot10^{-7}$	$4,3\cdot10^{0}$	$1,3\cdot10^{-8}$	N/A	$4,3\cdot10^{0}$	N/A	$3,8\cdot10^{-2}$	$3,2\cdot10^{-2}$
6	120	$1,7\cdot10^{-7}$	$2,7\cdot10^{0}$	$3,5\cdot10^{-9}$	N/A	$2,7\cdot10^{0}$	N/A	$4,5\cdot10^{-2}$	$3,2\cdot10^{-2}$
7	60	-	-	-	-	-	-	-	-
8	120	-	-	-	-	-	-	-	-
9	240	$4,2\cdot10^{-8}$	$1,3\cdot10^{0}$	$5,5\cdot10^{-10}$	N/A	$1,3\cdot10^{0}$	N/A	$5,3\cdot10^{-2}$	$3,1\cdot10^{-2}$

Table F.26; Derived parameters from the settlement analyses of the oedometer test on undisturbed peat sample M2.

Load step	Load	Taylor method			Casagrande method				
		$c_{v;10}$	m_v	$k_{v;10}$	$c_{v;10}$	m_v	$k_{v;10}$	$C_{\alpha;NEN}$	$C_{\alpha;HEAD}$
[-]	[kPa]	[m²/s]	[m²/MN]	[m/s]	[m²/s]	[m²/MN]	[m/s]	[-]	[-]
1	**4**	-	-	-	-	-	-	-	-
2	**7**	-	-	-	-	-	-	-	-
3	**15**	$4{,}7\cdot10^{-7}$	$6{,}6\cdot10^{0}$	$3{,}1\cdot10^{-8}$	N/A	$6{,}6\cdot10^{0}$	N/A	$2{,}7\cdot10^{-2}$	$2{,}6\cdot10^{-2}$
4	**30**	$3{,}2\cdot10^{-7}$	$4{,}8\cdot10^{0}$	$1{,}5\cdot10^{-8}$	N/A	$4{,}8\cdot10^{0}$	N/A	$3{,}1\cdot10^{-2}$	$2{,}7\cdot10^{-2}$
5	**60**	$2{,}2\cdot10^{-7}$	$4{,}2\cdot10^{0}$	$9{,}4\cdot10^{-9}$	N/A	$4{,}2\cdot10^{0}$	N/A	$3{,}9\cdot10^{-2}$	$3{,}2\cdot10^{-2}$
6	**120**	$1{,}4\cdot10^{-7}$	$2{,}6\cdot10^{0}$	$3{,}7\cdot10^{-9}$	N/A	$2{,}6\cdot10^{0}$	N/A	$4{,}1\cdot10^{-2}$	$3{,}0\cdot10^{-2}$
7	**60**	-	-	-	-	-	-	-	-
8	**120**	-	-	-	-	-	-	-	-
9	**240**	$9{,}4\cdot10^{-8}$	$1{,}4\cdot10^{0}$	$1{,}3\cdot10^{-9}$	N/A	$1{,}4\cdot10^{0}$	N/A	$5{,}1\cdot10^{-2}$	$3{,}1\cdot10^{-2}$

Table F.27; Derived parameters from the settlement analyses of the oedometer test on undisturbed peat sample M3.

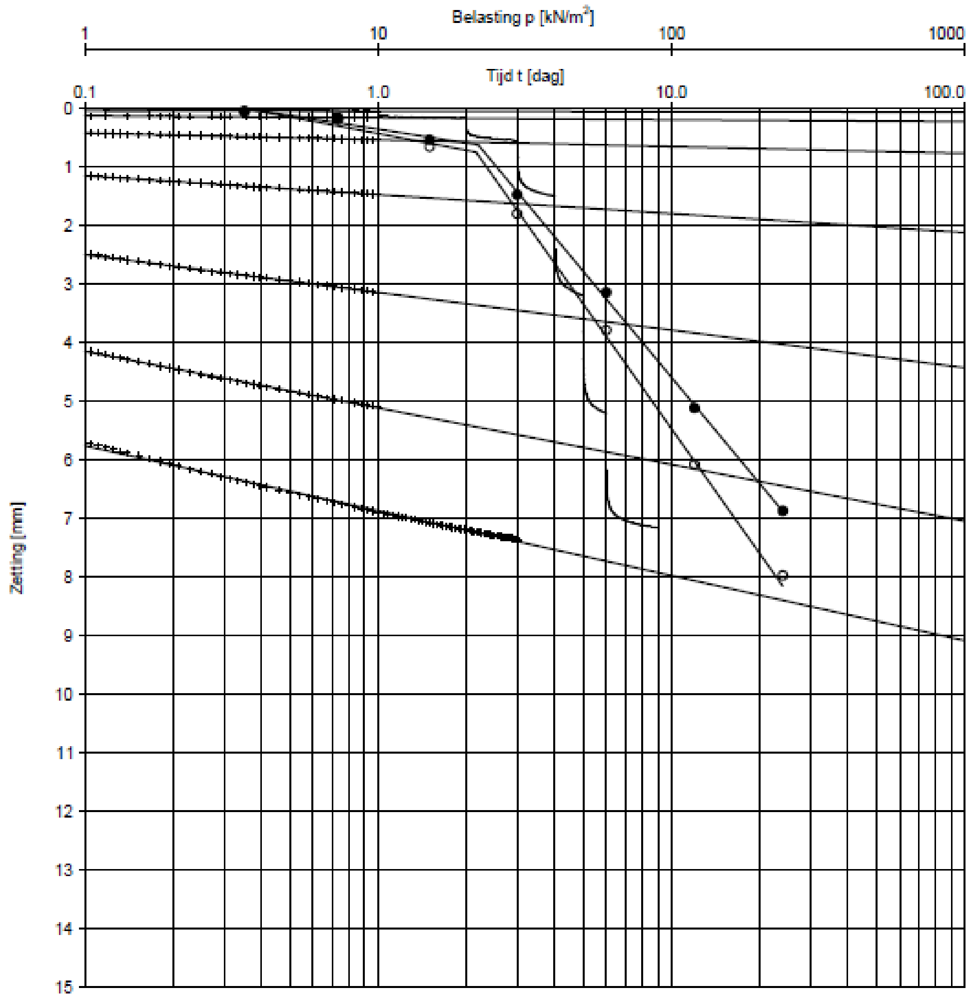
Load step	Load	Taylor method			Casagrande method				
		$c_{v;10}$	m_v	$k_{v;10}$	$c_{v;10}$	m_v	$k_{v;10}$	$C_{\alpha;NEN}$	$C_{\alpha;HEAD}$
[-]	[kPa]	[m²/s]	[m²/MN]	[m/s]	[m²/s]	[m²/MN]	[m/s]	[-]	[-]
1	**4**	-	-	-	-	-	-	-	-
2	**7**	-	-	-	-	-	-	-	-
3	**15**	$2{,}9\cdot10^{-7}$	$8{,}4\cdot10^{0}$	$2{,}4\cdot10^{-8}$	N/A	$8{,}4\cdot10^{0}$	N/A	$3{,}5\cdot10^{-2}$	$3{,}3\cdot10^{-2}$
4	**30**	$1{,}2\cdot10^{-7}$	$6{,}4\cdot10^{0}$	$7{,}9\cdot10^{-9}$	N/A	$6{,}4\cdot10^{0}$	N/A	$4{,}1\cdot10^{-2}$	$3{,}6\cdot10^{-2}$
5	**60**	$1{,}1\cdot10^{-7}$	$5{,}2\cdot10^{0}$	$6{,}0\cdot10^{-9}$	N/A	$5{,}2\cdot10^{0}$	N/A	$5{,}0\cdot10^{-2}$	$3{,}9\cdot10^{-2}$
6	**120**	$4{,}4\cdot10^{-8}$	$3{,}2\cdot10^{0}$	$1{,}4\cdot10^{-9}$	N/A	$3{,}2\cdot10^{0}$	N/A	$5{,}1\cdot10^{-2}$	$3{,}4\cdot10^{-2}$
7	**60**	-	-	-	-	-	-	-	-
8	**120**	-	-	-	-	-	-	-	-
9	**240**	$4{,}3\cdot10^{-8}$	$1{,}5\cdot10^{0}$	$6{,}4\cdot10^{-10}$	N/A	$1{,}5\cdot10^{0}$	N/A	$6{,}3\cdot10^{-2}$	$3{,}3\cdot10^{-2}$

Table F.28; Derived values of the oedometer stiffness from the results of the oedometer tests on the undisturbed peat and organic clay samples.

Soil type	Sample code	Oedometer stiffness (E_{oed}) [MN/m²] at loading step								
[-]	[-]	1	2	3	4	5	6	7	8	9
Peat	**M1**	-	-	0,1	0,2	0,2	0,4	-	-	0,8
	M2	-	-	0,2	0,2	0,2	0,4	-	-	0,7
	M3	-	-	0,1	0,2	0,2	0,3	-	-	0,7
Organic clay	**M1**	-	-	0,4	0,3	0,3	0,5	-	-	1,0
	M2	-	-	0,3	0,3	0,3	0,5	-	-	1,0
	M3	-	-	0,3	0,3	0,3	0,4	-	-	1,0

Table F.29; The average value of the hydraulic conductivity of the undisturbed peat and organic clay samples at the different load stages of the oedometer test.

Soil type	Method	Hydraulic conductivity ($k_{v;10}$) [m/s] at loading step								
[-]	[-]	1	2	3	4	5	6	7	8	9
Peat	Taylor	N/A	N/A	$3{,}00\cdot10^{-8}$	$1{,}46\cdot10^{-8}$	$9{,}47\cdot10^{-9}$	$2{,}87\cdot10^{-9}$	N/A	N/A	$8{,}30\cdot10^{-10}$
	Cassagrande	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Organic clay	Taylor	N/A	N/A	$3{,}30\cdot10^{-9}$	$1{,}73\cdot10^{-9}$	$7{,}43\cdot10^{-10}$	$2{,}70\cdot10^{-10}$	N/A	N/A	$9{,}57\cdot10^{-11}$
	Cassagrande	N/A	N/A	N/A	$1{,}17\cdot10^{-9}$	$5{,}03\cdot10^{-10}$	$2{,}27\cdot10^{-10}$	N/A	N/A	$7{,}20\cdot10^{-11}$



Boring	: B2	C	=	30.0	
Monster	: Klei M1	C'	=	4.3	
Diepte	: 0.00 m t.o.v. maaiveld	σ'_p	=	22	kN/m²
Grondsoort	: KLEI, sterk siltig, matig humeus, grijs				
		C_p	=	56.1	
		C_s	=	258.3	
γ	: 13.0 kN/m³	C'_p	=	7.3	
γ_{dr}	: 5.6 kN/m³	C'_s	=	41.4	
w	: 132.7 % [m/m]				
		H_i	=	19.0	mm
		D	=	50.1	mm

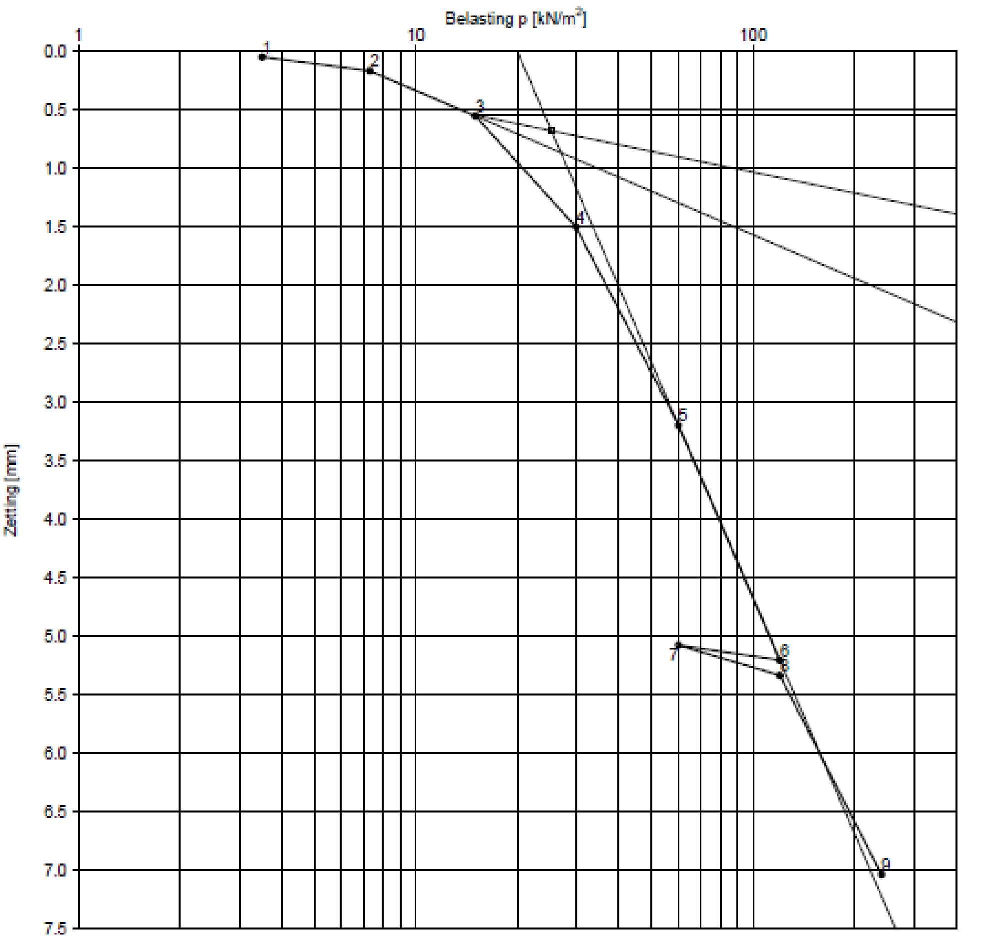
Uitgevoerd conform NEN 5118: 1991 / A1: 1997 nl.

Samendrukkingsproef methode KOPPEJAN

Opdr. 1718-0800-000

Afstudeeropdracht: Onderzoek Massastabilisatie 2018-2019

Figure F.16; Compression-stress plot from the results of the oedometer test on <u>undisturbed organic clay sample M1</u> for the Koppejan method.



Boring	: B2		Grensspanning			
Monster	: Klei M1		σ'_p	=	25	kN/m²
Diepte	: 0.00 m t.o.v. maaiveld		Primaire samendrukkings getal CR			
Grondsoort	: KLEI, sterk siltig, matig humeus, grijs		belasting < σ'_p	=	0.0196	
			belasting > σ'_p	=	0.3517	
			SR Zwelgetal 6-7	=	0.0224	
			RR Herbelastingsgetal 7-8	=	0.0456	
γ	: 13.0	kN/m³	H_0	=	19.0	mm
γ_{dr}	: 5.6	kN/m³	D	=	50.1	mm
w	: 132.7	% [m/m]				

Fugro NL Land B.V. Arnhem OED 6.05.0012 /1353.07P KLEI M1.OED Opg.:NW dd:29-Jan-2019

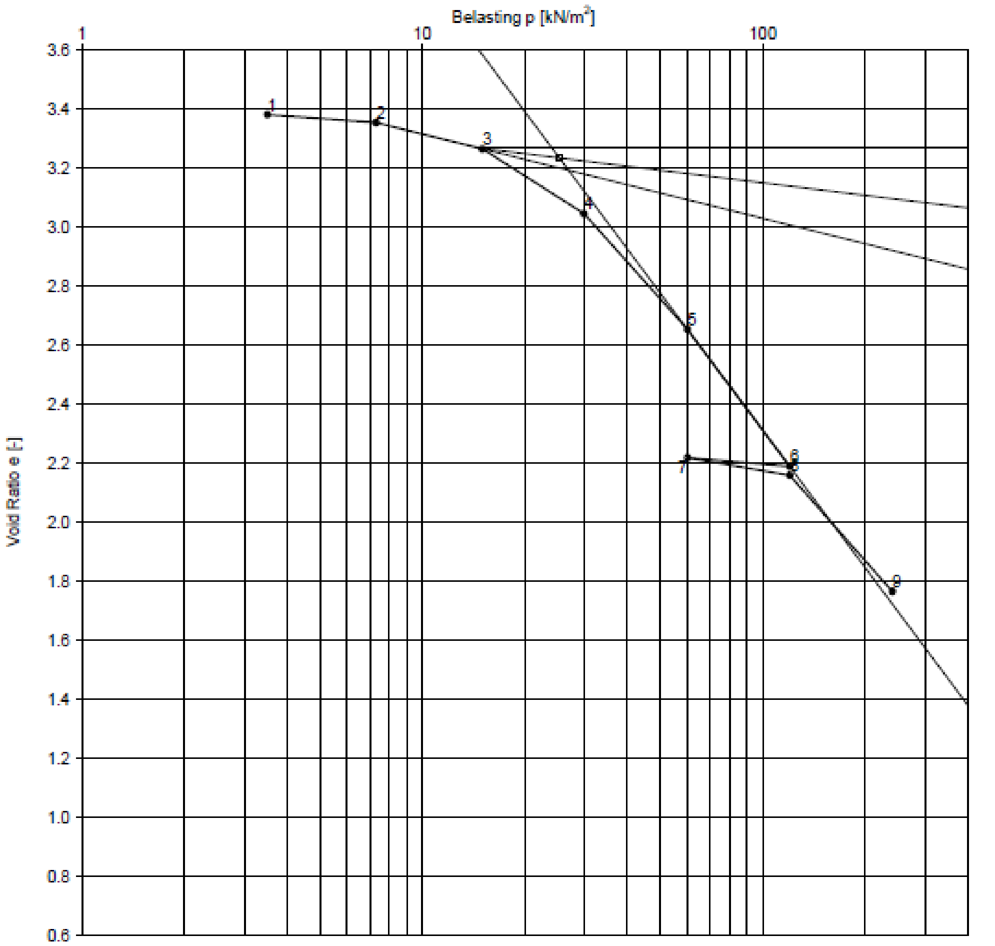
Uitgevoerd conform NEN 5118: 1991 / A1: 1997 nl.

Samendrukkingsproef resultaten z-log p

Opdr. 1718-0800-000

Afstudeeropdracht: Onderzoek Massastabilisatie 2018-2019

Figure F.17; Compression-stress plot from the results of the oedometer test on undisturbed organic clay sample M1 for the Anglo-Saxon method (linear strain).



Boring	: B2	Grensspanning			
Monster	: Klei M1	σ'_p	=	25	kN/m²
Diepte	: 0.00 m t.o.v. maaiveld	Primaire samendrukkings index C_c			
Grondsoort	: KLEI, sterk siltig, matig humeus, grijs	belasting < σ'_p	=	0.0862	
		belasting > σ'_p	=	1.5447	
		C_{sw} Zwelindex 6-7	=	0.0985	
		C_r Herbelastingsindex 7-8	=	0.2001	
γ	: 13.0 kN/m³	H_0	=	19.0	mm
γ_{dr}	: 5.6 kN/m³	D	=	50.1	mm
w	: 132.7 % [m/m]				
Void ratio e_0	: 3.392 (bij ρ_s = 2.50 t/m³)				

Uitgevoerd conform NEN 5118: 1991 / A1: 1997 nl.

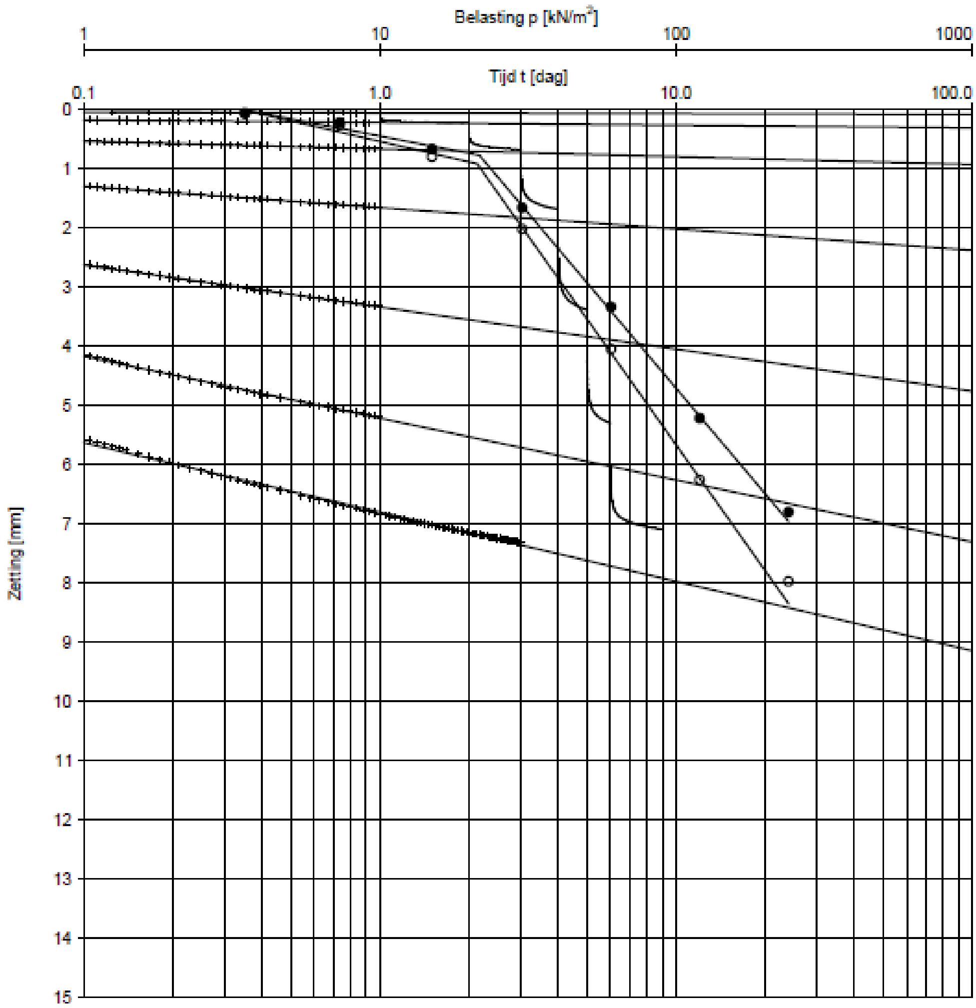
Fugro NL Land B.V. Arnhem OED 6.05.0012 /1353:13\P:KLIB M1.OED Opg: NW dd:29-Jan-2019

Samendrukkingsproef resultaten e-log p

Opdr. 1718-0800-000

Afstudeeropdracht: Onderzoek Massastabilisatie 2018-2019

Figure F.18; Compression-stress plot from the results of the oedometer test on <u>undisturbed organic clay sample M1</u> for the Anglo-Saxon method (void ratio).



Boring	: B2	C =	25.3
Monster	: Klei M2	C' =	4.1
Diepte	: 0.00 m t.o.v. maaiveld	σ'_p =	22 kN/m²
Grondsoort	: KLEI, sterk siltig, matig humeus, grijs		
		C_p =	44.7
		C_s =	232.0
γ	: 12.9 kN/m³	C'_p =	7.3
γ_{dr}	: 5.7 kN/m³	C'_s =	37.7
w	: 127.1 % [m/m]		
		H_i =	18.7 mm
		D =	50.1 mm

Uitgevoerd conform NEN 5118: 1991 / A1: 1997 nl.

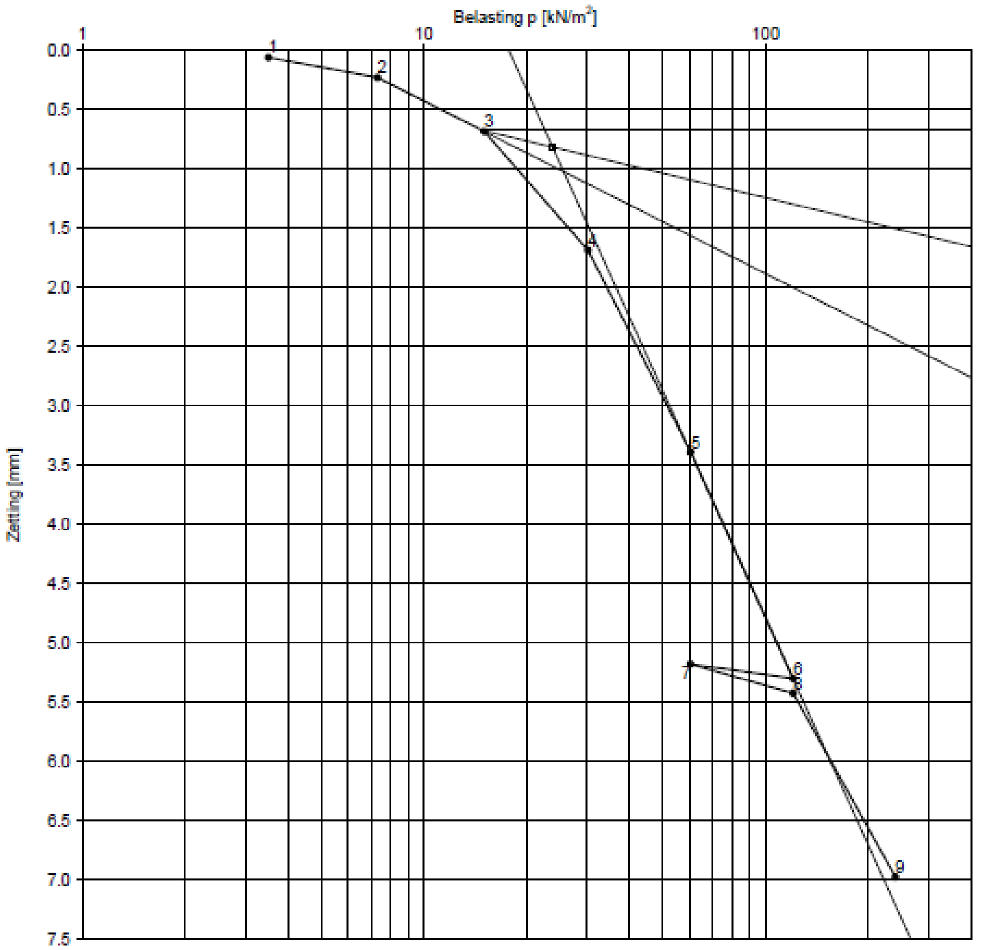
FugroNL Land B.V. Arnhem OED 6.05.0012 /0851:45P KLB M2.OED Opg.:NW dd:30-Jan-2019

Samendrukkingsproef methode KOPPEJAN

Opdr. 1718-0800-000

Afstudeeropdracht: Onderzoek Massastabilisatie 2018-2019

Figure F.19; Compression-stress plot from the results of the oedometer test on undisturbed organic clay sample M2 for the Koppejan method.



Boring	: B2	Grensspanning			
Monster	: Klei M2	σ'_p	=	23	kN/m²
Diepte	: 0.00 m t.o.v. maaiveld	Primaire samendrukkings getal CR			
Grondsoort	: KLEI, sterk siltig, matig humeus, grijs	belasting < σ'_p	=	0.0283	
		belasting > σ'_p	=	0.3405	
		SR Zwelgetal 6-7	=	0.0214	
		RR Herbelastingsgetal 7-8	=	0.0435	
		H_0	=	18.7	mm
γ : 12.9 kN/m³		D	=	50.1	mm
γ_{dr} : 5.7 kN/m³					
w : 127.1 % [m/m]					

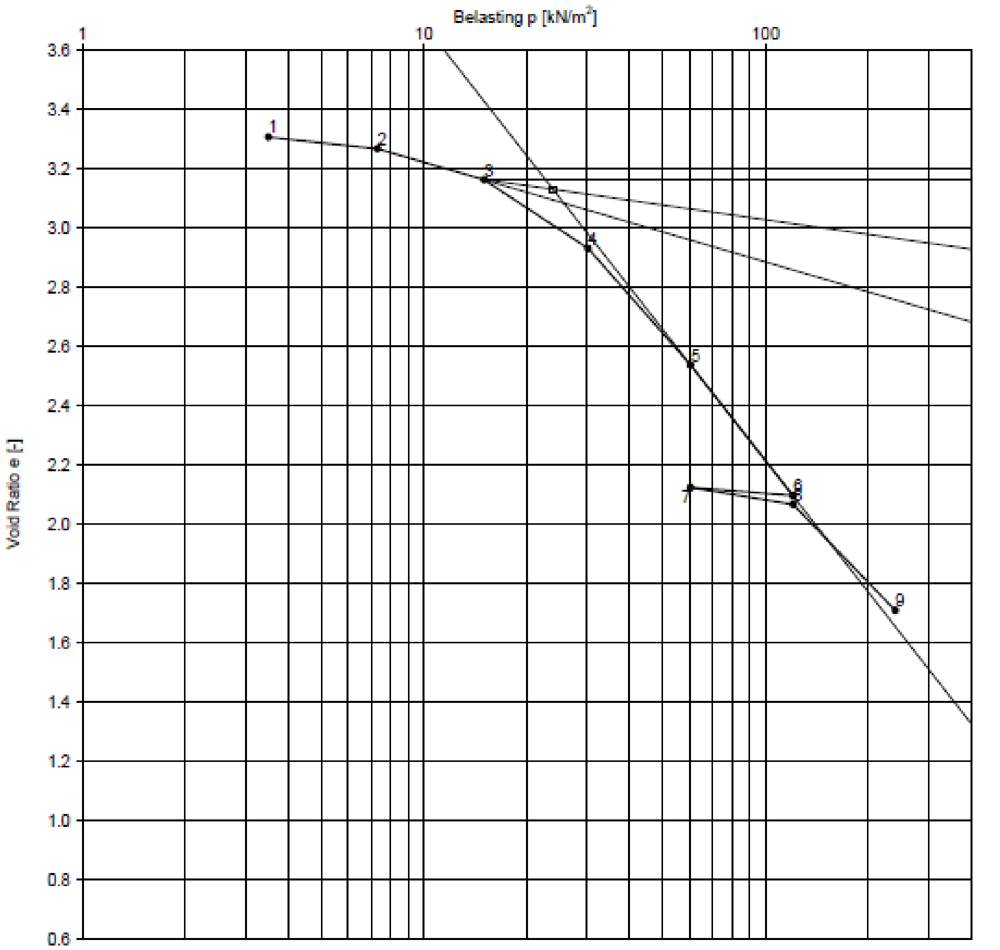
Uitgevoerd conform NEN 5118: 1991 / A1: 1997 nl.

Samendrukkingsproef resultaten z-log p

Opdr. 1718-0800-000

Afstudeeropdracht: Onderzoek Massastabilisatie 2018-2019

Figure F.20; Compression-stress plot from the results of the oedometer test on <u>*undisturbed organic clay sample M2*</u> *for the Anglo-Saxon method (linear strain).*



Boring	:	B2
Monster	:	Klei M2
Diepte	:	0.00 m t.o.v. maaiveld
Grondsoort	:	KLEI, sterk siltig, matig humeus, grijs

γ	:	12.9	kN/m^3
γ_{dr}	:	5.7	kN/m^3
w	:	127.1	% [m/m]
Void ratio e_0	:	3.321	(bij ρ_s = 2.50 t/m^3)

Grensspanning σ'_p	=	23	kN/m^2
Primaire samendrukkings index C_c			
belasting < σ'_p	=	0.1223	
belasting > σ'_p	=	1.4711	
C_{sw} Zwelindex 6-7	=	0.0924	
C_r Herbelastingsindex 7-8	=	0.1880	
H_0	=	18.7	mm
D	=	50.1	mm

Fugro NL Land B.V. Arnhem OED 605.0012 /1425:58P KLB M2.OED Opg.: NW dd:29-Jan-2019

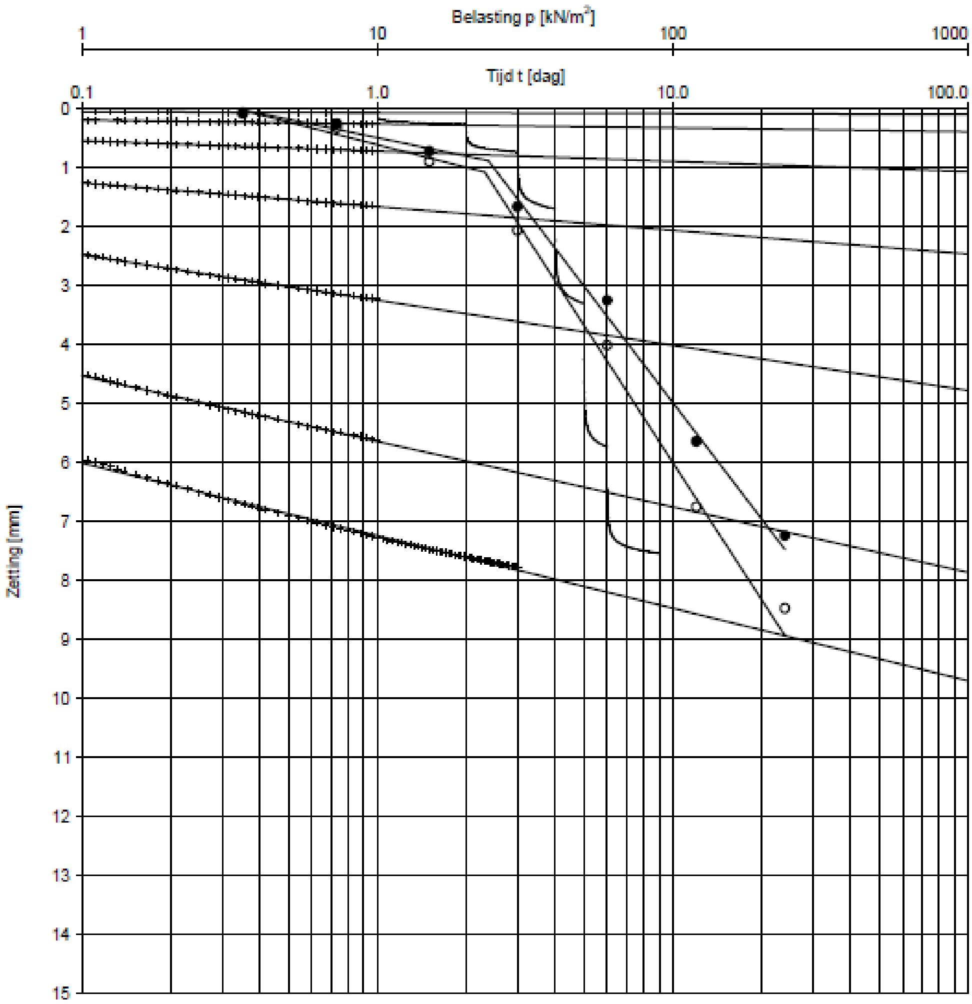
Uitgevoerd conform NEN 5118: 1991 / A1: 1997 nl.

Samendrukkingsproef resultaten e-log p

Opdr. 1718-0800-000

Afstudeeropdracht: Onderzoek Massastabilisatie 2018-2019

Figure F.21; Compression-stress plot from the results of the oedometer test on <u>undisturbed organic clay sample M2</u> for the Anglo-Saxon method (void ratio).



Boring	: B2		C	=	21.7	
Monster	: Klei M3		C'	=	3.9	
Diepte	: 0.00 m t.o.v. maaiveld		σ'_p	=	24	kN/m²
Grondsoort	: KLEI, sterk siltig, matig humeus, grijs					
			C_p	=	42.7	
			C_s	=	176.8	
γ	:	13.2 kN/m³	C'_p	=	6.7	
γ_{dr}	:	5.9 kN/m³	C'_s	=	37.7	
w	:	123.0 % [m/m]				
			H_i	=	19.1	mm
			D	=	50.1	mm

Uitgevoerd conform NEN 5118: 1991 / A1: 1997 nl.

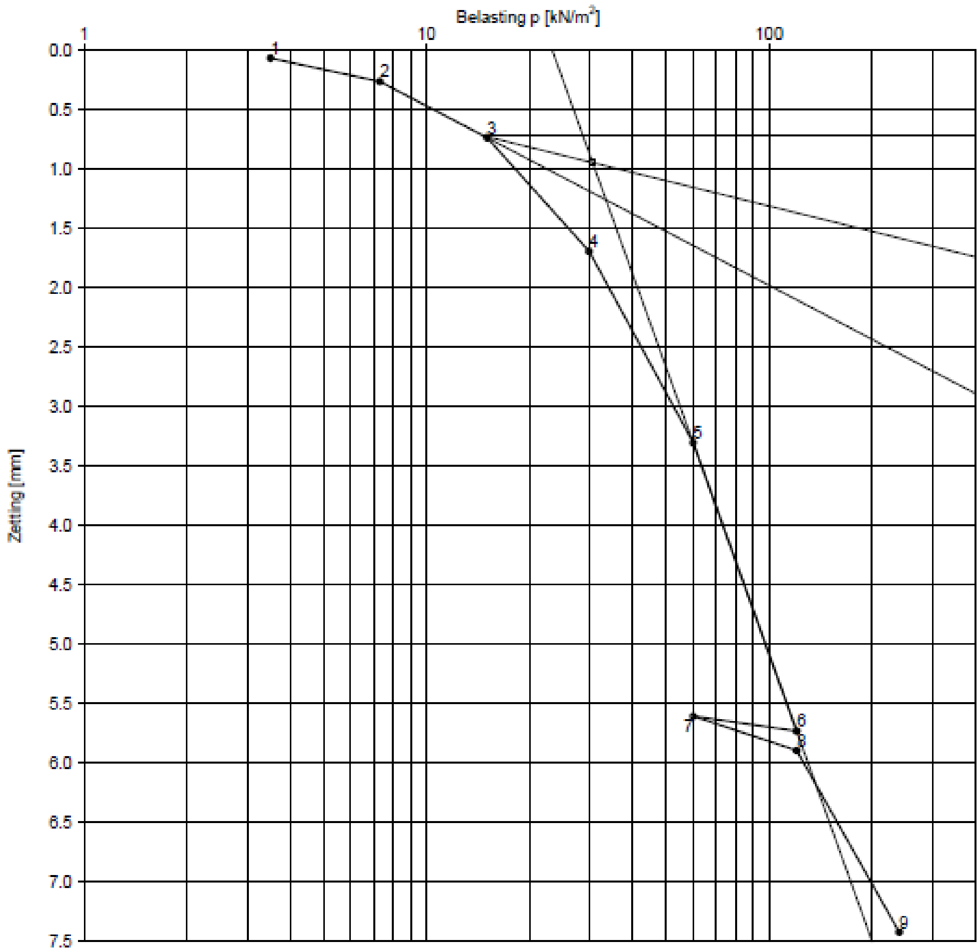
Fugro NL Land B.V. Arnhem OED 6.05.0012 /0952:11P KLB M3.OED Opg.:NW dd:30-Jan-2019

Samendrukkingsproef methode KOPPEJAN

Opdr. 1718-0800-000

Afstudeeropdracht: Onderzoek Massastabilisatie 2018-2019

Figure F.22; Compression-stress plot from the results of the oedometer test on <u>undisturbed organic clay sample M3</u> for the Koppejan method.



Boring	: B2	
Monster	: Klei M3	
Diepte	: 0.00 m t.o.v. maaiveld	
Grondsoort	: KLEI, sterk siltig, matig humeus, grijs	
γ	:	13.2 kN/m³
γ_{dr}	:	5.9 kN/m³
w	:	123.0 % [m/m]

Grensspanning			
σ'_p	=	30	kN/m²
Primaire samendrukkings getal CR			
belasting < σ'_p	=	0.0321	
belasting > σ'_p	=	0.4203	
SR Zwelgetal 6-7	=	0.0215	
RR Herbelastingsgetal 7-8	=	0.0497	
H_0	=	19.1	mm
D	=	50.1	mm

Fugro NL Land B.V. Arnhem OED 6.05.0012 /14:43:52/P KLB M3.OED Opg.:NW dd:29-Jan-2019

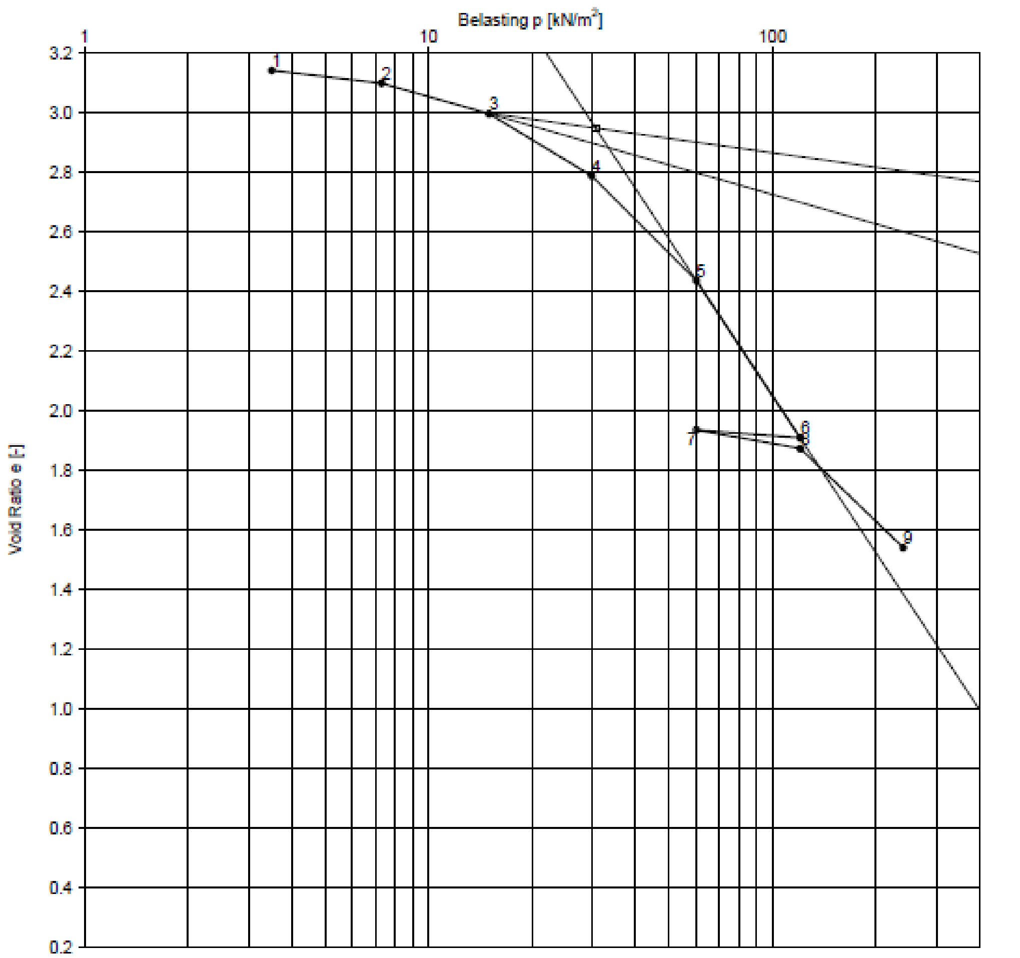
Uitgevoerd conform NEN 5118: 1991 / A1: 1997 nl.

Samendrukkingsproef resultaten z-log p

Opdr. 1718-0800-000

Afstudeeropdracht: Onderzoek Massastabilisatie 2018-2019

Figure F.23; Compression-stress plot from the results of the oedometer test on <u>undisturbed organic clay sample M3</u> for the Anglo-Saxon method (linear strain).



Boring	:	B2
Monster	:	Klei M3
Diepte	:	0.00 m t.o.v. maaiveld
Grondsoort	:	KLEI, sterk siltig, matig humeus, grijs

γ	:	13.2 kN/m³
γ_{dr}	:	5.9 kN/m³
w	:	123.0 % [m/m]
Void ratio e_0	:	3.156 (bij ρ_s = 2.50 t/m³)

Grensspanning			
σ'_p	=	30	kN/m²
Primaire samendrukkings index C_c			
belasting < σ'_p	=	0.1336	
belasting > σ'_p	=	1.7469	
C_{sw} Zwelindex 6-7	=	0.0893	
C_r Herbelastingsindex 7-8	=	0.2067	
H_0	=	19.1	mm
D	=	50.1	mm

Uitgevoerd conform NEN 5118: 1991 / A1: 1997 nl.

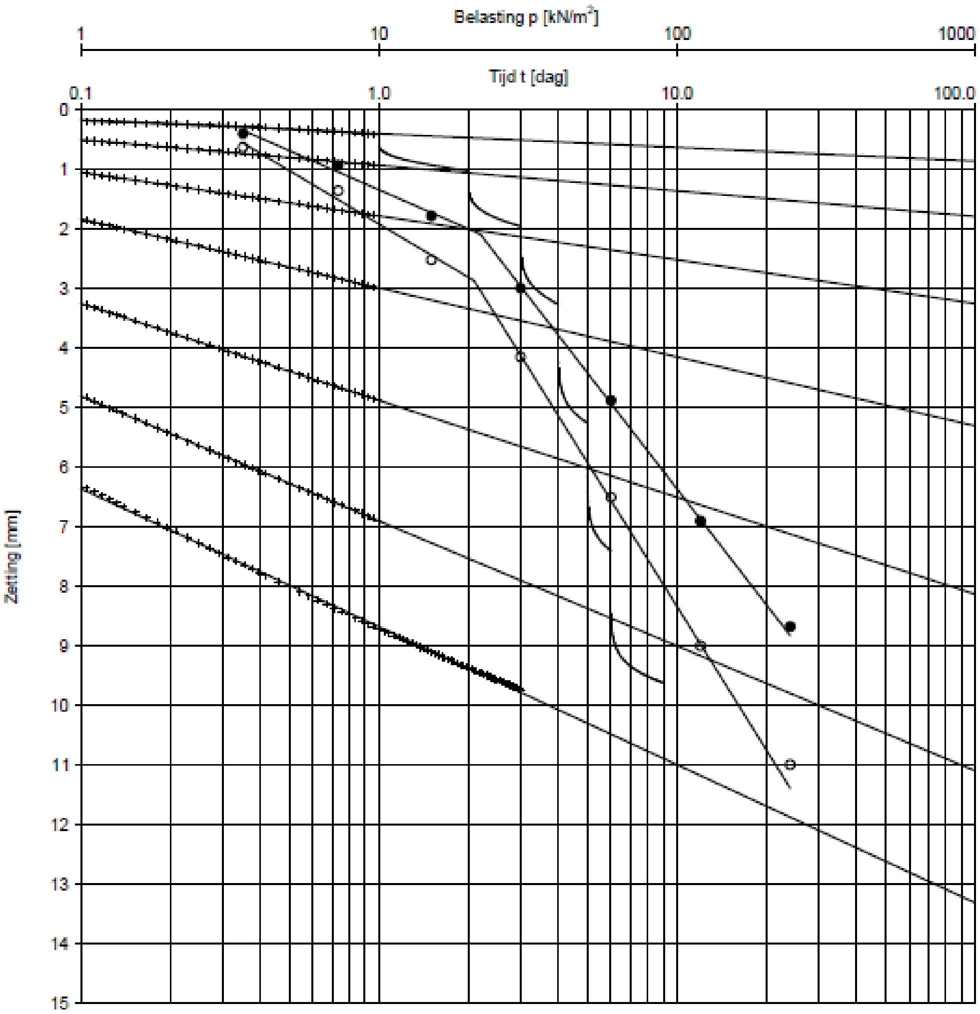
FugroNL Land B.V. Arnhem OED 605.0012 /1443:57/P KLEI M3.OED Opg.:NW dd:29-Jan-2019

Samendrukkingsproef resultaten e-log p

Opdr. 1718-0800-000

Afstudeeropdracht: Onderzoek Massastabilisatie 2018-2019

Figure F.24; Compression-stress plot from the results of the oedometer test on undisturbed organic clay sample M3 for the Anglo-Saxon method (void ratio).



Boring	: B1		C	=	8.0	
Monster	: Veen M1		C'	=	3.4	
Diepte	: 0.00 m t.o.v. maaiveld		σ'_p	=	22	kN/m²
Grondsoort	: VEEN, mineraalarm, bruin					
			C_p	=	19.7	
			C_s	=	54.3	
γ	:	9.0 kN/m³	C'_p	=	6.6	
γ_{dr}	:	2.0 kN/m³	C'_s	=	27.7	
w	:	354.0 % [m/m]				
			H_i	=	18.7	mm
			D	=	50.0	mm

Uitgevoerd conform NEN 5118: 1991 / A1: 1997 nl.

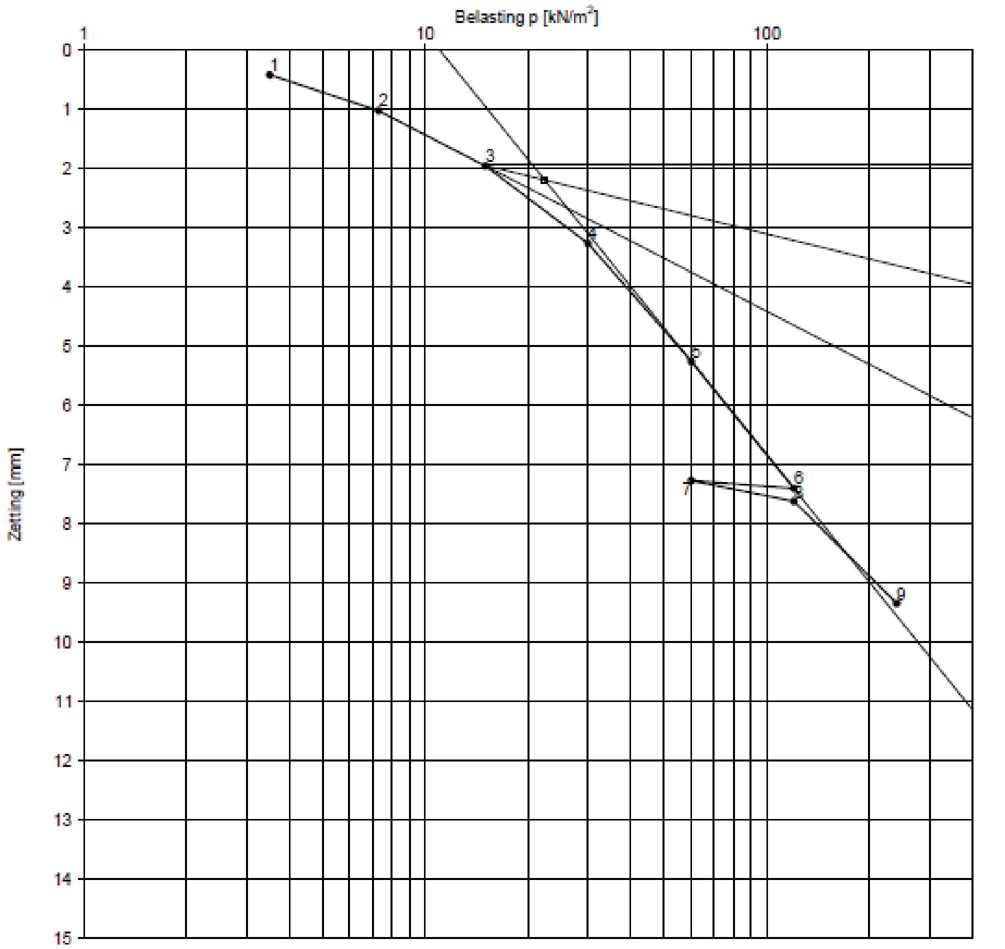
Fugro NL Land B.V. Arnhem OED 6.05.0012 /0849.05P-VEEN M1.OED Opg.:NW dd:30-Jan-2019

Samendrukkingsproef methode KOPPEJAN

Opdr. 1718-0800-000

Afstudeeropdracht: Onderzoek Massastabilisatie 2018-2019

Figure F.25; Compression-stress plot from the results of the oedometer test on undisturbed peat sample M1 for the Koppejan method.



Boring	: B1	
Monster	: Veen M1	
Diepte	: 0.00 m t.o.v. maaiveld	
Grondsoort	: VEEN, mineraalarm, bruin	
γ	:	9.0 kN/m³
γ_{dr}	:	2.0 kN/m³
w	:	354.0 % [m/m]

Grensspanning σ'_p	=	21	kN/m²
Primaire samendrukkings getal CR			
belasting < σ'_p	=	0.1017	
belasting > σ'_p	=	0.3816	
SR Zwelgetal 6-7	=	0.0237	
RR Herbelastingsgetal 7-8	=	0.0625	
H_0	=	18.7	mm
D	=	50.0	mm

Uitgevoerd conform NEN 5118: 1991 / A1: 1997 nl.

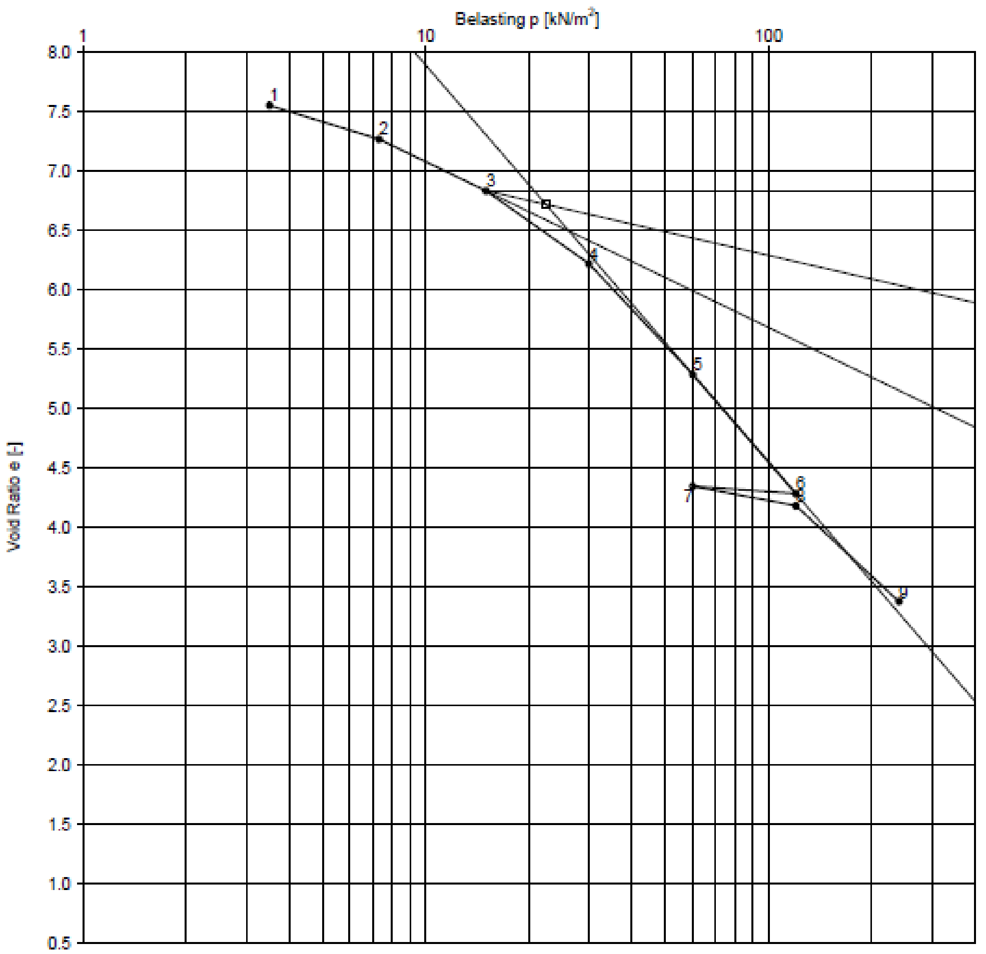
FugroNL Land B.V. Arnhem OED 6.05.0012 /1303-36P VEEN M1.OED Opg.:NW dd:29-Jan-2019

Samendrukkingsproef resultaten z-log p

Opdr. 1718-0800-000

Afstudeeropdracht: Onderzoek Massastabilisatie 2018-2019

Figure F.26; Compression-stress plot from the results of the oedometer test on undisturbed peat sample M1 for the Anglo-Saxon method (linear strain).



Boring	:	B1
Monster	:	Veen M1
Diepte	:	0.00 m t.o.v. maaiveld
Grondsoort	:	VEEN, mineraalarm, bruin

γ	:	9.0 kN/m^3
γ_{dr}	:	2.0 kN/m^3
w	:	354.0 % [m/m]
Void ratio e_0	:	7.747 (bij ρ_s = 1.77 t/m^3)

Grensspanning			
σ'_p	=	21	kN/m^2
Primaire samendrukkings index C_c			
belasting < σ'_p	=	0.8893	
belasting > σ'_p	=	3.3382	
C_{sw} Zwelindex 6-7	=	0.2072	
C_r Herbelastingsindex 7-8	=	0.5467	
H_0	=	18.7	mm
D	=	50.0	mm

Uitgevoerd conform NEN 5118: 1991 / A1: 1997 nl.

FugroNL Land B.V. Arnhem OED 6.05.0012 /1303.41P.VEEN M1.OED Opg.:NW dd:29-Jan-2019

Samendrukkingsproef resultaten e-log p

Opdr. 1718-0800-000

Afstudeeropdracht: Onderzoek Massastabilisatie 2018-2019

Figure F.27; Compression-stress plot from the results of the oedometer test on <u>undisturbed peat sample M1</u> for the Anglo-Saxon method (void ratio).

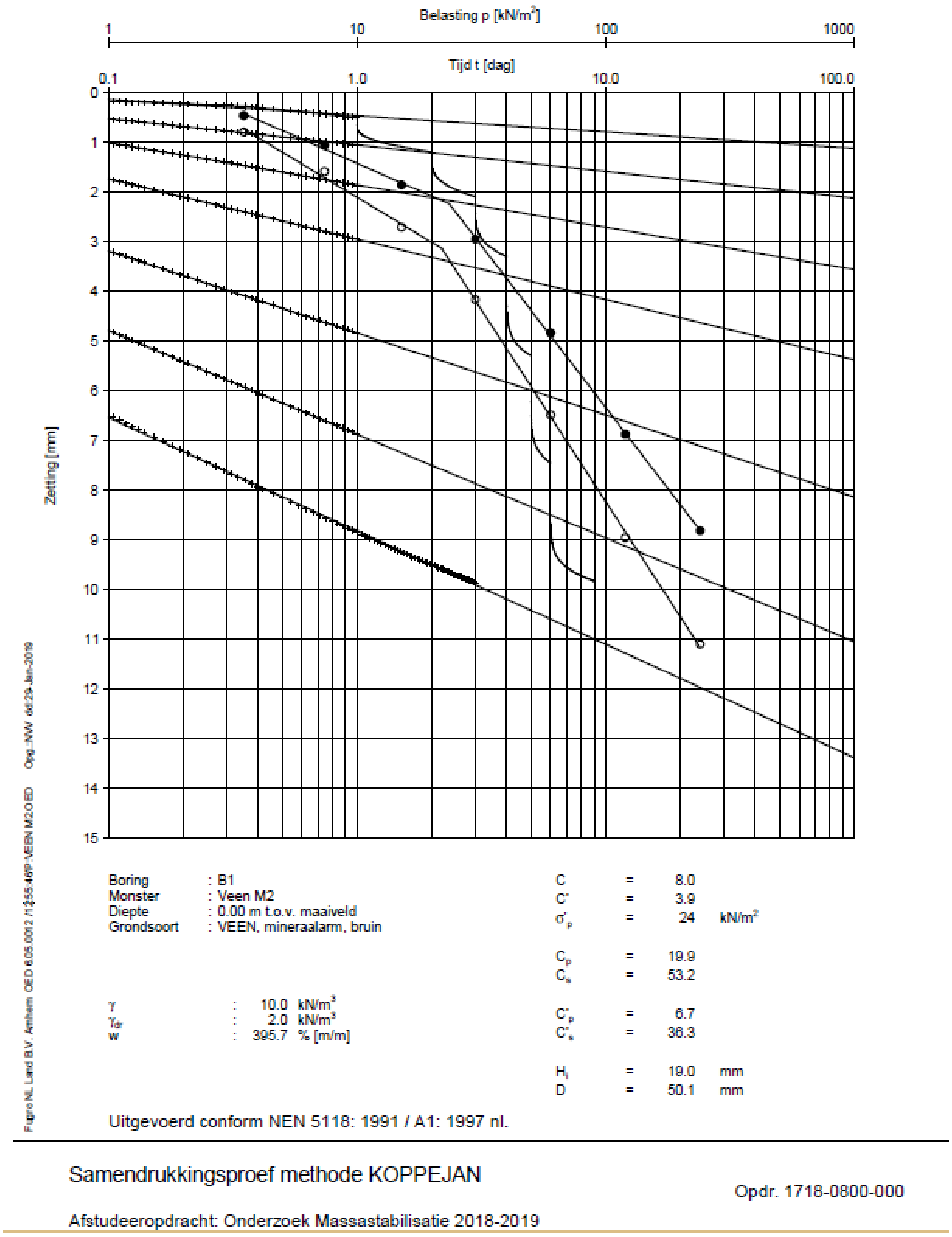
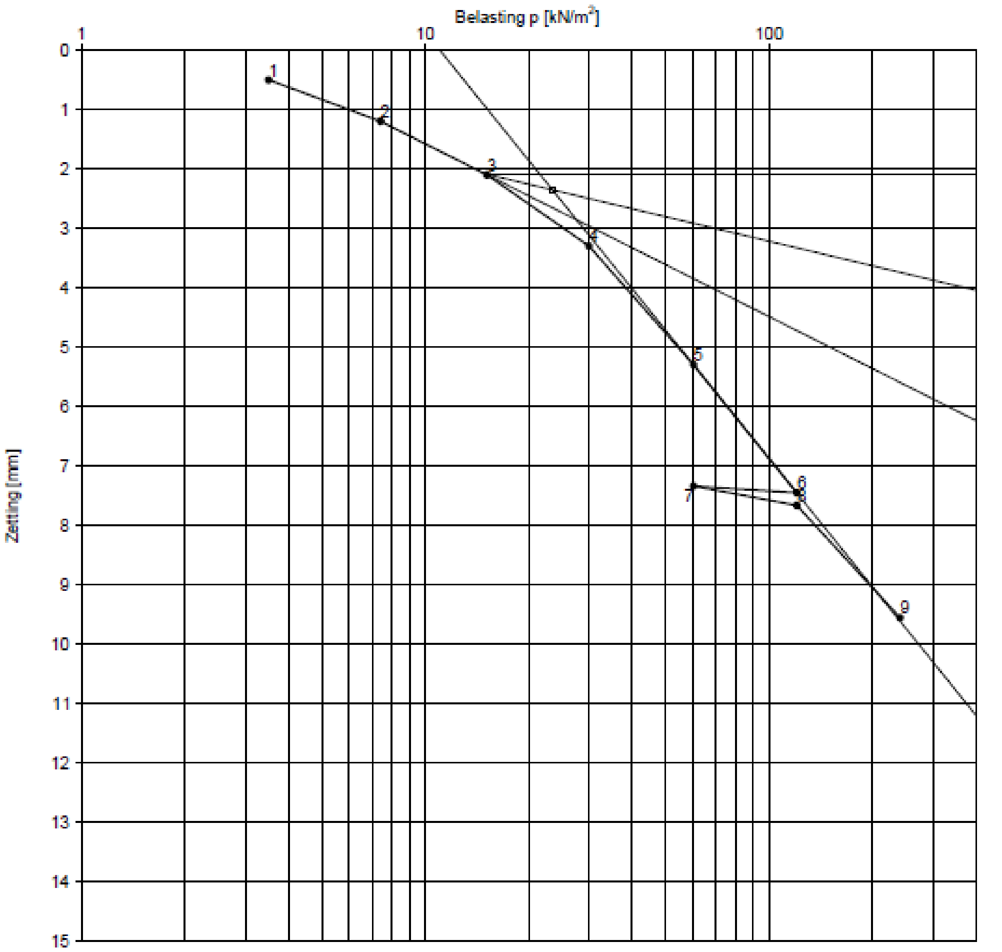


Figure F.28; Compression-stress plot from the results of the oedometer test on <u>undisturbed peat sample M2</u> for the Koppejan method.



Boring	:	B1
Monster	:	Veen M2
Diepte	:	0.00 m t.o.v. maaiveld
Grondsoort	:	VEEN, mineraalarm, bruin

γ	:	10.0	kN/m³
γ_{dr}	:	2.0	kN/m³
w	:	395.7	% [m/m]

Grensspanning σ'_p	=	22	kN/m²
Primaire samendrukkings getal CR			
belasting < σ'_p	=	0.1127	
belasting > σ'_p	=	0.3776	
SR Zwelgetal 6-7	=	0.0193	
RR Herbelastingsgetal 7-8	=	0.0570	
H_0	=	19.0	mm
D	=	50.1	mm

Fugro NL Land B.V. Arnhem OED 605.0012 /1255.03/P.VEEN M2.OED Opg.:NW dd 29-Jan-2019

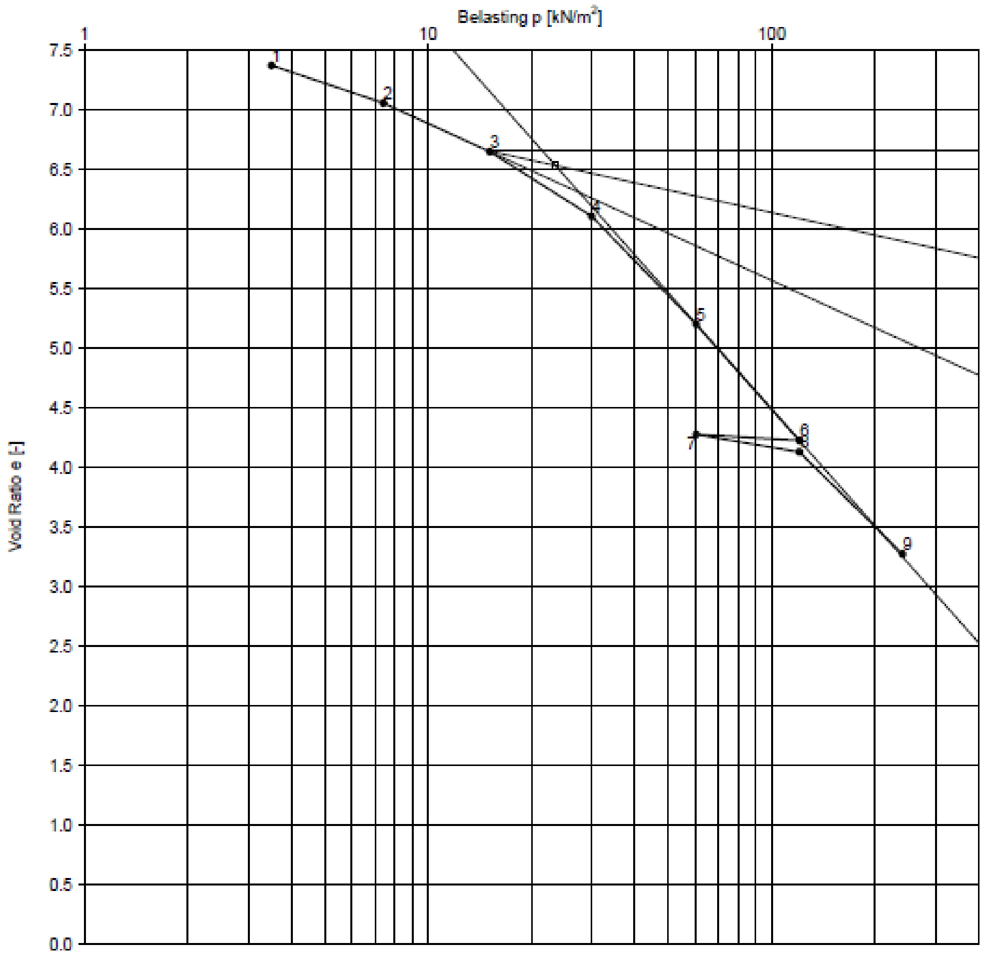
Uitgevoerd conform NEN 5118: 1991 / A1: 1997 nl.

Samendrukkingsproef resultaten z-log p

Opdr. 1718-0800-000

Afstudeeropdracht: Onderzoek Massastabilisatie 2018-2019

Figure F.29; Compression-stress plot from the results of the oedometer test on <u>undisturbed peat sample M2</u> for the Anglo-Saxon method (linear strain).



Boring : B1
Monster : Veen M2
Diepte : 0.00 m t.o.v. maaiveld
Grondsoort : VEEN, mineraalarm, bruin

γ : 10.0 kN/m^3
γ_{dr} : 2.0 kN/m^3
w : 395.7 % [m/m]
Void ratio e_0 : 7.599 (bij ρ_s = 1.77 t/m^3)

Grensspanning			
σ'_p	=	22	kN/m^2
Primaire samendrukkings index C_c			
belasting < σ'_p	=	0.9687	
belasting > σ'_p	=	3.2472	
C_{sw} Zwelindex 6-7	=	0.1658	
C_r Herbelastingsindex 7-8	=	0.4898	
H_0	=	19.0	mm
D	=	50.1	mm

Uitgevoerd conform NEN 5118: 1991 / A1: 1997 nl.

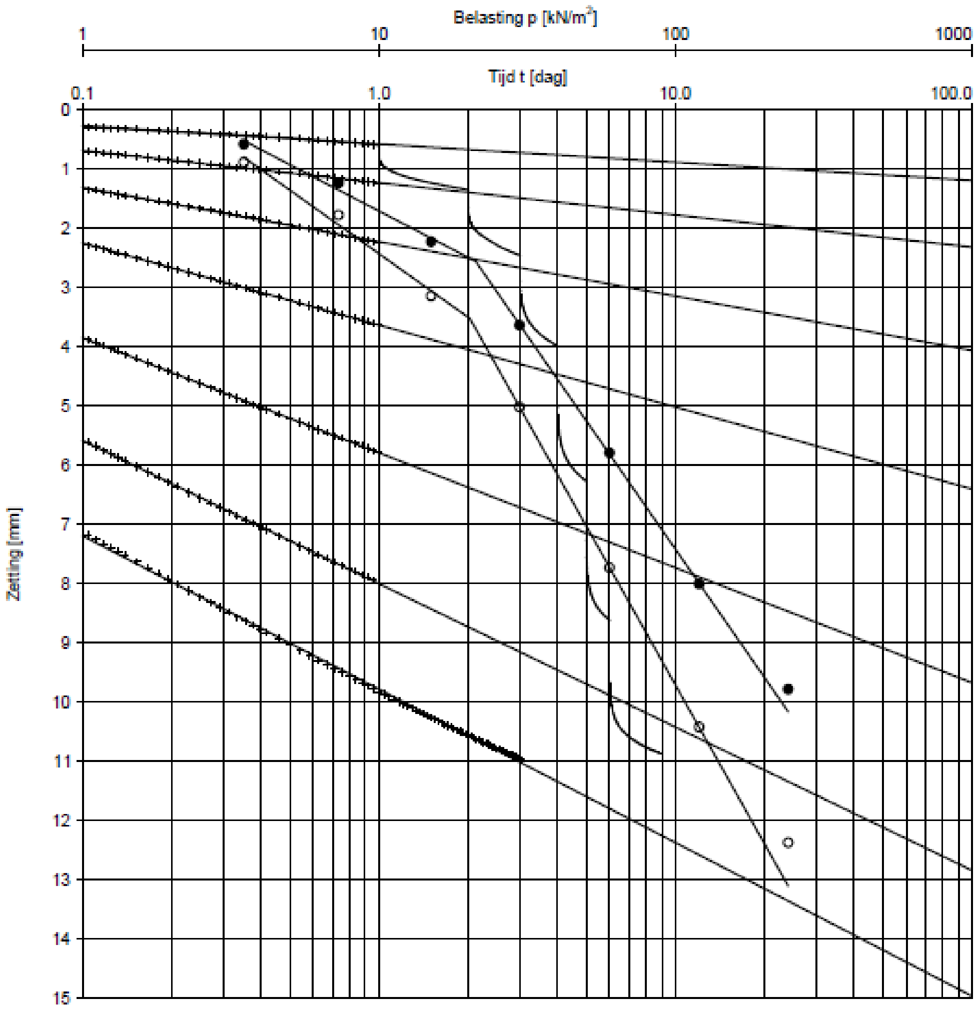
FugroNL Land B.V. Arnhem OED 605.0012 /1255.08/P:VEEN M2.OED Opg.:NW dd 29-Jan-2019

Samendrukkingsproef resultaten e-log p

Opdr. 1718-0800-000

Afstudeeropdracht: Onderzoek Massastabilisatie 2018-2019

Figure F.30; Compression-stress plot from the results of the oedometer test on <u>undisturbed peat sample M2</u> for the Anglo-Saxon method (void ratio).



Boring	: B1	C	=	6.6	
Monster	: Veen M3	C'	=	3.0	
Diepte	: 0.00 m t.o.v. maaiveld	σ'_p	=	21	kN/m²
Grondsoort	: VEEN, mineraalarm, bruin				
		C_p	=	16.4	
		C_s	=	44.2	
γ	: 9.2 kN/m³	C'_p	=	5.9	
γ_{dr}	: 1.7 kN/m³	C'_s	=	25.0	
w	: 433.6 % [m/m]				
		H_i	=	18.5	mm
		D	=	50.1	mm

Uitgevoerd conform NEN 5118: 1991 / A1: 1997 nl.

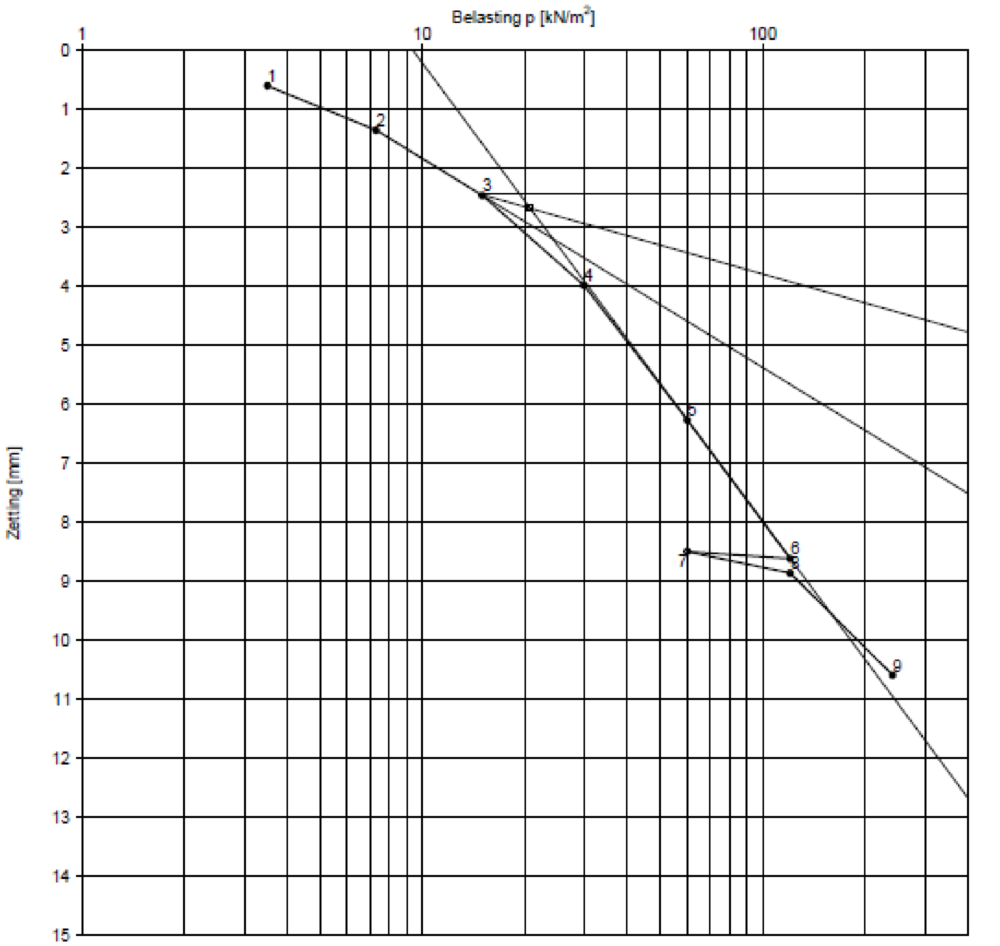
Fugro NL Land B.V. Arnhem OED 605.0012 /0950-40P-VEEN M3.OED Opg.:NW dd:30-Jan-2019

Samendrukkingsproef methode KOPPEJAN

Opdr. 1718-0800-000

Afstudeeropdracht: Onderzoek Massastabilisatie 2018-2019

Figure F.31; Compression-stress plot from the results of the oedometer test on undisturbed peat sample M3 for the Koppejan method.



Boring	: B1	Grensspanning		
Monster	: Veen M3	σ'_p	= 20	kN/m^2
Diepte	: 0.00 m t.o.v. maaiveld	Primaire samendrukkings getal CR		
Grondsoort	: VEEN, mineraalarm, bruin	belasting < σ'_p	= 0.1273	
		belasting > σ'_p	= 0.4191	
		SR Zwelgetal 6-7	= 0.0218	
		RR Herbelastingsgetal 7-8	= 0.0651	
		H_0	= 18.5	mm
γ	: 9.2 kN/m^3	D	= 50.1	mm
γ_{dr}	: 1.7 kN/m^3			
w	: 433.6 % [m/m]			

FugroNL Land B.V. Arnhem OED 6.05.0012 /1151:35P:VEEN M3.OED Opg.:NW dd:29-Jan-2019

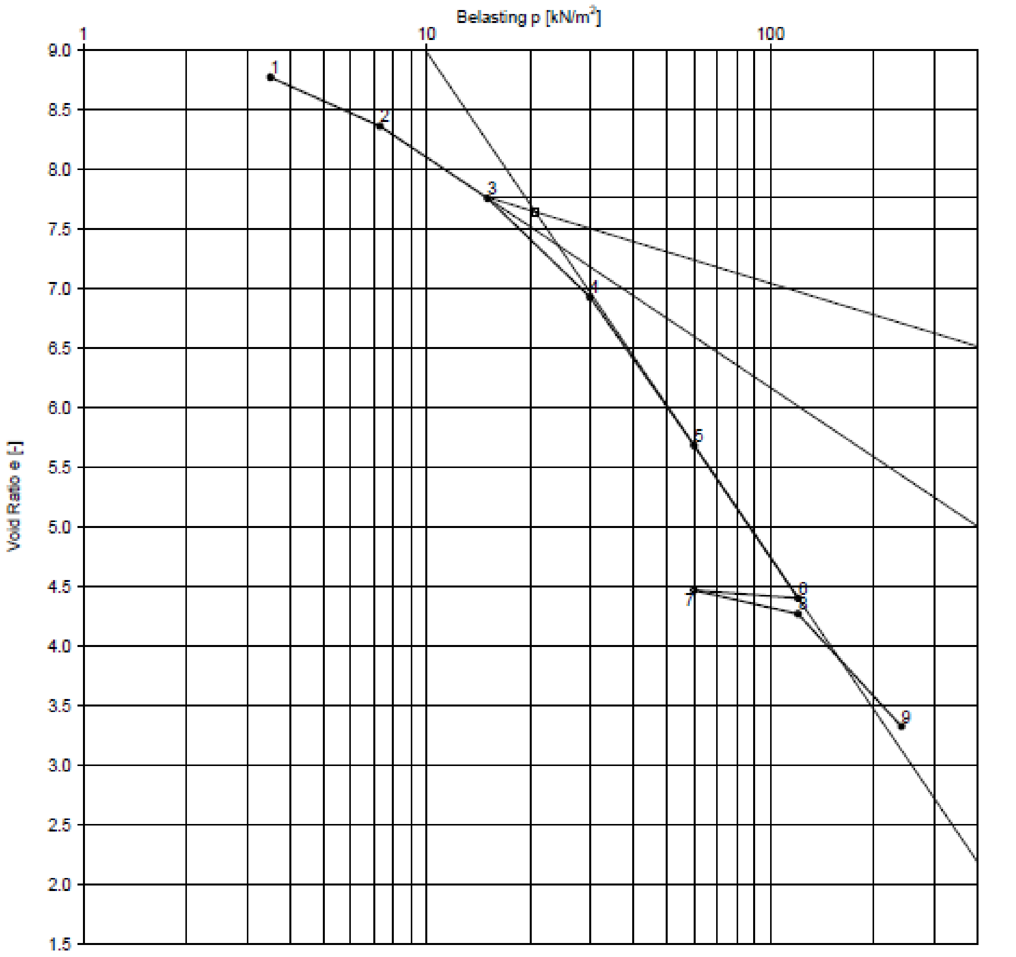
Uitgevoerd conform NEN 5118: 1991 / A1: 1997 nl.

Samendrukkingsproef resultaten z-log p

Opdr. 1718-0800-000

Afstudeeropdracht: Onderzoek Massastabilisatie 2018-2019

Figure F.32; Compression-stress plot from the results of the oedometer test on <u>undisturbed peat sample M3</u> for the Anglo-Saxon method (linear strain).



Boring	: B1	Grensspanning			
Monster	: Veen M3	σ'_p	=	20	kN/m²
Diepte	: 0.00 m t.o.v. maaiveld	Primaire samendrukkings index C_c			
Grondsoort	: VEEN, mineraalarm, bruin	belasting < σ'_p	=	1.2859	
		belasting > σ'_p	=	4.2342	
		C_{sw} Zwelindex 6-7	=	0.2199	
		C_r Herbelastingsindex 7-8	=	0.6579	
		H_0	=	18.5	mm
γ	: 9.2 kN/m³	D	=	50.1	mm
γ_{dr}	: 1.7 kN/m³				
w	: 433.6 % [m/m]				
Void ratio e_0	: 9.103 (bij ρ_s = 1.77 t/m³)				

Uitgevoerd conform NEN 5118: 1991 / A1: 1997 nl.

FugroNL Land B.V. Arnhem OED 605.0012 /1151:41P:VEEN M3.OED Opg.:NW dd:29-Jan-2019

Samendrukkingsproef resultaten e-log p

Opdr. 1718-0800-000

Afstudeeropdracht: Onderzoek Massastabilisatie 2018-2019

Figure F.33; Compression-stress plot from the results of the oedometer test on <u>undisturbed peat sample M3</u> for the Anglo-Saxon method (void ratio).

F.2 Phase 2 – Binder selection

F.2.1 Settlement curves (laboratory stabilisation procedure)

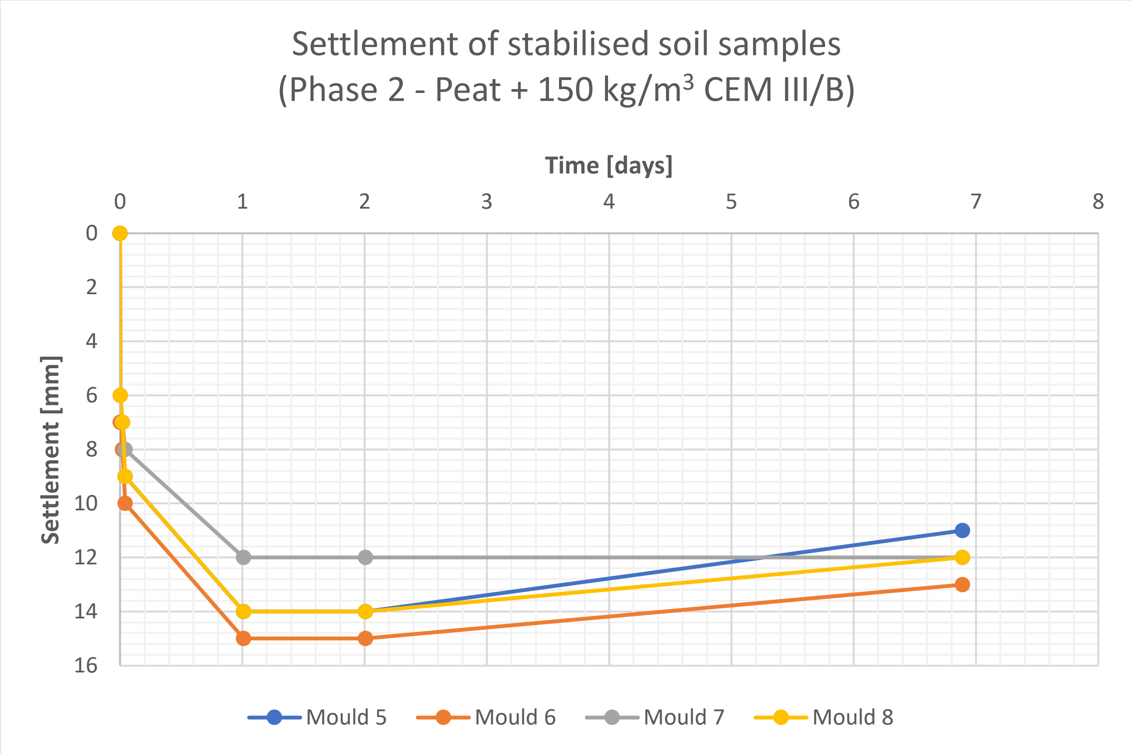


Figure F.34; The measured settlement of the peat samples stabilised with 150 kg/m³ of blast-furnace slag cement (CEM III/B) in phase 2 of the laboratory research.

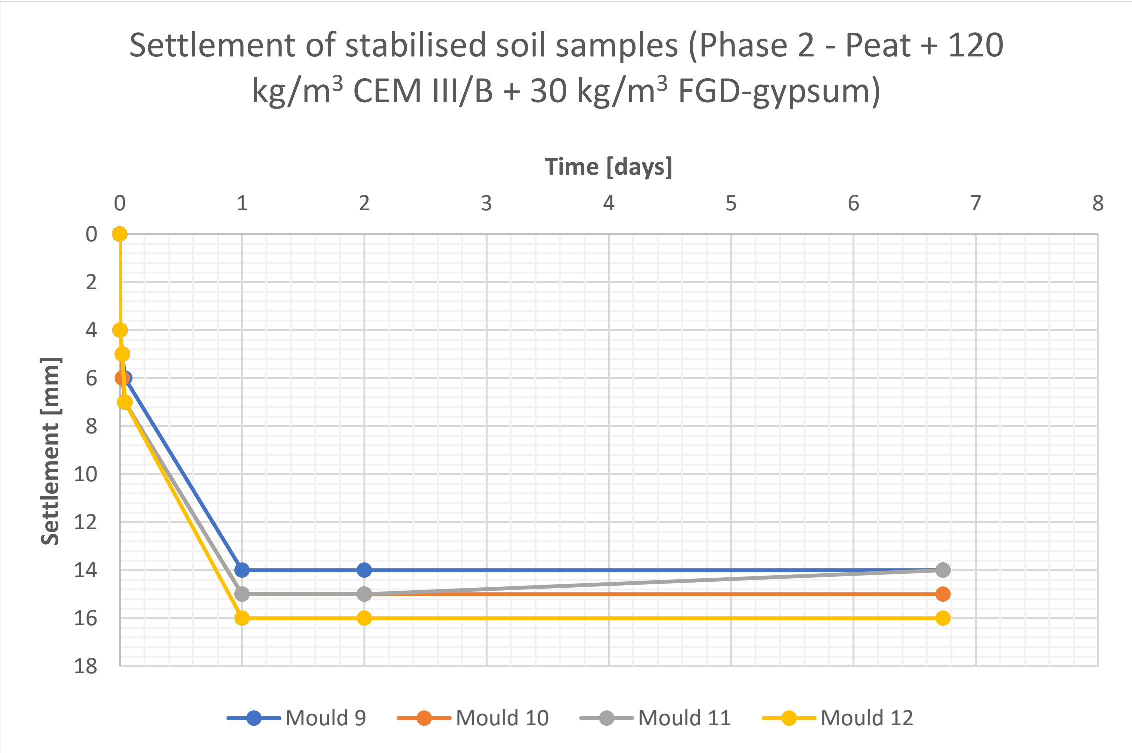


Figure F.35; The measured settlement of the peat samples stabilised with 120 kg/m³ of blast-furnace slag cement (CEM III/B) and 30 kg/m³ of FGD-gypsum in phase 2 of the laboratory research.

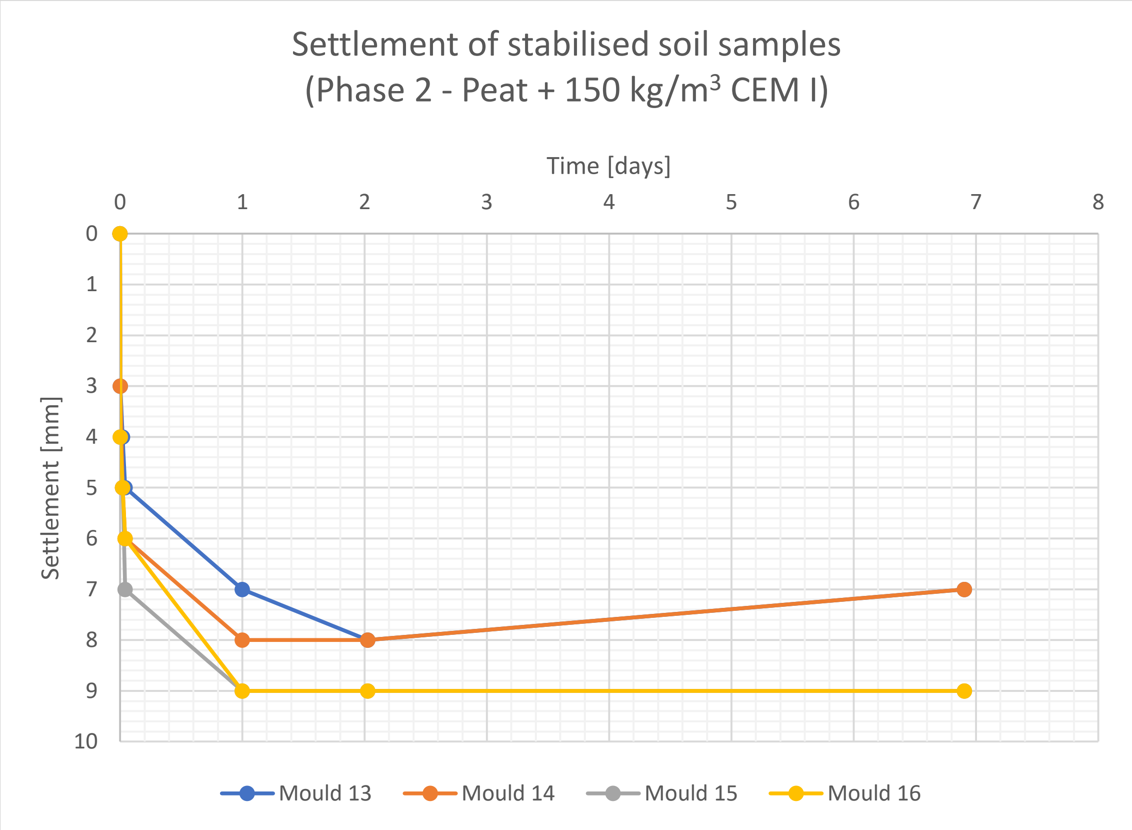


Figure F.36; The measured settlement of the peat samples stabilised with 150 kg/m³ of Portland cement (CEM I) in phase 2 of the laboratory research.

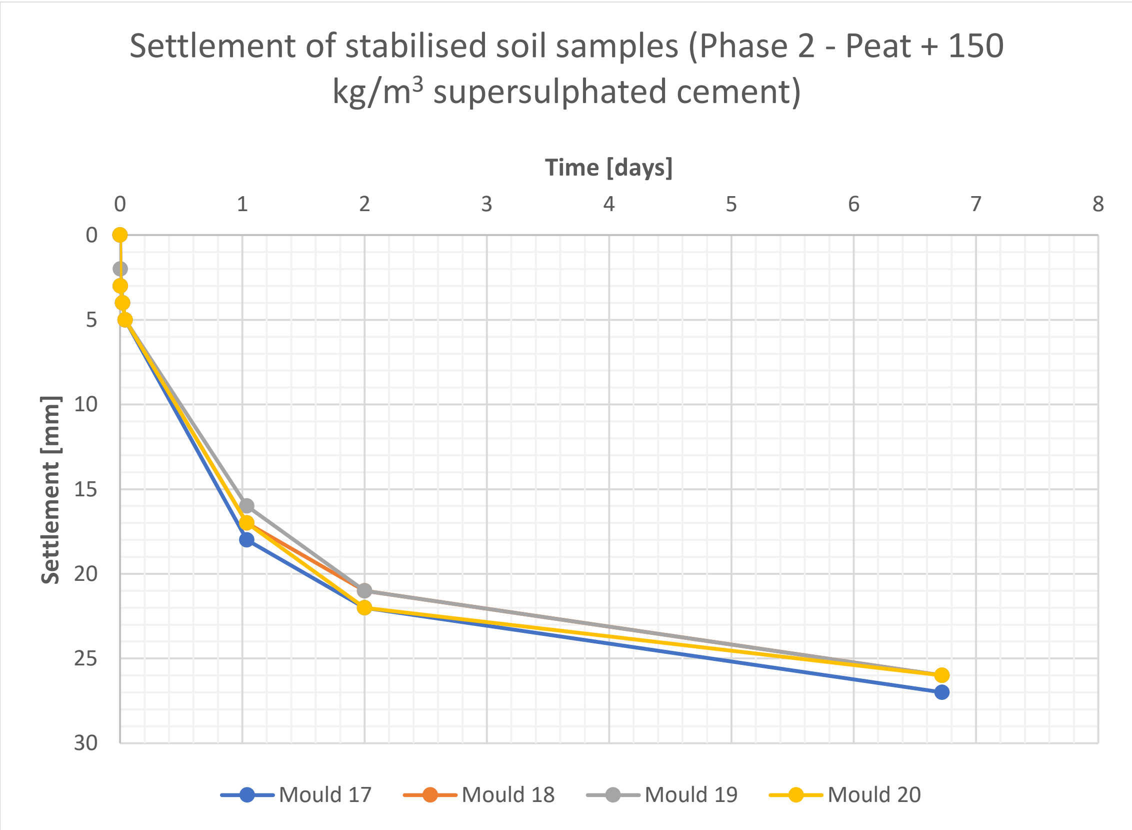


Figure F.37; The measured settlement of the peat samples stabilised with 150 kg/m³ of supersulphated cement in phase 2 of the laboratory research. The supersulphated cement consisted of 127,5 kg/m³ of ground-granulated blast-furnace slag (GGBS), 15 kg/m³ of FGD-gypsum and 7,5 kg/m³ of Portland cement (CEM I).

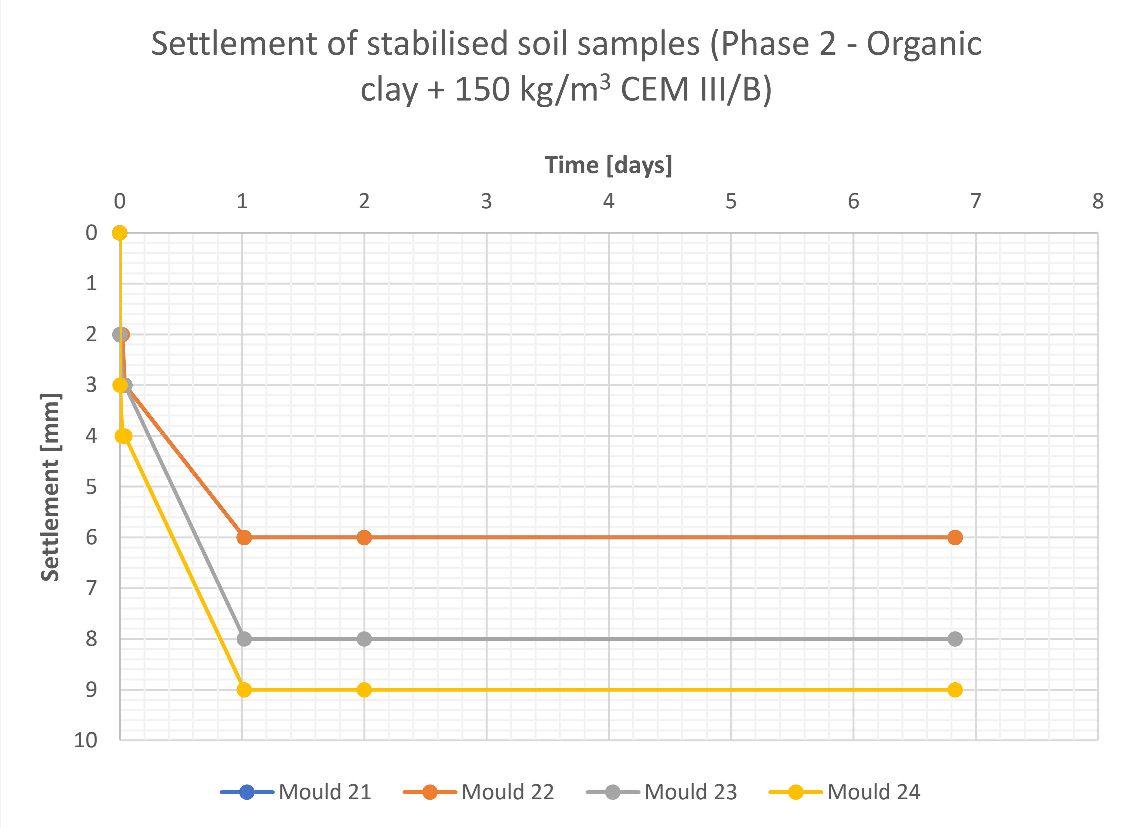


Figure F.38; The measured settlement of the organic clay samples stabilised with 150 kg/m³ of blast-furnace slag cement (CEM III/B) in phase 2 of the laboratory research. It should be noted that the curve for mould 21 is almost entirely identical to the curve for mould 22.

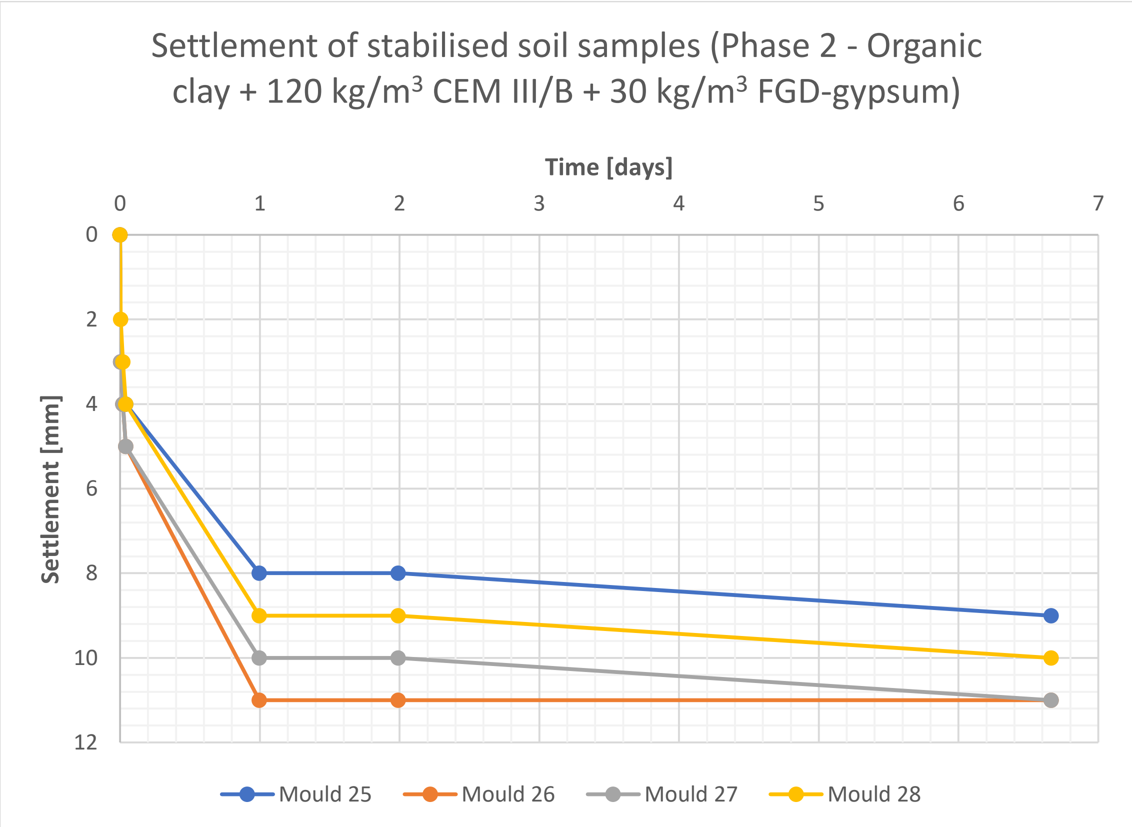


Figure F.39; The measured settlement of the organic clay samples stabilised with 120 kg/m³ of blast-furnace slag cement (CEM III/B) and 30 kg/m³ of FGD-gypsum in phase 2 of the laboratory research.

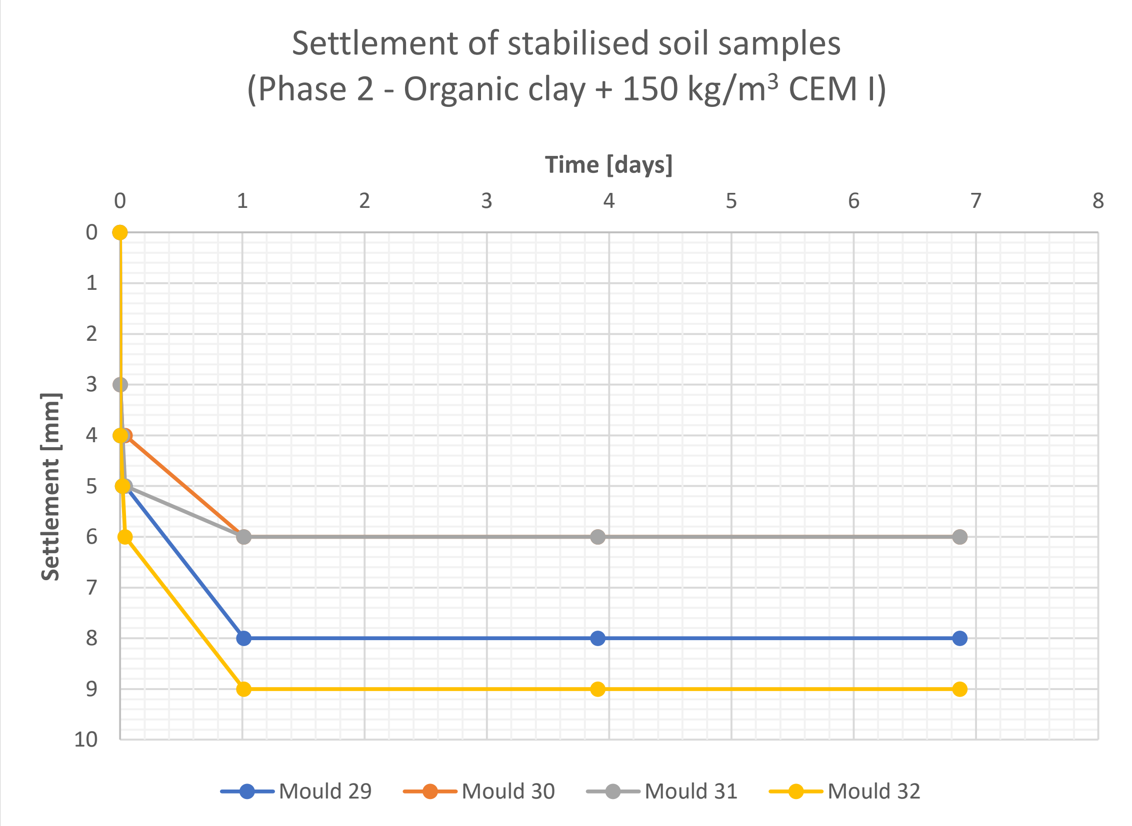


Figure F.40; The measured settlement of the organic clay samples stabilised with 150 kg/m³ of Portland cement (CEM I) in phase 2 of the laboratory research.

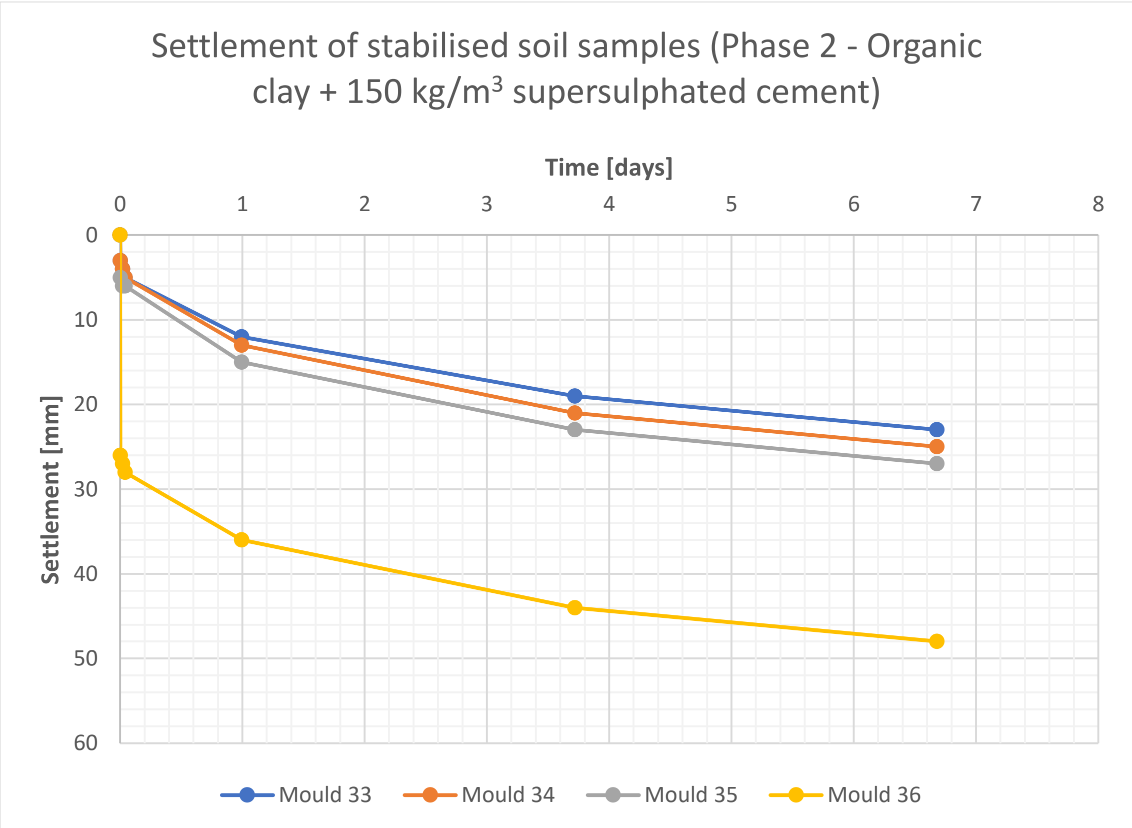


Figure F.41; The measured settlement of the organic clay samples stabilised with 150 kg/m³ of supersulphated cement in phase 2 of the laboratory research. The supersulphated cement consisted of 127,5 kg/m³ of ground-granulated blast-furnace slag (GGBS), 15 kg/m³ of FGD-gypsum and 7,5 kg/m³ of Portland cement (CEM I).

F.2.2 Unit weight of stabilised soil samples (laboratory stabilisation procedure)

Table F.30; The measured unit weights of the different mixtures during phase 2 of the laboratory research. The (?) indicate that the measured unit weights were deemed unreliable.

Mixture	Mould number	Bulk unit weight directly after filling the mould [kN/m³]	Bulk unit weight after 7 days of loading [kN/m³]	Bulk unit weight of extruded sample [kN/m³]
PEAT + 150 kg/m³ CEM III/B	5	11,9	12,1	11,9
	6	11,7	12,0	11,8
	7	11,6	11,9	11,7
	8	11,7	12,0	11,8
PEAT + 120 kg/m³ CEM III/B + 30 kg/m³ FGD-gypsum	9	11,7	12,0	11,8
	10	11,7	12,0	11,8
	11	11,6	11,9	11,8
	12	11,6	11,9	11,8
PEAT + 150 kg/m³ CEM I	13	11,9	12,1	12,0
	14	11,8	12,0	11,9
	15	12,0	12,2	12,1
	16	11,8	12,0	12,0
PEAT + 150 kg/m³ supersulphated cement	17	11,8	12,1	12,7 (?)
	18	11,8	12,1	12,6 (?)
	19	11,8	12,1	12,7 (?)
	20	11,8	12,1	12,7 (?)
ORGANIC CLAY + 150 kg/m³ CEM III/B	21	13,8	14,1	13,9
	22	14,0	14,2	14,0
	23	13,9	14,2	14,1
	24	13,9	14,3	14,1
ORGANIC CLAY + 120 kg/m³ CEM III/B + 30 kg/m³ FGD-gypsum	25	13,7	14,0	13,9
	26	13,6	14,0	13,9
	27	13,7	14,0	13,9
	28	13,8	14,1	13,9
ORGANIC CLAY + 150 kg/m³ CEM I	29	13,6	13,9	13,8
	30	13,8	14,0	13,9
	31	13,7	14,0	13,9
	32	13,7	14,2	14,1
ORGANIC CLAY + 150 kg/m³ supersulphated cement	33	14,0	14,5	14,4
	34	13,9	14,5	14,2
	35	13,9	14,5	14,3
	36	13,0 (?)	14,7 (?)	14,5 (?)

F.2.3 Unconfined compression tests

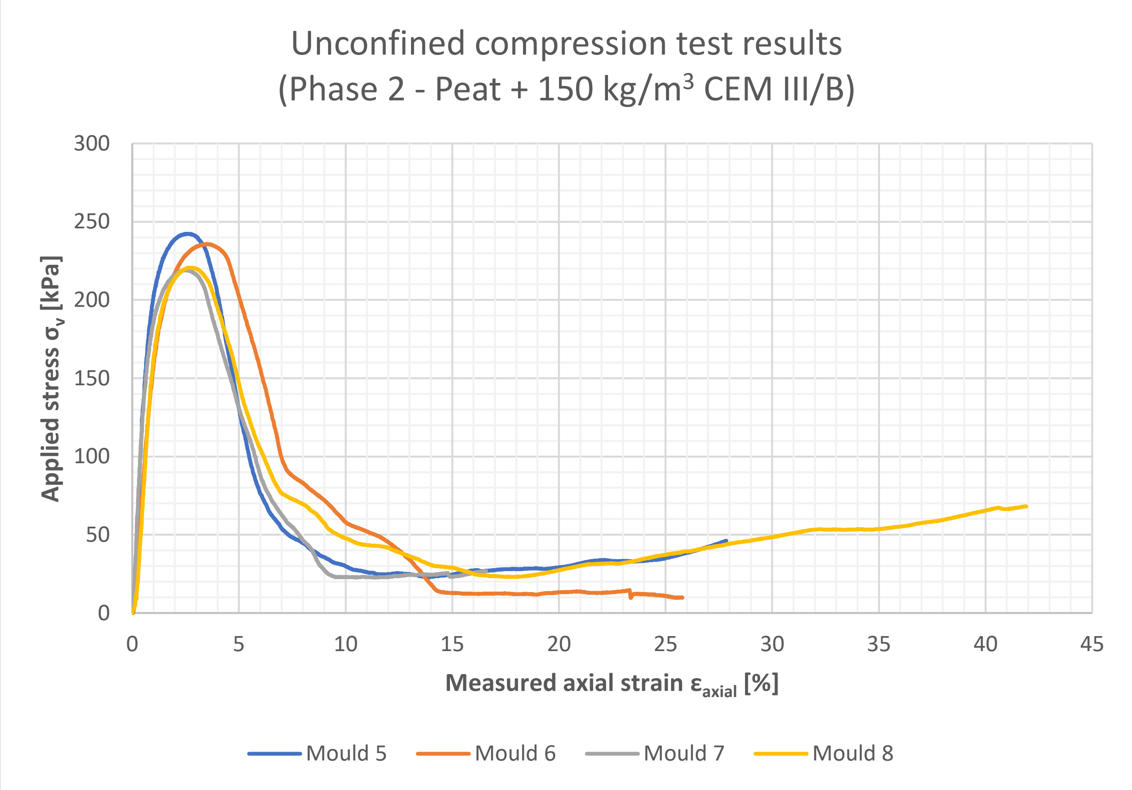


Figure F.42; The stress-strain diagrams obtained from the unconfined compression tests for the peat samples stabilised with 150 kg/m³ of blast-furnace slag cement (CEM III/B) in phase 2 of the laboratory research.

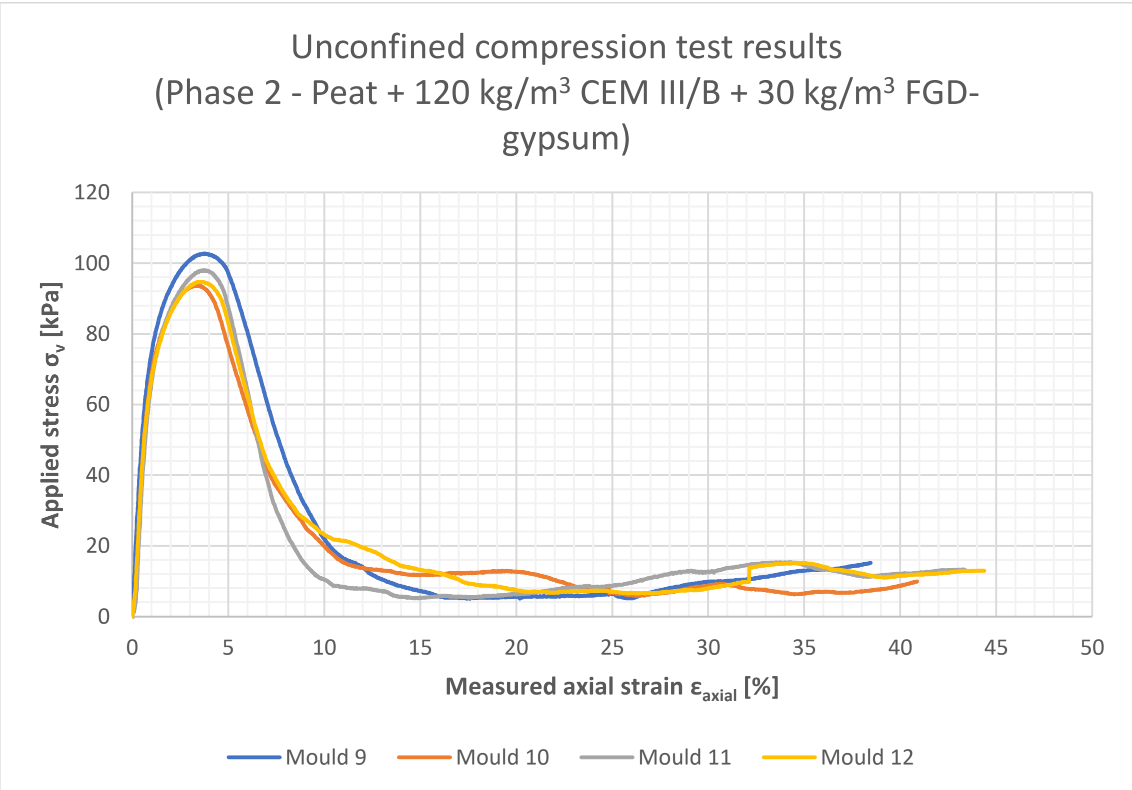


Figure F.43; The stress-strain diagrams obtained from the unconfined compression tests for the peat samples stabilised with 120 kg/m³ of blast-furnace slag cement (CEM III/B) and 30 kg/m³ of FGD-gypsum in phase 2 of the laboratory research.

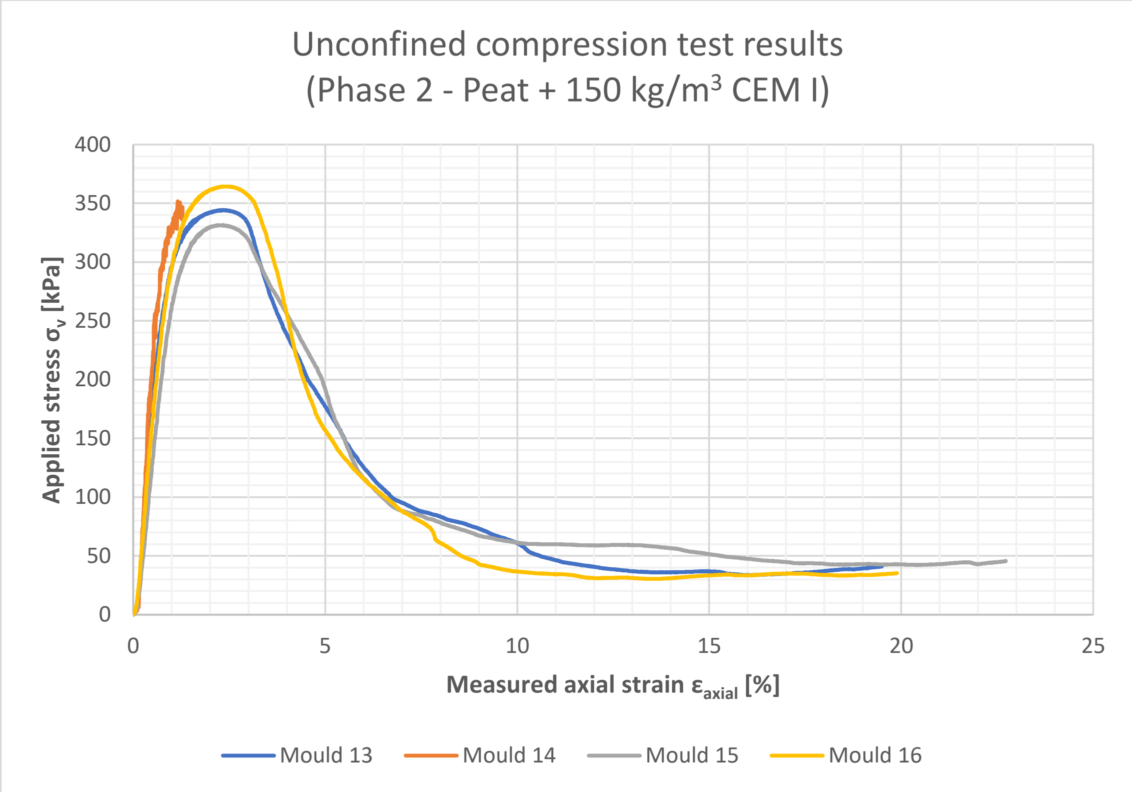


Figure F.44; The stress-strain diagrams obtained from the unconfined compression tests for the peat samples stabilised with 150 kg/m³ of Portland cement (CEM I) in phase 2 of the laboratory research.

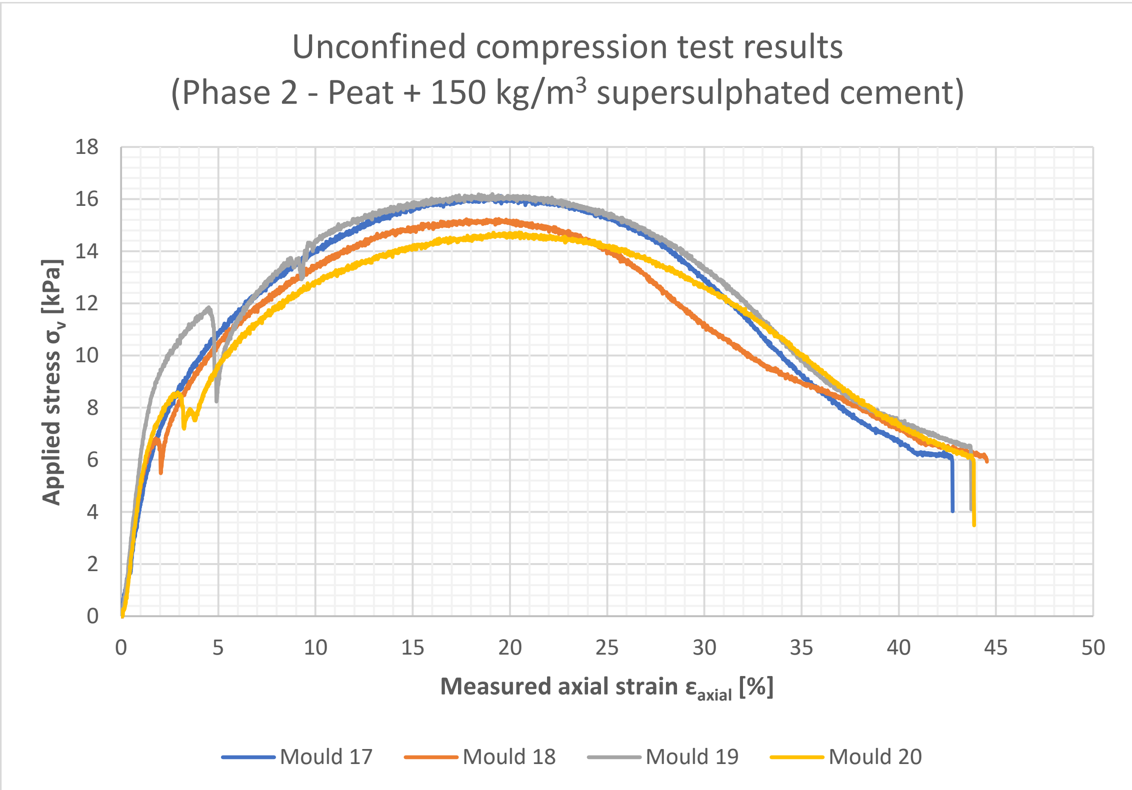


Figure F.45; The stress-strain diagrams obtained from the unconfined compression tests for the peat samples stabilised with 150 kg/m³ of supersulphated cement in phase 2 of the laboratory research. The supersulphated cement consisted of 127,5 kg/m³ of ground-granulated blast-furnace slag (GGBS), 15 kg/m³ of FGD-gypsum and 7,5 kg/m³ of Portland cement (CEM I).

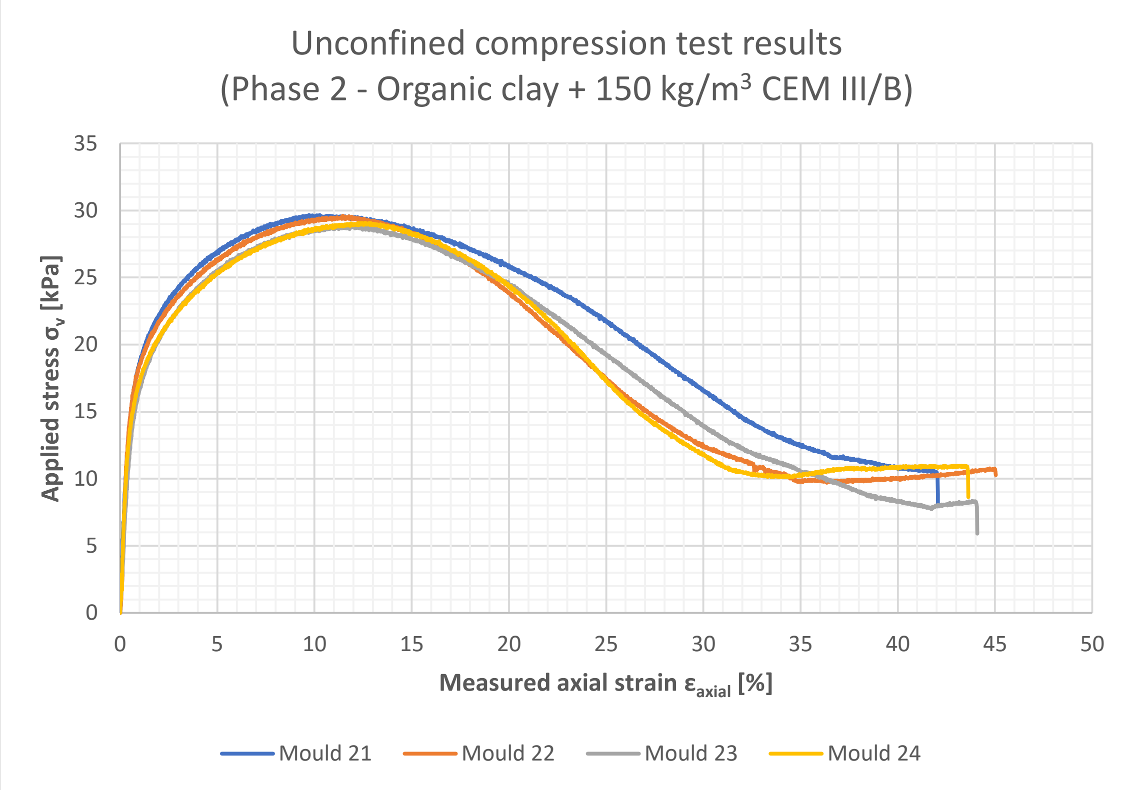


Figure F.46; The stress-strain diagrams obtained from the unconfined compression tests for the organic clay samples stabilised with 150 kg/m³ of blast-furnace slag cement (CEM III/B) in phase 2 of the laboratory research.

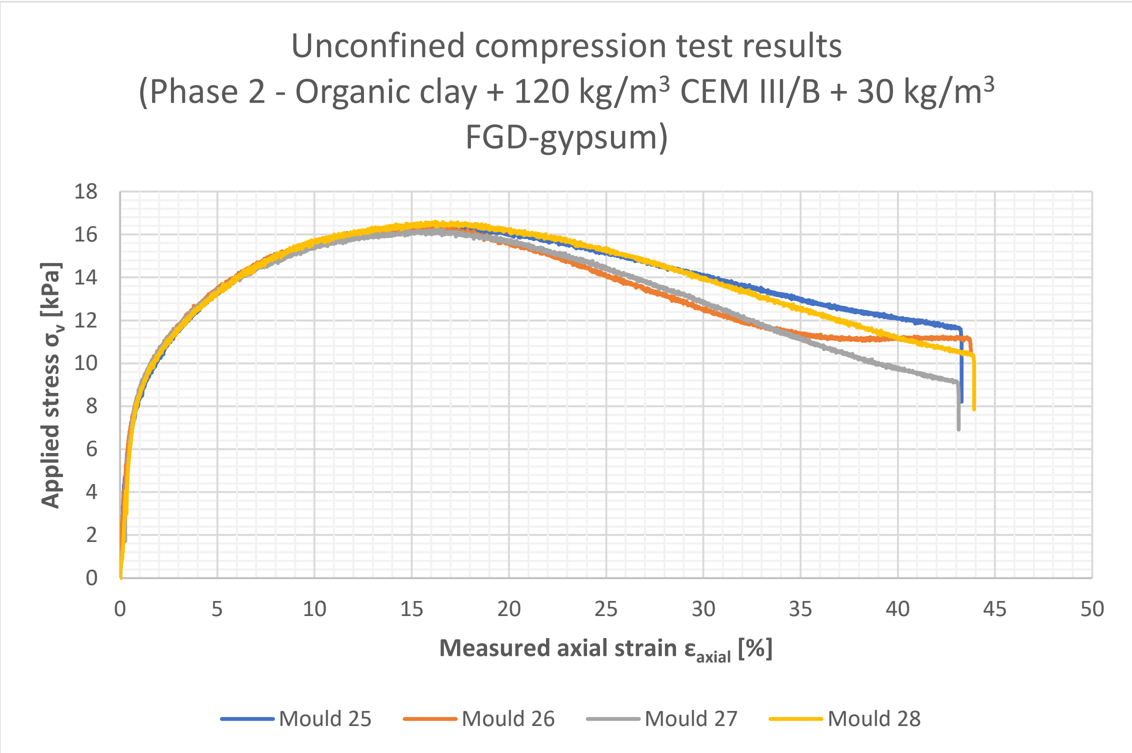


Figure F.47; The stress-strain diagrams obtained from the unconfined compression tests for the organic clay samples stabilised with 120 kg/m³ of blast-furnace slag cement (CEM III/B) and 30 kg/m³ of FGD-gypsum in phase 2 of the laboratory research.

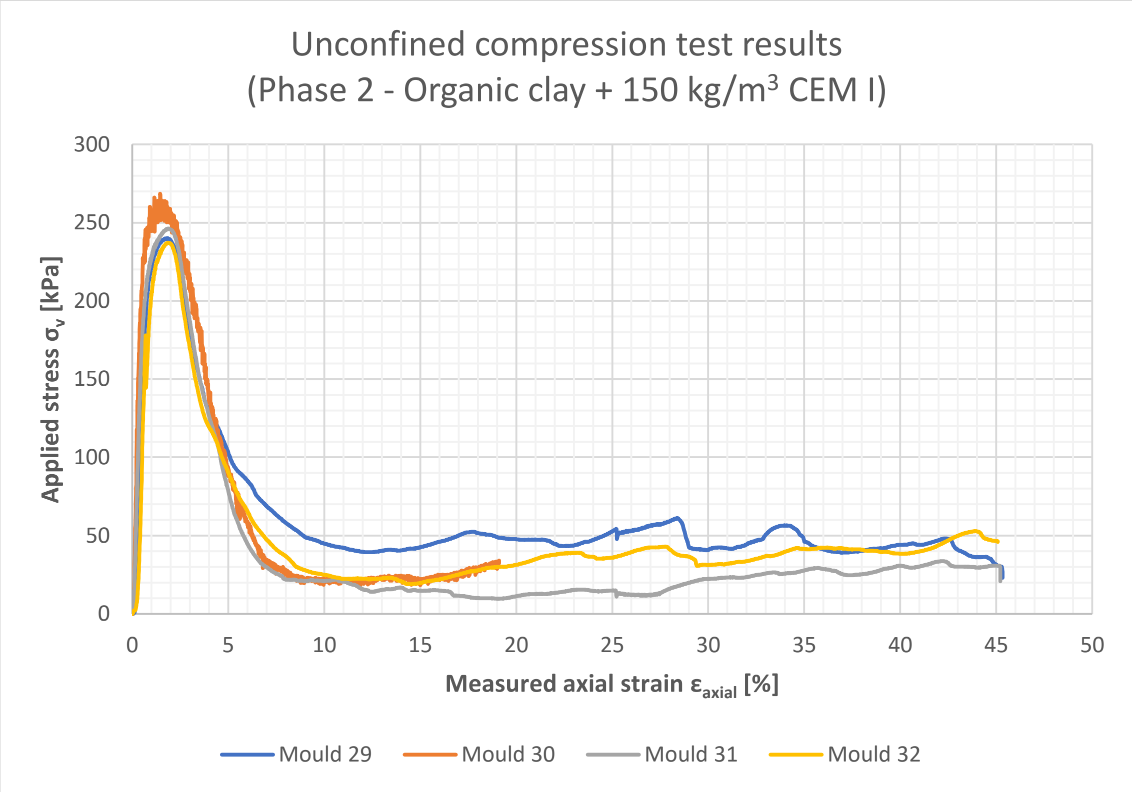


Figure F.48; The stress-strain diagrams obtained from the unconfined compression tests for the organic clay samples stabilised with 150 kg/m³ of Portland cement (CEM I) in phase 2 of the laboratory research.

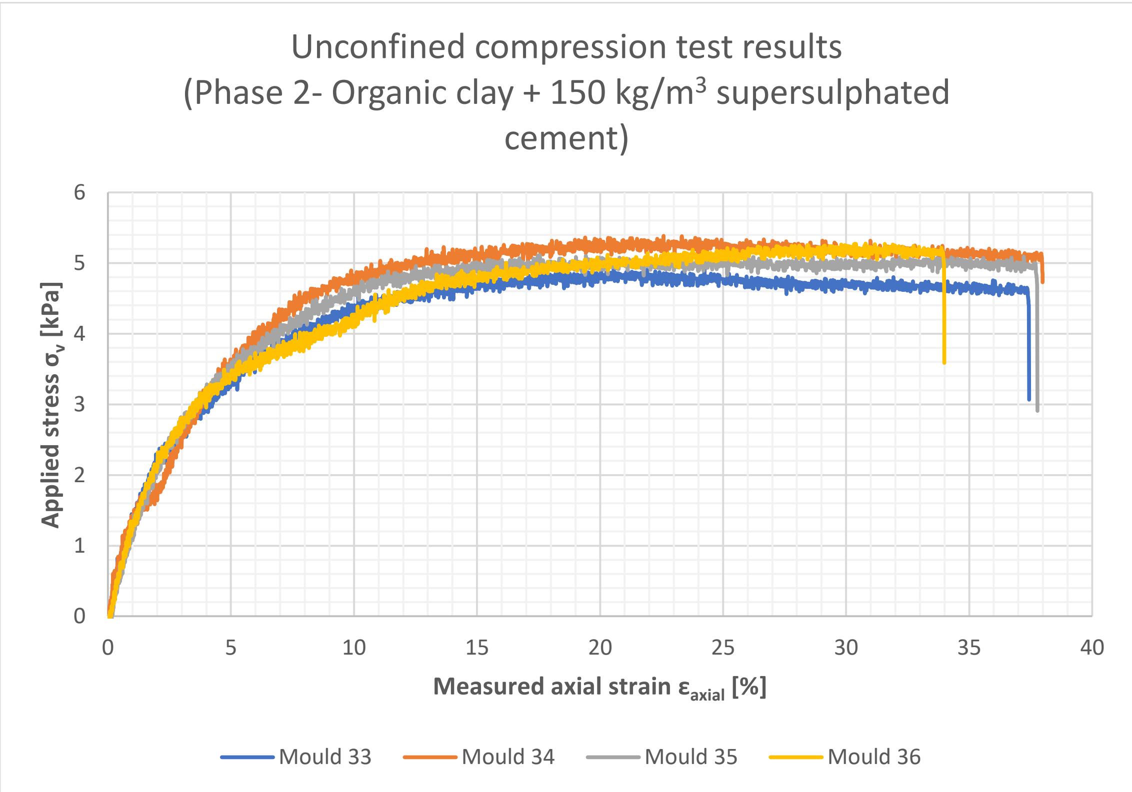


Figure F.49; The stress-strain diagrams obtained from the unconfined compression tests for the organic clay samples stabilised with 150 kg/m³ of supersulphated cement in phase 2 of the laboratory research. The supersulphated cement consisted of 127,5 kg/m³ of ground-granulated blast-furnace slag (GGBS), 15 kg/m³ of FGD-gypsum and 7,5 kg/m³ of Portland cement (CEM I).

Table F.31; Properties of the stabilised peat and organic clay samples as measured during the unconfined compression tests of phase 2 of the laboratory research. NM = not measured.

GENERAL INFORMATION MIXTURE					SAMPLE INFORMATION										
Soil type	**Binder**	**Binder dosage**	**Curing time**	**Mould number**	d	h	h/d	$\gamma_{stab;bulk}$	$\gamma_{stab;dry}$	w_{stab}	w_{nat} **(estimated)**	w/b	UCS	ε_f	**Test rate**
[-]	**[-]**	**[kg binder / m^3 soil]**	**[days]**	**[-]**	**[mm]**	**[mm]**	**[-]**	**[kN/m^3]**	**[kN/m^3]**	**[%]**	**[%]**	**[-]**	**[kPa]**	**[%]**	**[N/s]**
Peat	CEM III	150	7	05	150	266	1,77	11,93	4,15	188	324	5,2	242	2,6	20
Peat	CEM III	150	7	06	150	265	1,77	11,82	NM	NM	NM	5,2	236	3,5	
Peat	CEM III	150	7	07	150	266	1,77	11,73	NM	NM	NM	5,2	219	2,5	
Peat	CEM III	150	7	08	150	260	1,73	11,83	NM	NM	NM	5,2	221	2,7	
Peat	CEM III + FGD-gypsum	150	7	09	150	260	1,73	11,84	4,20	182	313	5,2	103	3,7	20
Peat	CEM III + FGD-gypsum	150	7	10	150	262	1,75	11,83	NM	NM	NM	5,2	94	3,4	
Peat	CEM III + FGD-gypsum	150	7	11	150	265	1,77	11,76	NM	NM	NM	5,2	98	3,7	
Peat	CEM III + FGD-gypsum	150	7	12	150	259	1,73	11,79	NM	NM	NM	5,2	95	3,6	
Peat	CEM I	150	7	13	150	266	1,77	11,97	4,51	165	282	5,2	344	2,3	25
Peat	CEM I	150	7	14	150	270	1,80	11,90	NM	NM	NM	5,2	352	1,2	
Peat	CEM I	150	7	15	150	273	1,82	12,05	NM	NM	NM	5,2	331	2,3	
Peat	CEM I	150	7	16	150	268	1,79	11,95	NM	NM	NM	5,2	364	2,4	

GENERAL INFORMATION MIXTURE					SAMPLE INFORMATION										
Soil type	**Binder**	**Binder dosage**	**Curing time**	**Mould number**	d	h	h/d	$\gamma_{stab;bulk}$	$\gamma_{stab;dry}$	w_{stab}	w_{nat} **(estimated)**	w/b	UCS	ε_f	**Test rate**
[-]	**[-]**	**[kg binder / m³ soil]**	**[days]**	**[-]**	**[mm]**	**[mm]**	**[-]**	**[kN/m³]**	**[kN/m³]**	**[%]**	**[%]**	**[-]**	**[kPa]**	**[%]**	**[N/s]**
Peat	Super-sulphated cement	150	7	17	150	250	1,67	12,65	4,40	187	313	5,1	16	19,5	10
Peat	Super-sulphated cement	150	7	18	150	259	1,73	12,62	NM	NM	NM	5,1	15	19,4	
Peat	Super-sulphated cement	150	7	19	150	253	1,69	12,65	NM	NM	NM	5,1	16	19,1	
Peat	Super-sulphated cement	150	7	20	150	252	1,68	12,68	NM	NM	NM	5,1	15	20,5	
Organic clay	CEM III	150	7	21	150	258	1,72	13,95	7,28	91,6	115	4,8	30	9,7	10
Organic clay	CEM III	150	7	22	150	271	1,81	14,05	NM	NM	NM	4,8	30	11,5	
Organic clay	CEM III	150	7	23	150	266	1,77	14,07	NM	NM	NM	4,8	29	12,0	
Organic clay	CEM III	150	7	24	150	264	1,76	14,15	NM	NM	NM	4,8	29	12,6	

GENERAL INFORMATION MIXTURE					**SAMPLE INFORMATION**										
Soil type	**Binder**	**Binder dosage**	**Curing time**	**Mould number**	d	h	h/d	$\gamma_{stab;bulk}$	$\gamma_{stab;dry}$	w_{stab}	w_{nat} **(estimated)**	w/b	UCS	ε_f	**Test rate**
[-]	**[-]**	**[kg binder / m³ soil]**	**[days]**	**[-]**	**[mm]**	**[mm]**	**[-]**	**[kN/m³]**	**[kN/m³]**	**[%]**	**[%]**	**[-]**	**[kPa]**	**[%]**	**[N/s]**
Organic clay	CEM III + FGD-gypsum	150	7	25	150	264	1,76	13,91	7,14	94,7	120	4,9	16	16,6	5
Organic clay	CEM III + FGD-gypsum	150	7	26	150	266	1,77	13,87	NM	NM	NM	4,9	16	16,1	
Organic clay	CEM III + FGD-gypsum	150	7	27	150	262	1,75	13,91	NM	NM	NM	4,9	16	16,2	
Organic clay	CEM III + FGD-gypsum	150	7	28	150	264	1,76	13,93	NM	NM	NM	4,9	17	16,2	
Organic clay	CEM I	150	7	29	150	272	1,81	13,85	7,48	85,1	108	5,0	240	1,9	20
Organic clay	CEM I	150	7	30	150	269	1,79	13,95	NM	NM	NM	5,0	268	1,5	
Organic clay	CEM I	150	7	31	150	272	1,81	13,89	NM	NM	NM	5,0	246	1,9	
Organic clay	CEM I	150	7	32	150	269	1,79	14,07	NM	NM	NM	5,0	237	1,9	

GENERAL INFORMATION MIXTURE					SAMPLE INFORMATION											
Soil type	Binder	Binder dosage	Curing time	Mould number	d	h	h/d	$\gamma_{stab;bulk}$	$\gamma_{stab;dry}$	w_{stab}	w_{nat} (estimated)	w/b	UCS	ε_f	Test rate
[-]	[-]	[kg binder / m³ soil]	[days]	[-]	[mm]	[mm]	[-]	[kN/m³]	[kN/m³]	[%]	[%]	[-]	[kPa]	[%]	[N/s]
Organic clay	Super-sulphated cement	150	7	33	150	245	1,63	14,41	7,68	87,7	111	4,9	5	20,6	5
Organic clay	Super-sulphated cement	150	7	34	150	245	1,63	14,23	NM	NM	NM	4,9	5	22,6	
Organic clay	Super-sulphated cement	150	7	35	150	245	1,63	14,33	NM	NM	NM	4,9	5	17,5	
Organic clay	Super-sulphated cement	150	7	36	150	236	1,57	14,50	NM	NM	NM	4,9	5	30,8	

F.3 Phase 3 – Binder dosage selection

F.3.1 Settlement curves (laboratory stabilisation procedure)

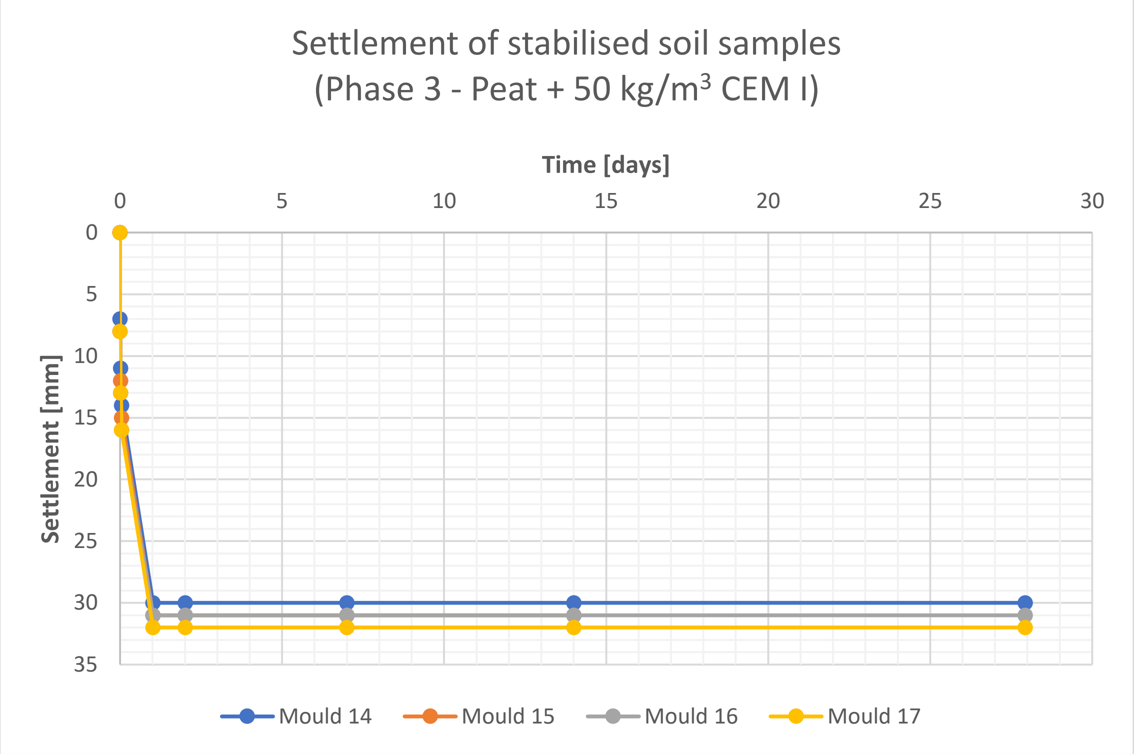


Figure F.50; The measured settlement of the peat samples stabilised with 50 kg/m³ of Portland cement (CEM I) in phase 3 of the laboratory research.

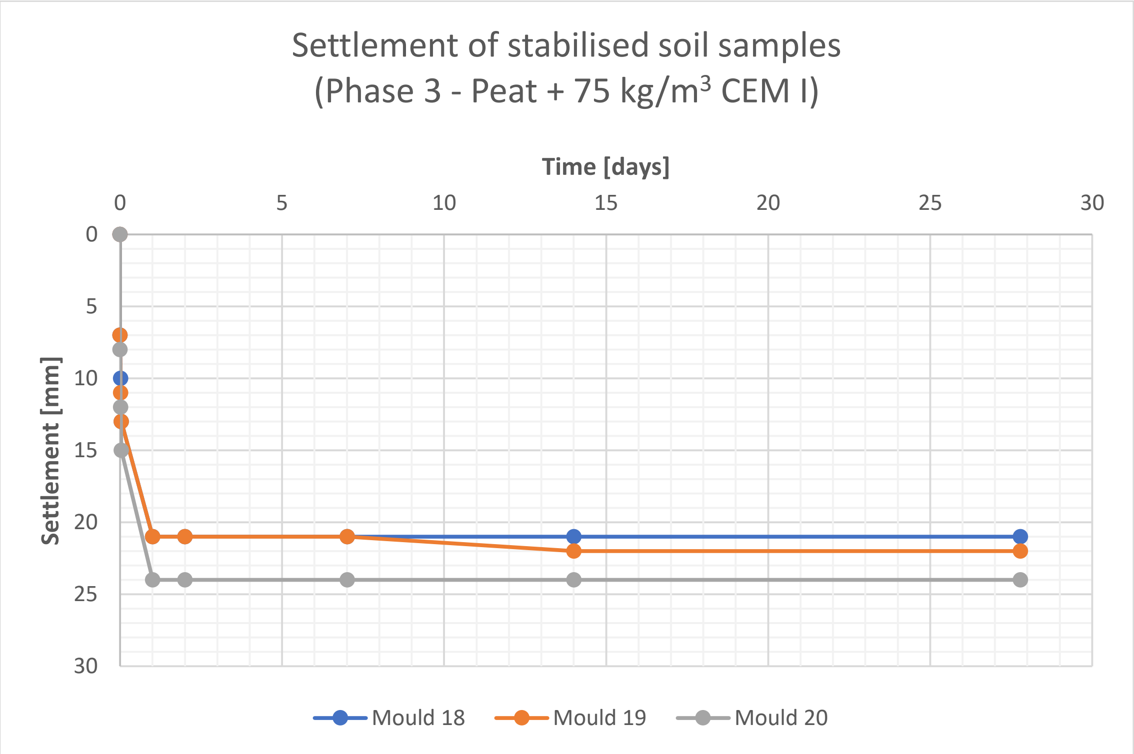


Figure F.51; The measured settlement of the peat samples stabilised with 75 kg/m³ of Portland cement (CEM I) in phase 3 of the laboratory research.

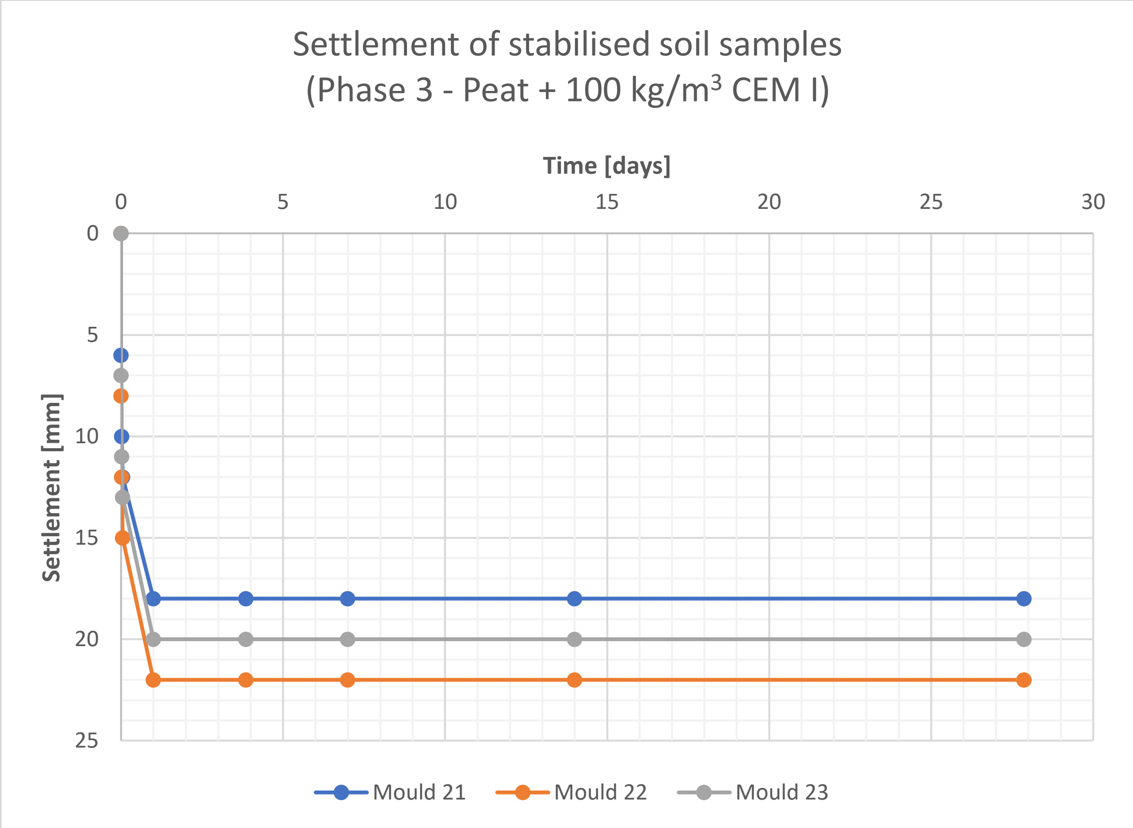


Figure F.52; The measured settlement of the peat samples stabilised with 100 kg/m³ of Portland cement (CEM I) in phase 3 of the laboratory research.

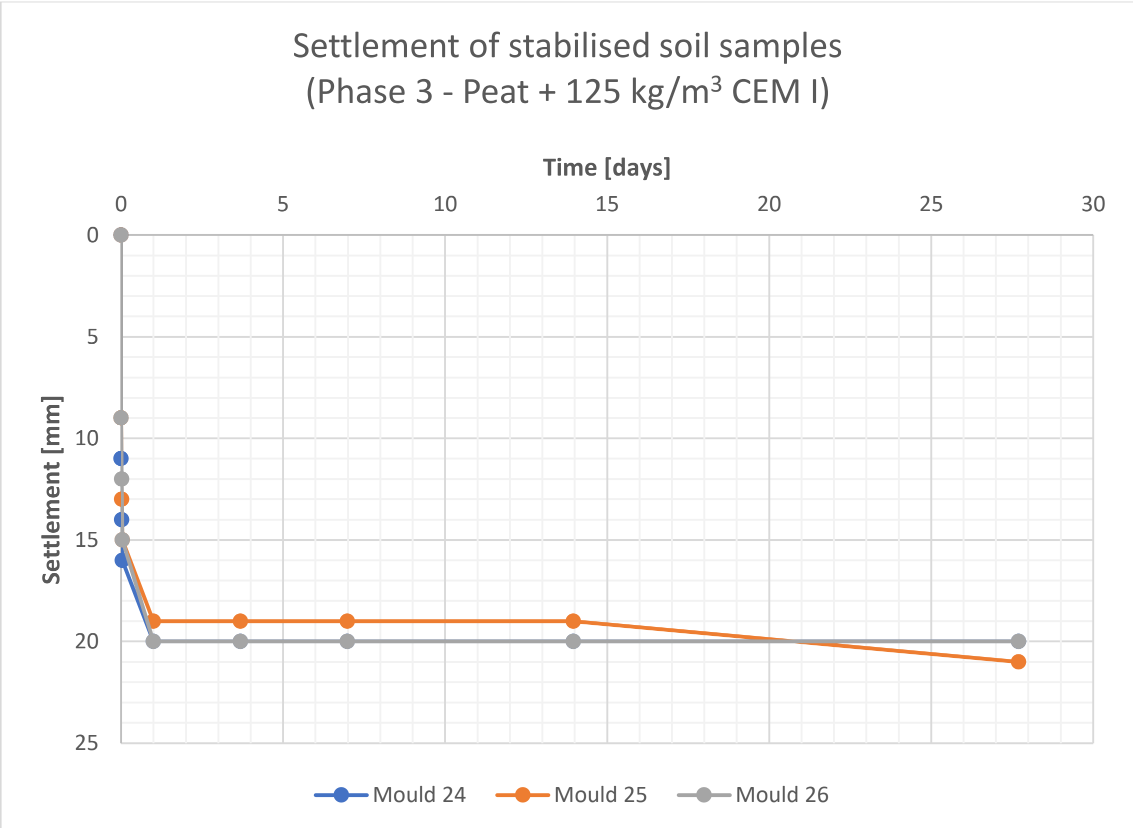


Figure F.53; The measured settlement of the peat samples stabilised with 125 kg/m³ of Portland cement (CEM I) in phase 3 of the laboratory research.

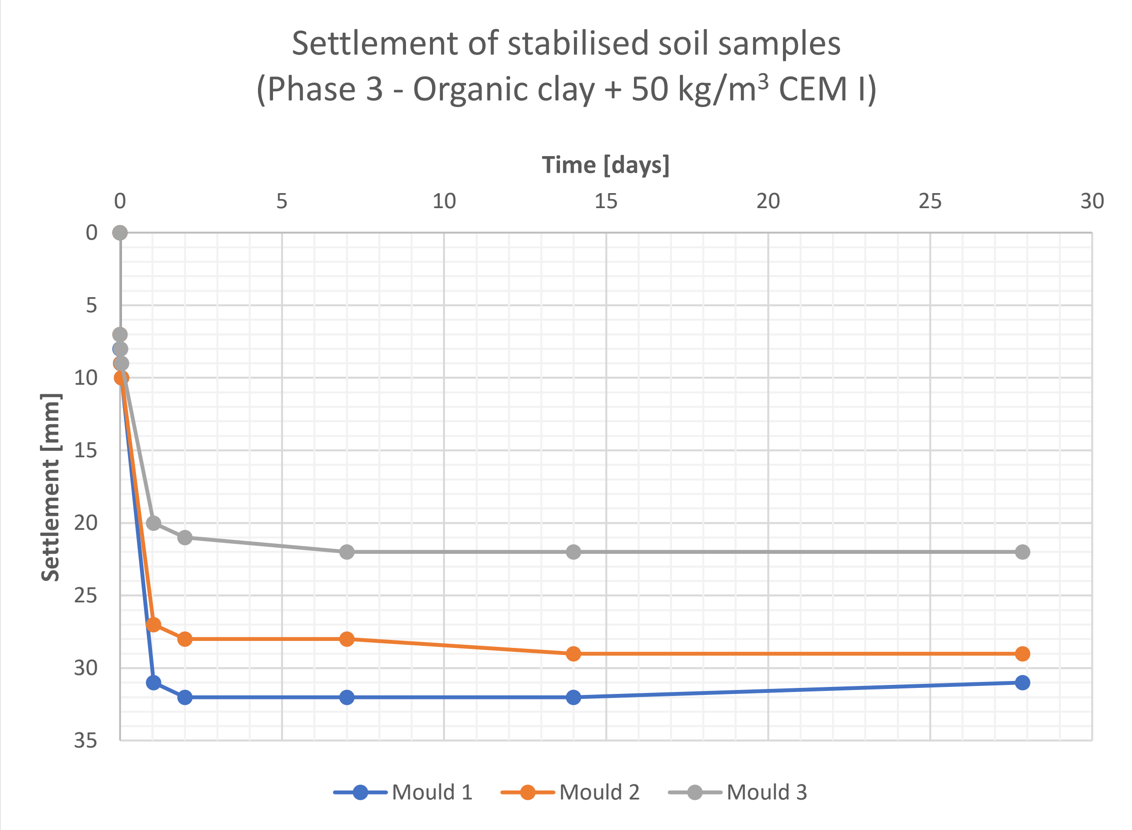


Figure F.54; The measured settlement of the organic clay samples stabilised with 50 kg/m³ of Portland cement (CEM I) in phase 3 of the laboratory research.

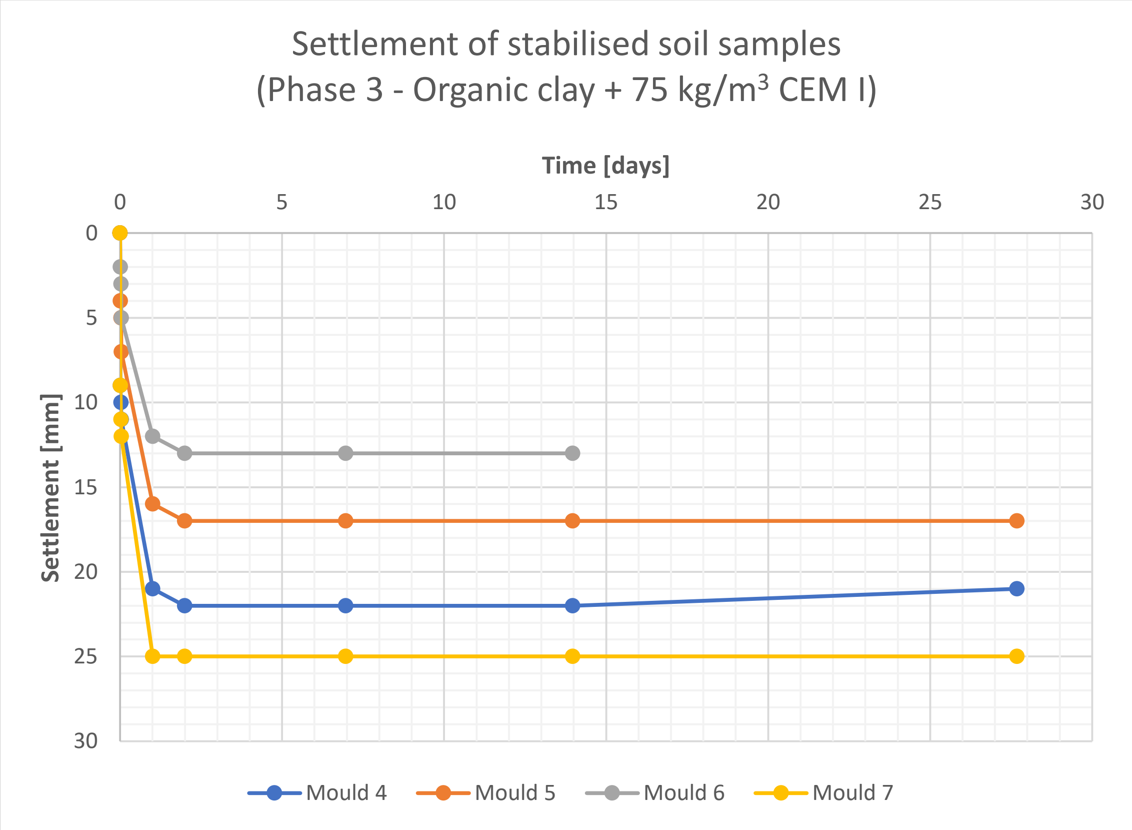


Figure F.55; The measured settlement of the organic clay samples stabilised with 75 kg/m³ of Portland cement (CEM I) in phase 3 of the laboratory research.

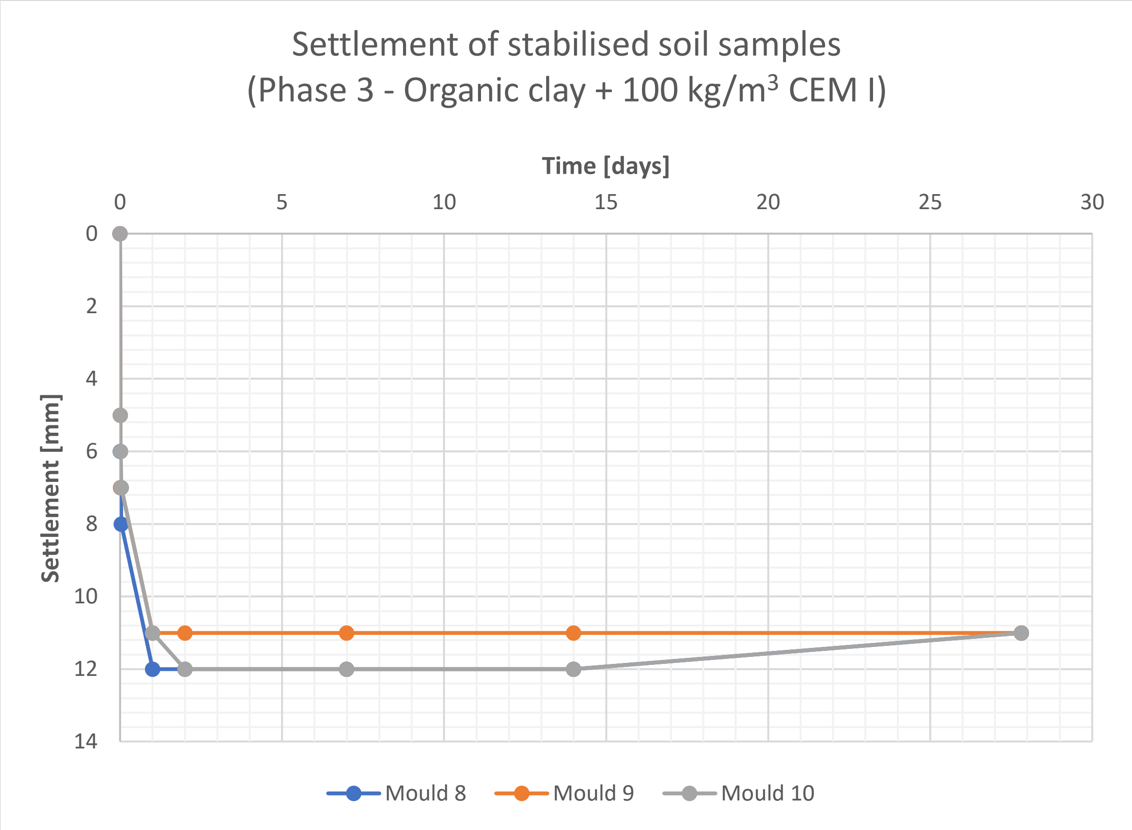


Figure F.56; The measured settlement of the organic clay samples stabilised with 100 kg/m³ of Portland cement (CEM I) in phase 3 of the laboratory research.

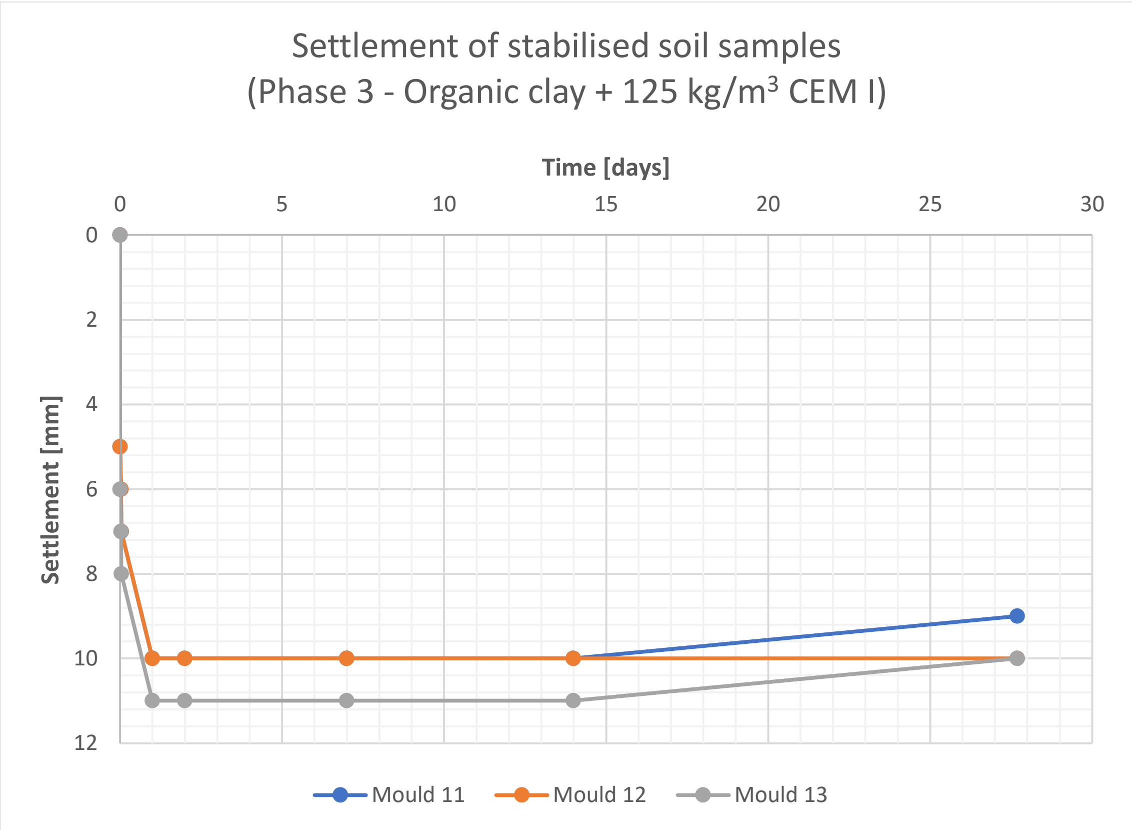


Figure F.57; The measured settlement of the organic clay samples stabilised with 125 kg/m³ of Portland cement (CEM I) in phase 3 of the laboratory research.

F.3.2 Unit weight of stabilised soil samples (laboratory stabilisation procedure)

Table F.32; The measured unit weights of the different mixtures during phase 3 of the laboratory research. The (?) indicate that the measured unit weights were deemed unreliable.

Mixture	Mould number	Bulk unit weight directly after filling the mould [kN/m³]	Bulk unit weight after 28 days of loading [kN/m³]	Bulk unit weight of extruded sample [kN/m³]
PEAT + 50 kg/m³ CEM I	14	11,1	11,4	11,3
	15	11,1	11,5	11,4
	16	11,0	11,4	11,2
	17	11,1	11,5	11,4
PEAT + 75 kg/m³ CEM I	18	11,3	11,7	11,6
	19	11,3	11,7	11,6
	20	11,3	11,7	11,6
PEAT + 100 kg/m³ CEM I	21	11,5	11,8	11,6
	22	11,4	11,8	11,7
	23	11,4	11,8	11,7
PEAT + 125 kg/m³ CEM I	24	11,6	12,0	11,9
	25	11,5	12,0	11,9
	26	11,5	11,9	11,8
ORGANIC CLAY + 50 kg/m³ CEM I	1	12,6 (?)	13,7	13,5
	2	12,8 (?)	13,7	13,7
	3	13,2 (?)	13,8	13,7
ORGANIC CLAY + 75 kg/m³ CEM I	4	13,0 (?)	13,8	13,7
	5	13,3 (?)	13,8	13,7
	6	13,5 (?)	13,9	13,7
	7	12,8 (?)	13,7	13,6
ORGANIC CLAY + 100 kg/m³ CEM I	8	13,7	14,1	13,9
	9	13,7	14,1	13,9
	10	13,6	14,0	13,9
ORGANIC CLAY + 125 kg/m³ CEM I	11	13,9	14,3	14,1
	12	13,8	14,3	14,1
	13	13,9	14,3	14,1

F.3.3 Unconfined compression tests

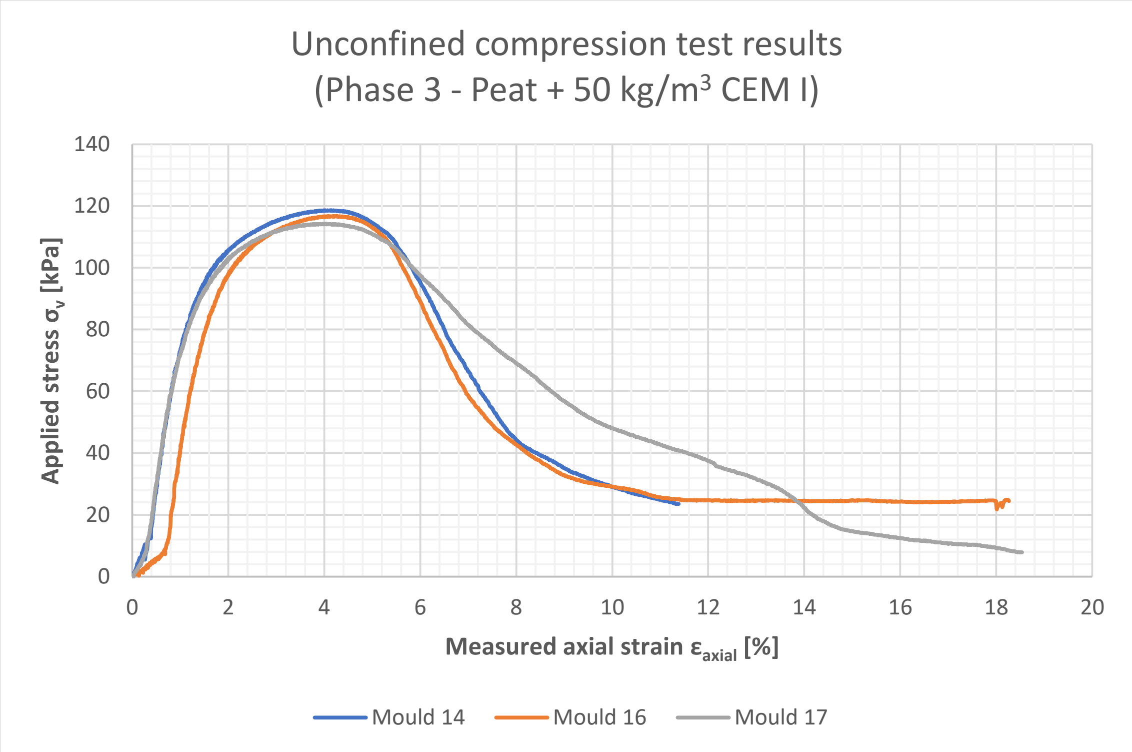


Figure F.58; The stress-strain diagrams obtained from the unconfined compression tests for the peat samples stabilised with 50 kg/m³ of Portland cement (CEM I) in phase 3 of the laboratory research.

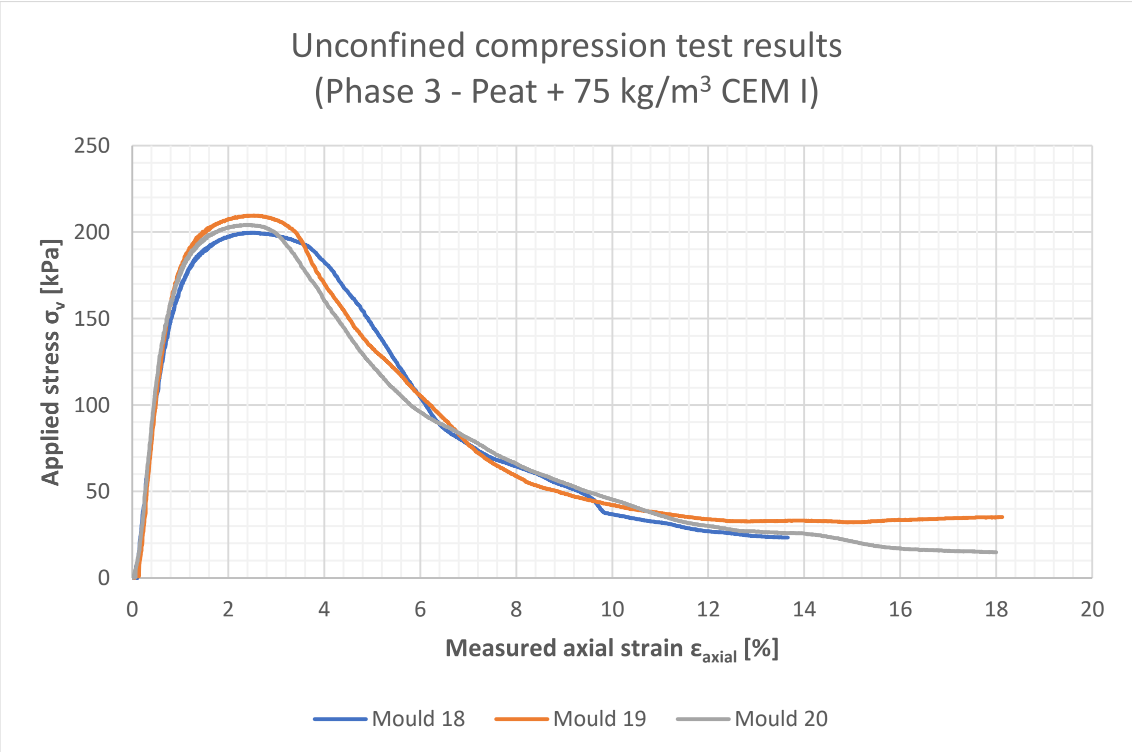


Figure F.59; The stress-strain diagrams obtained from the unconfined compression tests for the peat samples stabilised with 75 kg/m³ of Portland cement (CEM I) in phase 3 of the laboratory research.

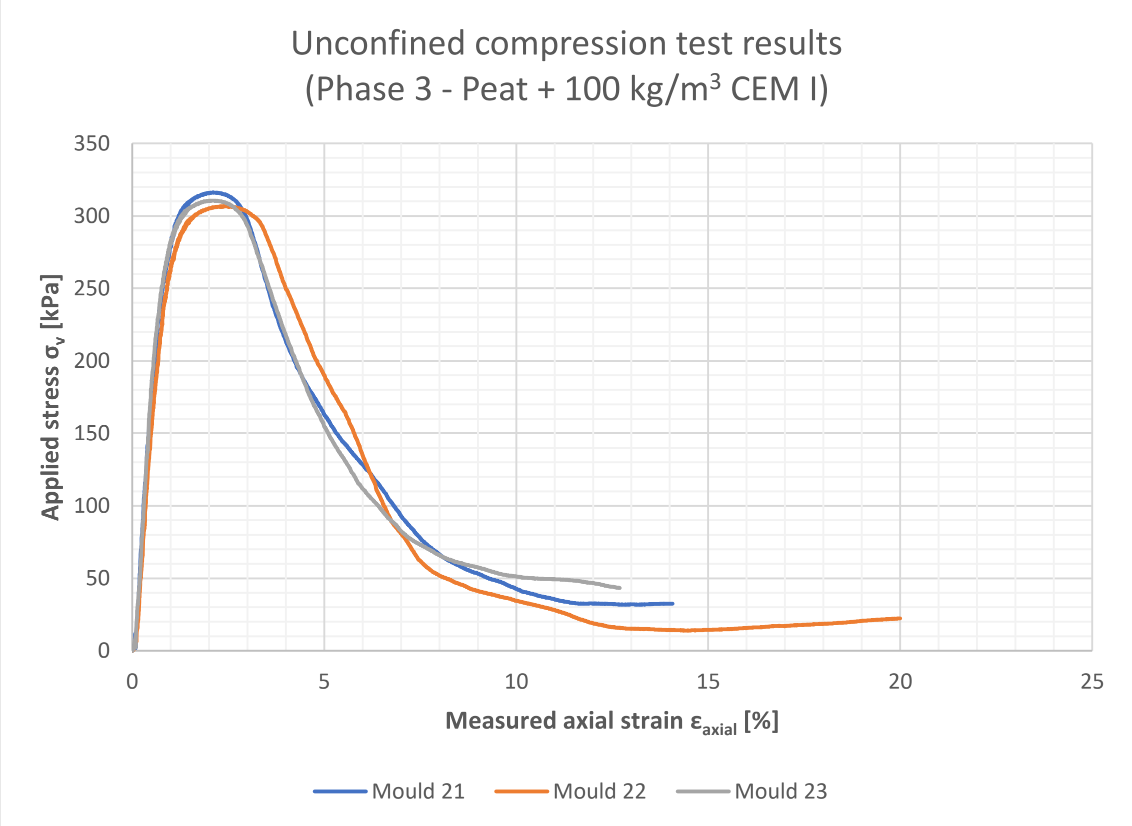


Figure F.60; The stress-strain diagrams obtained from the unconfined compression tests for the peat samples stabilised with 100 kg/m³ of Portland cement (CEM I) in phase 3 of the laboratory research.

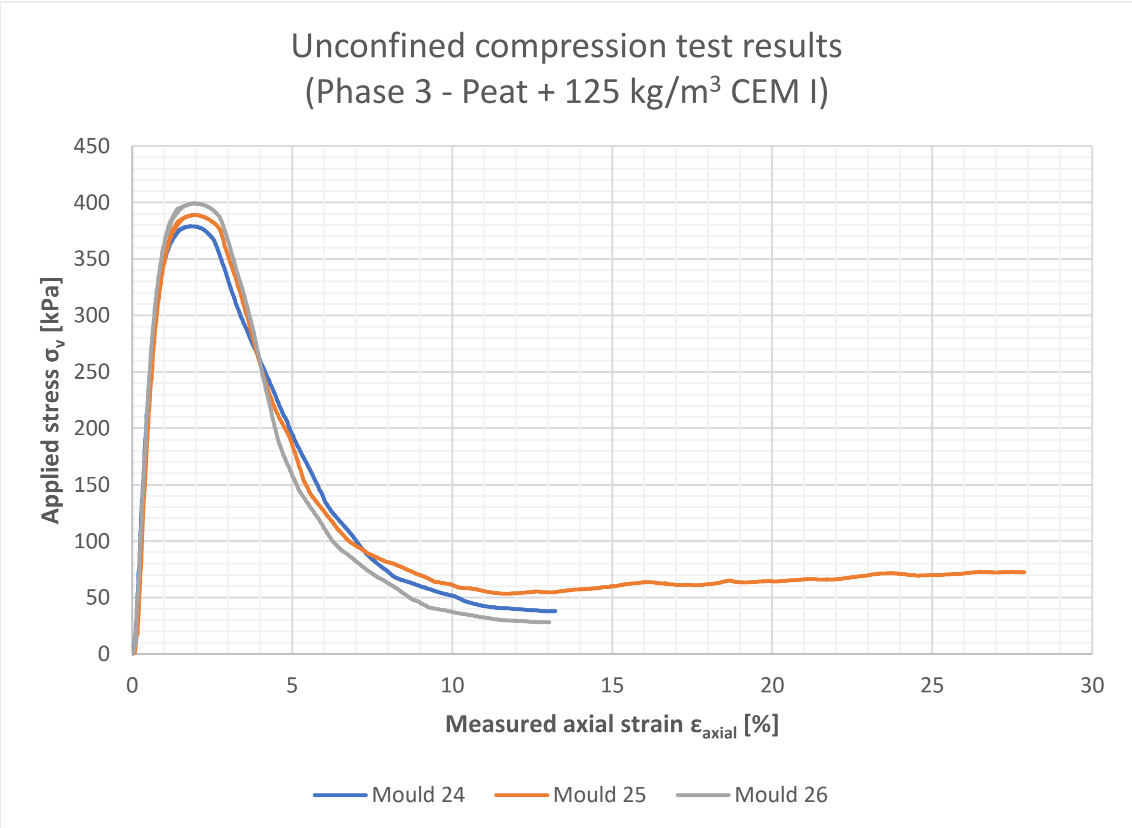


Figure F.61; The stress-strain diagrams obtained from the unconfined compression tests for the peat samples stabilised with 125 kg/m³ of Portland cement (CEM I) in phase 3 of the laboratory research.

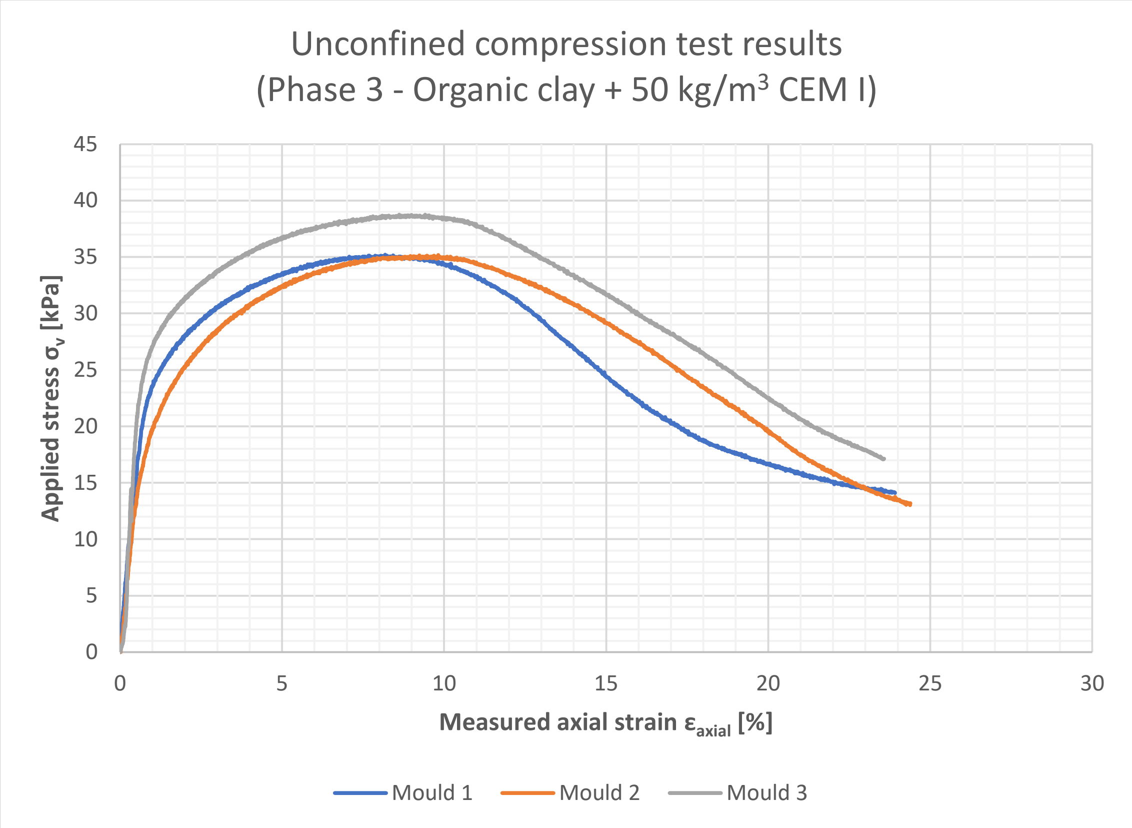


Figure F.62; The stress-strain diagrams obtained from the unconfined compression tests for the organic clay samples stabilised with 50 kg/m³ of Portland cement (CEM I) in phase 3 of the laboratory research.

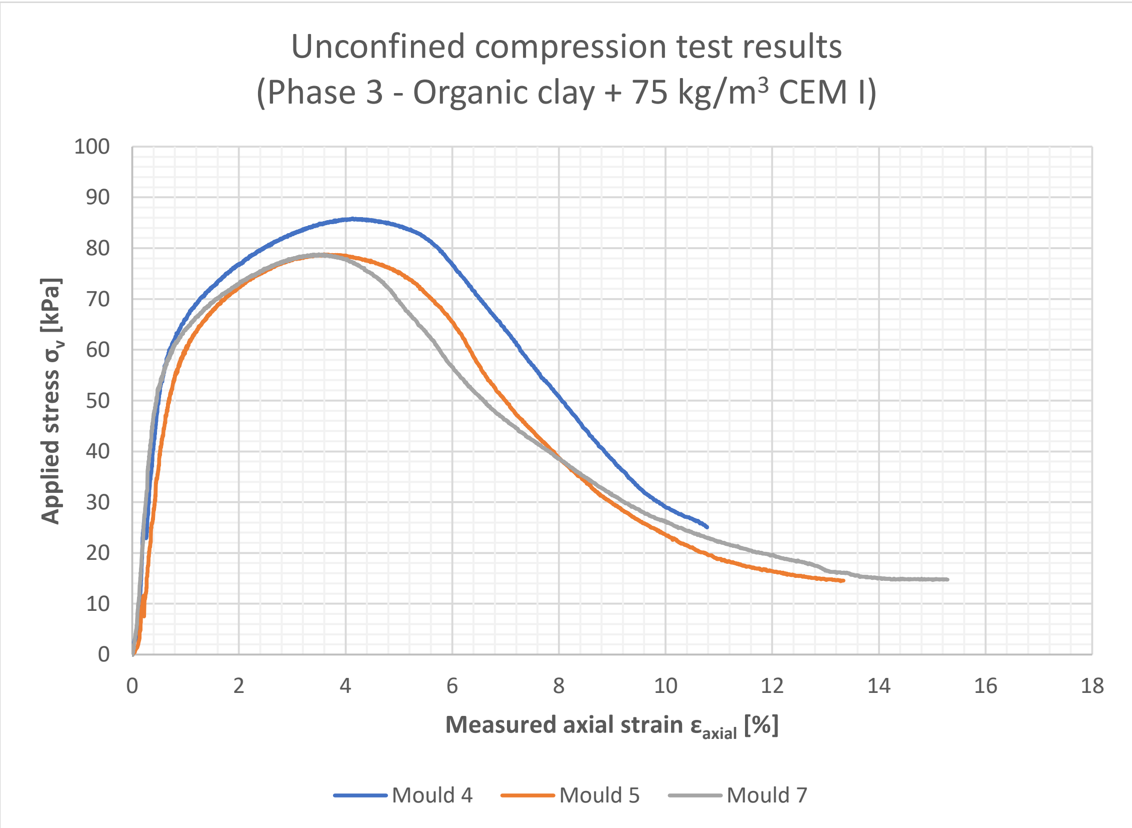


Figure F.63; The stress-strain diagrams obtained from the unconfined compression tests for the organic clay samples stabilised with 75 kg/m³ of Portland cement (CEM I) in phase 3 of the laboratory research.

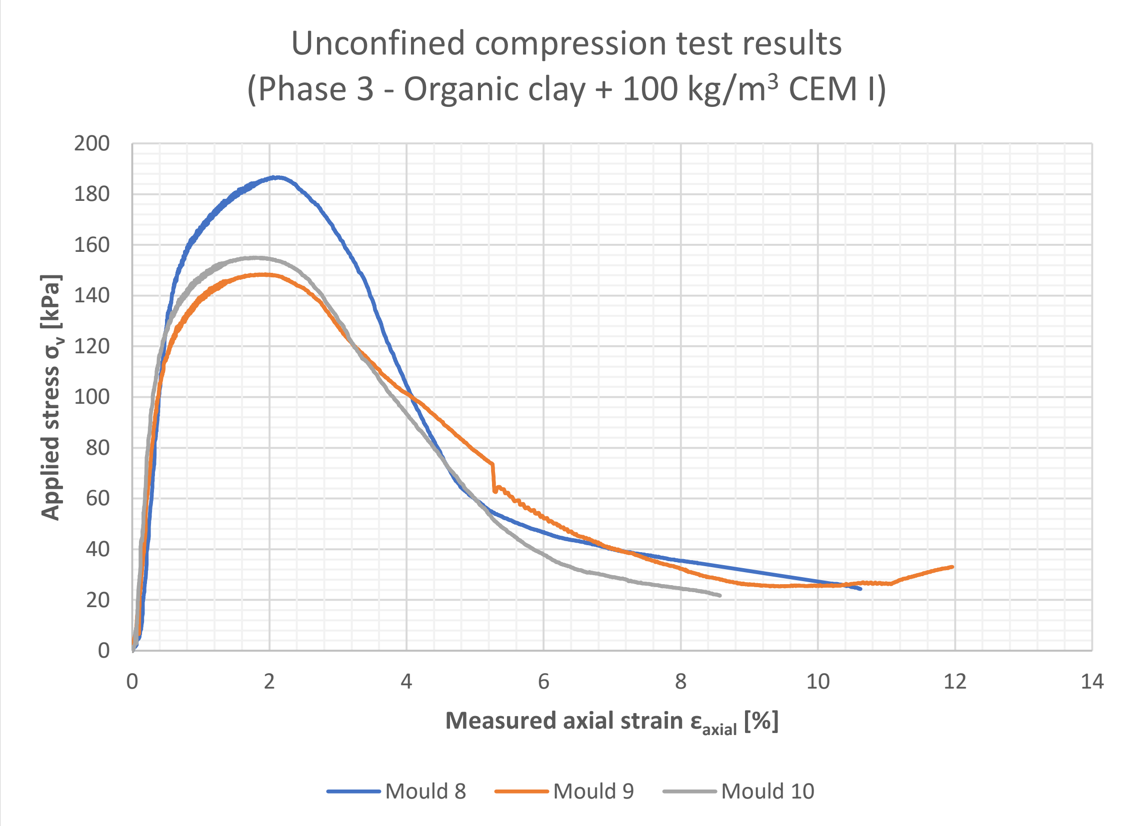


Figure F.64; The stress-strain diagrams obtained from the unconfined compression tests for the organic clay samples stabilised with 100 kg/m³ of Portland cement (CEM I) in phase 3 of the laboratory research.

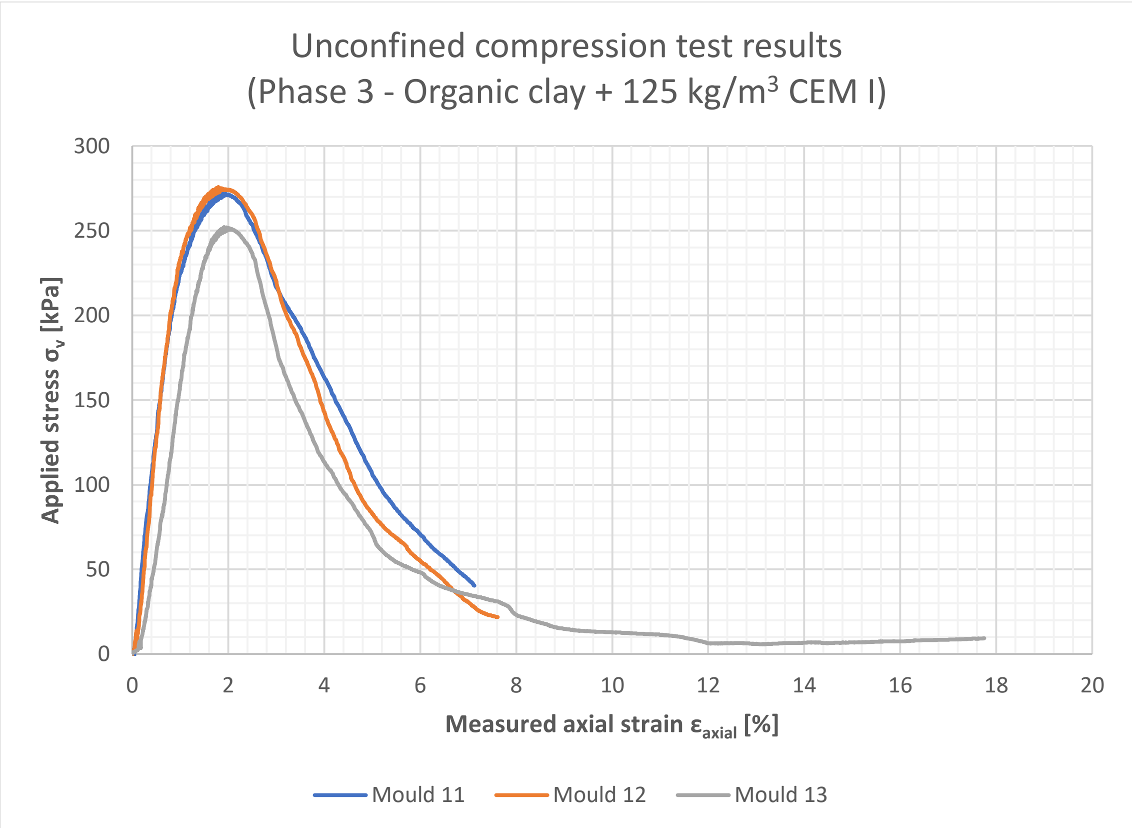


Figure F.65; The stress-strain diagrams obtained from the unconfined compression tests for the organic clay samples stabilised with 125 kg/m³ of Portland cement (CEM I) in phase 3 of the laboratory research.

Table F.33; Properties of the stabilised peat and organic clay samples as measured during the unconfined compression tests of phase 3 of the laboratory research. NM = not measured.

GENERAL INFORMATION MIXTURE					**SAMPLE INFORMATION**											
Soil type	**Binder**	**Binder dosage**	**Curing time**	**Mould number**	d	h	h/d	$\gamma_{stab;bulk}$	$\gamma_{stab;dry}$	w_{stab}	w_{nat} (estimated)	w/b	UCS	ε_f	Test rate
[-]	**[-]**	**[kg binder / m³ soil]**	**[days]**	**[-]**	**[mm]**	**[mm]**	**[-]**	**[kN/m³]**	**[kN/m³]**	**[%]**	**[%]**	**[-]**	**[kPa]**	**[%]**	**[N/s]**
Peat	CEM I	50	28	14	150	243	1,62	11,28	3,42	229	283	15,5	119	4,1	10
Peat	CEM I	50	28	16	150	248	1,65	11,23	3,35	235	290	15,5	117	4,2	
Peat	CEM I	50	28	17	150	238	1,59	11,37	3,42	232	286	15,5	114	4,0	
Peat	CEM I	75	28	18	150	259	1,73	11,57	3,71	212	283	10,2	200	2,5	15
Peat	CEM I	75	28	19	150	256	1,71	11,56	3,67	215	287	10,2	210	2,5	
Peat	CEM I	75	28	20	150	257	1,71	11,56	3,68	214	286	10,2	204	2,4	
Peat	CEM I	100	28	21	150	257	1,71	11,64	3,89	199	293	7,8	316	2,1	20
Peat	CEM I	100	28	22	150	260	1,73	11,66	3,90	199	293	7,8	307	2,5	
Peat	CEM I	100	28	23	150	259	1,73	11,66	3,85	203	298	7,8	311	2,1	
Peat	CEM I	125	28	24	150	259	1,73	11,90	4,23	182	289	6,2	379	1,8	30
Peat	CEM I	125	28	25	150	259	1,73	11,87	4,20	183	290	6,2	389	2,0	
Peat	CEM I	125	28	26	150	259	1,73	11,79	4,26	177	281	6,2	399	1,9	
Organic clay	CEM I	50	28	01	150	257	1,71	13,54	6,58	106	115	14,8	35	8,2	10
Organic clay	CEM I	50	28	02	150	262	1,75	13,66	6,64	106	115	14,8	35	9,8	
Organic clay	CEM I	50	28	03	150	255	1,70	13,66	6,74	103	112	14,8	39	9,4	
Organic clay	CEM I	75	28	04	150	264	1,76	13,71	6,85	100	114	9,8	86	4,1	10
Organic clay	CEM I	75	28	05	150	269	1,79	13,72	6,76	103	117	9,8	79	3,6	
Organic clay	CEM I	75	28	07	150	265	1,77	13,62	6,71	103	117	9,8	79	3,5	

GENERAL INFORMATION MIXTURE					SAMPLE INFORMATION										
Soil type	**Binder**	**Binder dosage**	**Curing time**	**Mould number**	d	h	h/d	$\gamma_{stab;bulk}$	$\gamma_{stab;dry}$	w_{stab}	w_{nat} (estimated)	w/b	UCS	ε_f	Test rate
[-]	**[-]**	**[kg binder / m³ soil]**	**[days]**	**[-]**	**[mm]**	**[mm]**	**[-]**	**[kN/m³]**	**[kN/m³]**	**[%]**	**[%]**	**[-]**	**[kPa]**	**[%]**	**[N/s]**
Organic clay	CEM I	100	28	08	150	268	1,79	13,93	7,18	93,9	110	7,2	187	2,1	10
Organic clay	CEM I	100	28	09	150	265	1,77	13,95	7,18	94,3	110	7,2	145	1,9	
Organic clay	CEM I	100	28	10	150	268	1,79	13,90	7,15	94,5	111	7,2	155	1,8	
Organic clay	CEM I	125	28	11	150	266	1,77	14,14	7,54	87,6	106	5,7	272	1,8	15
Organic clay	CEM I	125	28	12	150	262	1,75	14,13	7,65	84,8	103	5,7	276	1,8	
Organic clay	CEM I	125	28	13	150	265	1,77	14,14	7,46	89,7	109	5,7	252	1,9	

F.4 Phase 4 – UCS curing curve determination

F.4.1 Settlement curves (laboratory stabilisation procedure)

F.4.1.1 Method 1 (multiple batches)

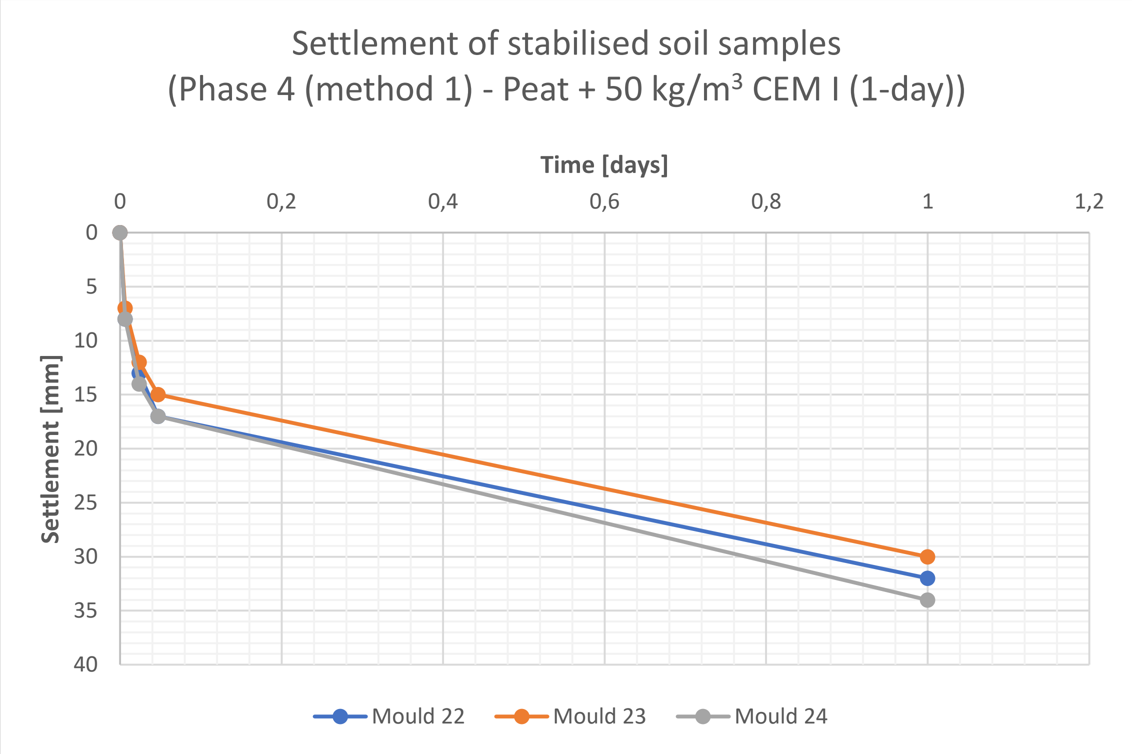


Figure F.66; The measured settlement of the peat samples stabilised with 50 kg/m³ of Portland cement (CEM I) in phase 4 (method 1) of the laboratory research. These samples were left to cure for 24 hours.

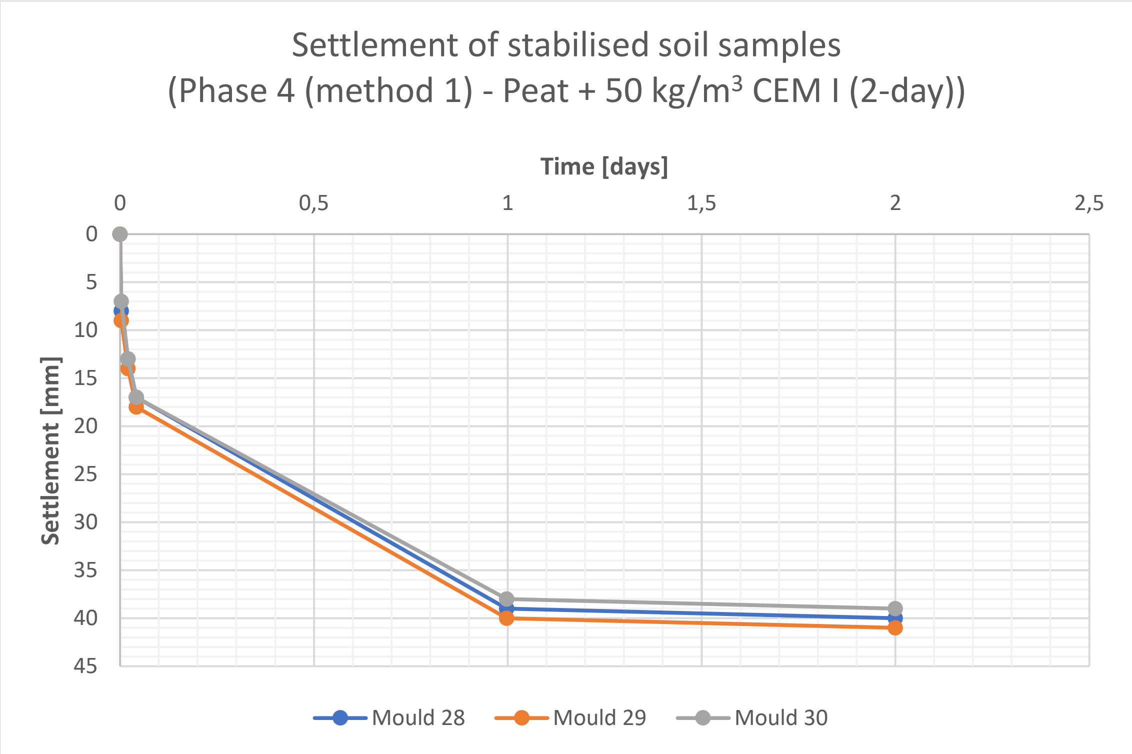


Figure F.67; The measured settlement of the peat samples stabilised with 50 kg/m³ of Portland cement (CEM I) in phase 4 (method 1) of the laboratory research. These samples were left to cure for 48 hours.

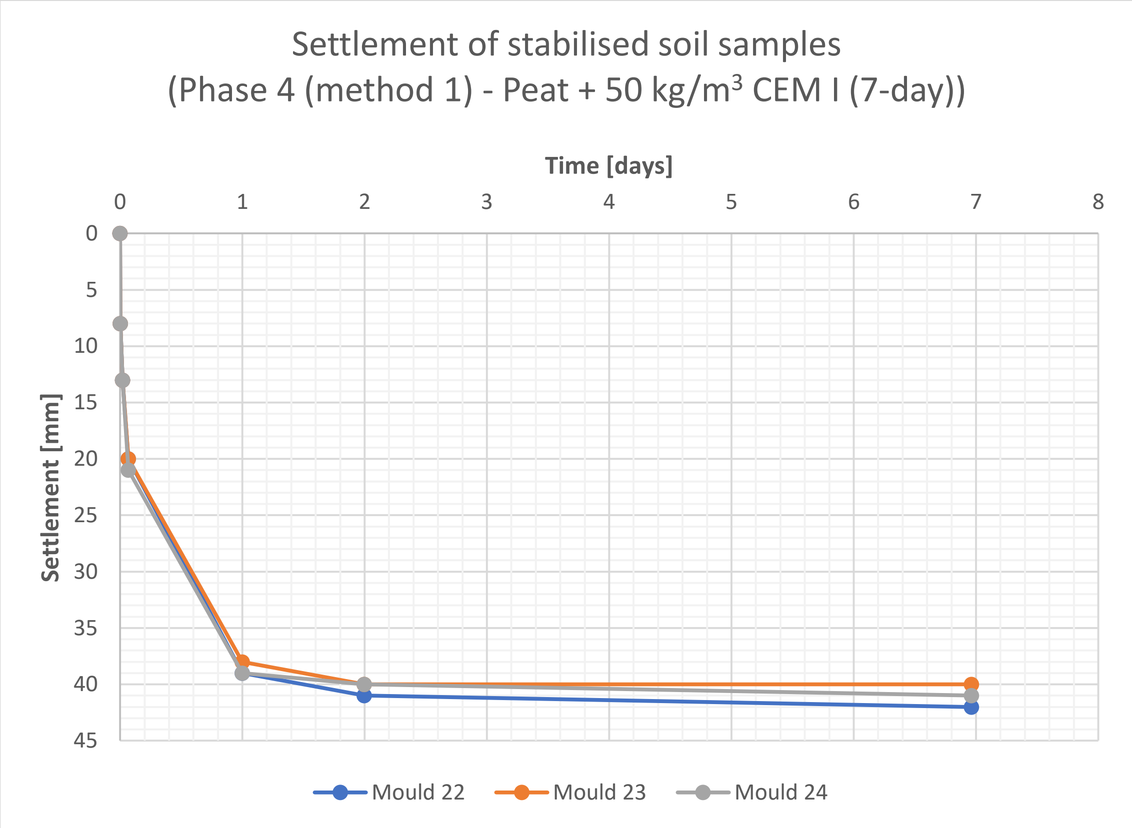


Figure F.68; The measured settlement of the peat samples stabilised with 50 kg/m³ of Portland cement (CEM I) in phase 4 (method 1) of the laboratory research. These samples were left to cure for 7 days.

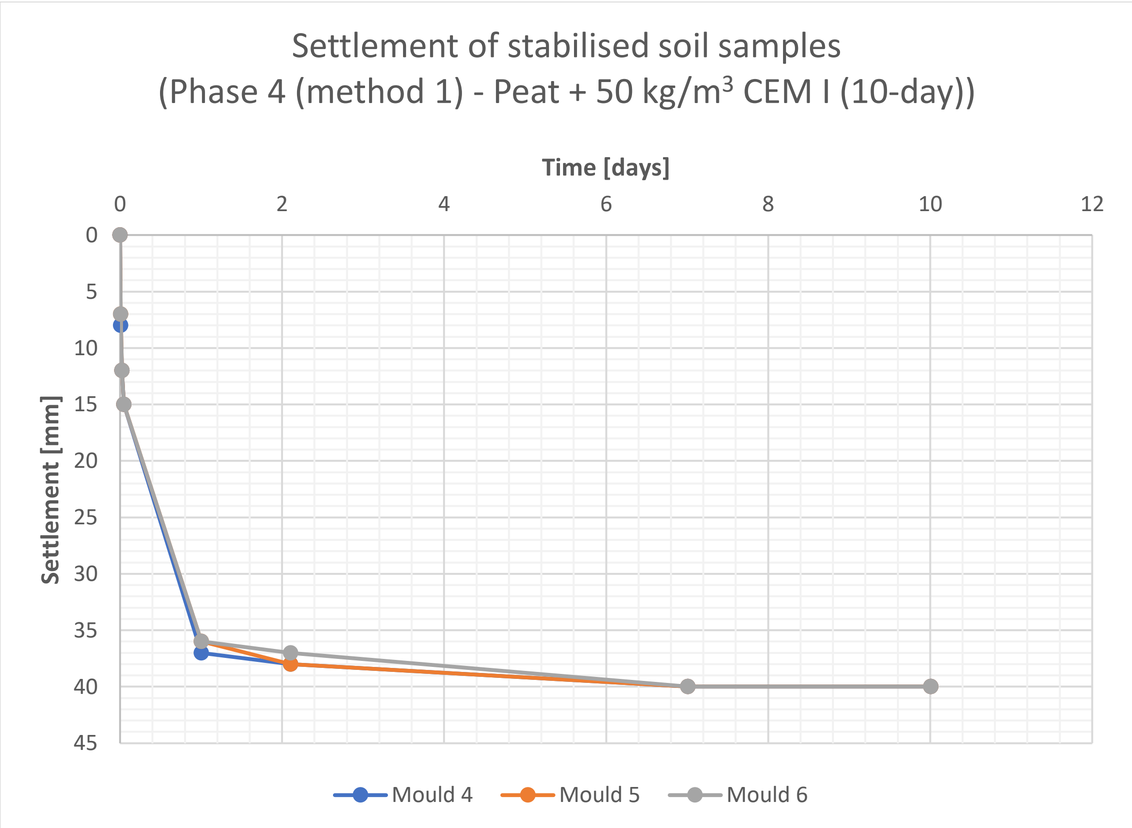


Figure F.69; The measured settlement of the peat samples stabilised with 50 kg/m³ of Portland cement (CEM I) in phase 4 (method 1) of the laboratory research. These samples were left to cure for 10 days.

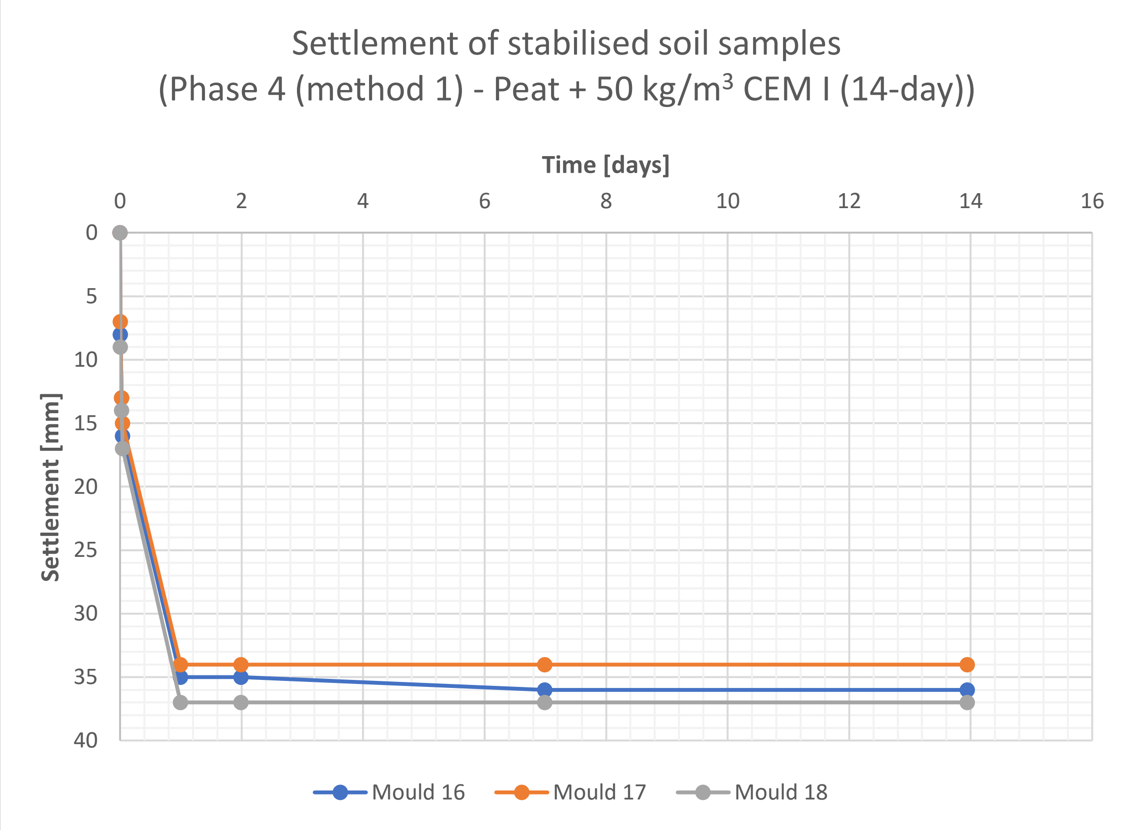


Figure F.70; The measured settlement of the peat samples stabilised with 50 kg/m³ of Portland cement (CEM I) in phase 4 (method 1) of the laboratory research. These samples were left to cure for 14 days.

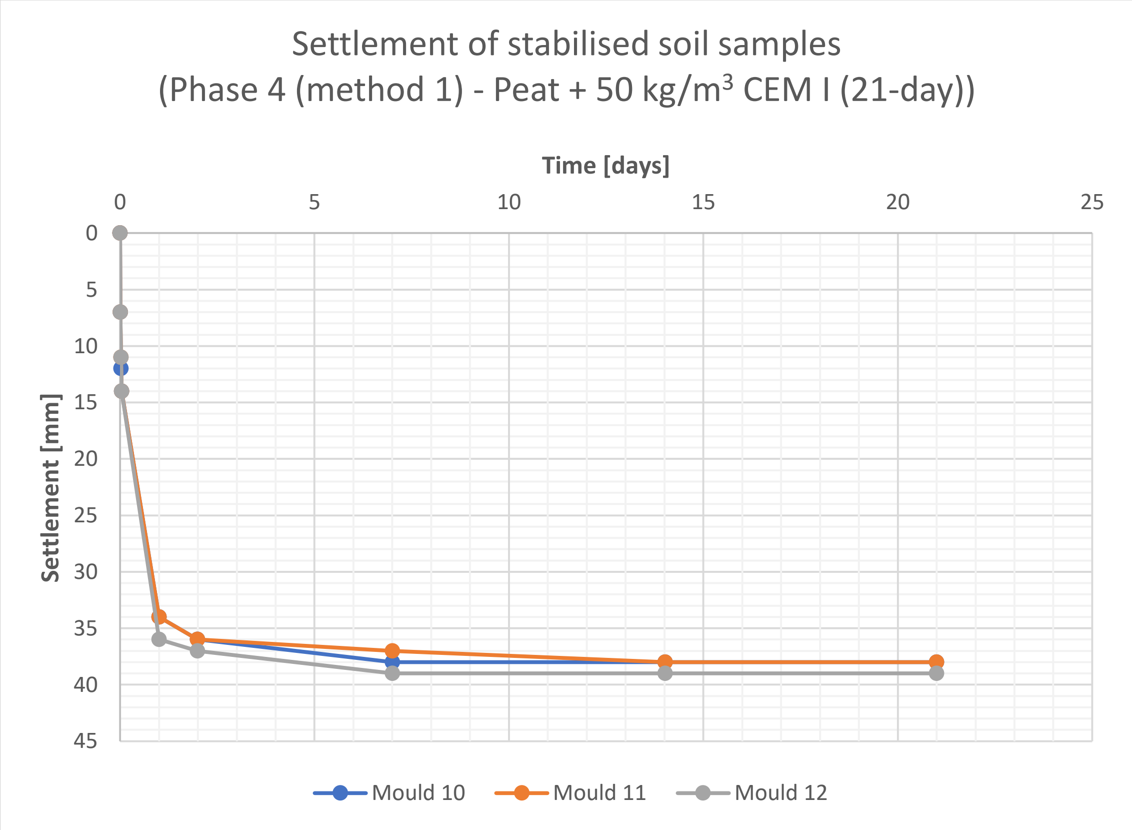


Figure F.71; The measured settlement of the peat samples stabilised with 50 kg/m³ of Portland cement (CEM I) in phase 4 (method 1) of the laboratory research. These samples were left to cure for 21 days.

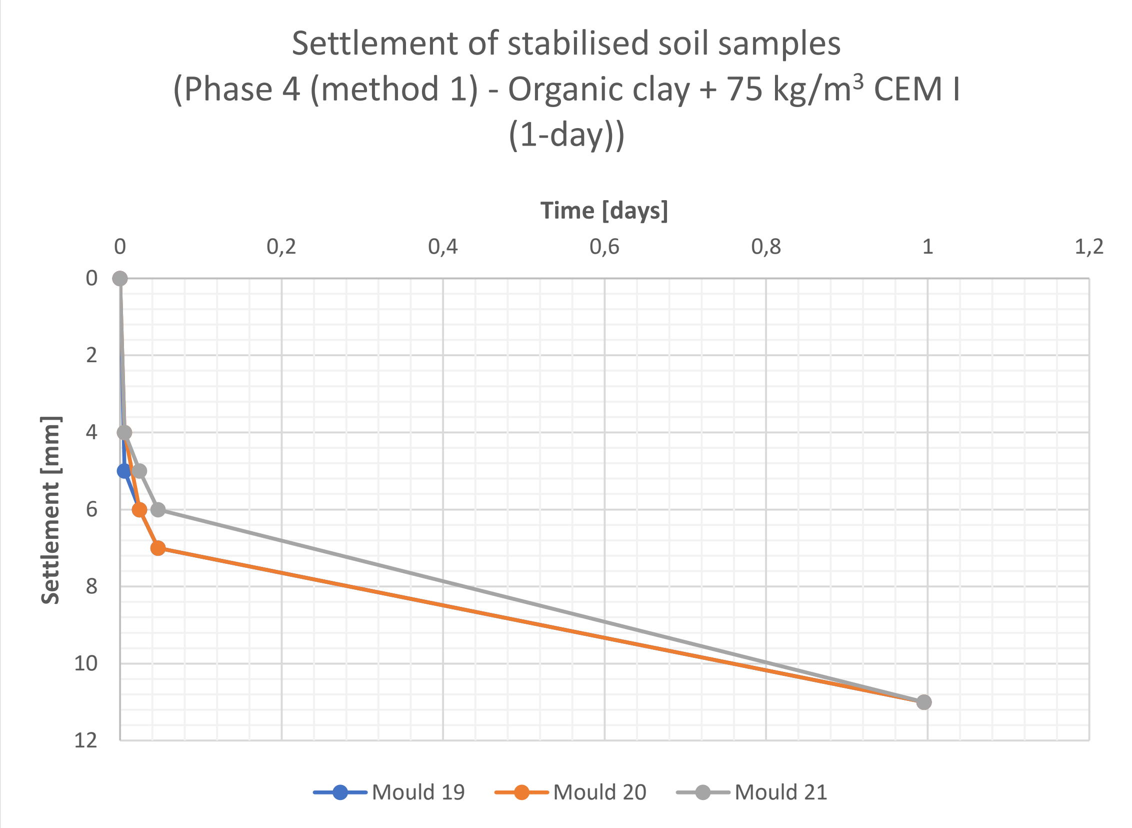


Figure F.72; The measured settlement of the organic clay samples stabilised with 75 kg/m³ of Portland cement (CEM I) in phase 4 (method 1) of the laboratory research. These samples were left to cure for 24 hours.

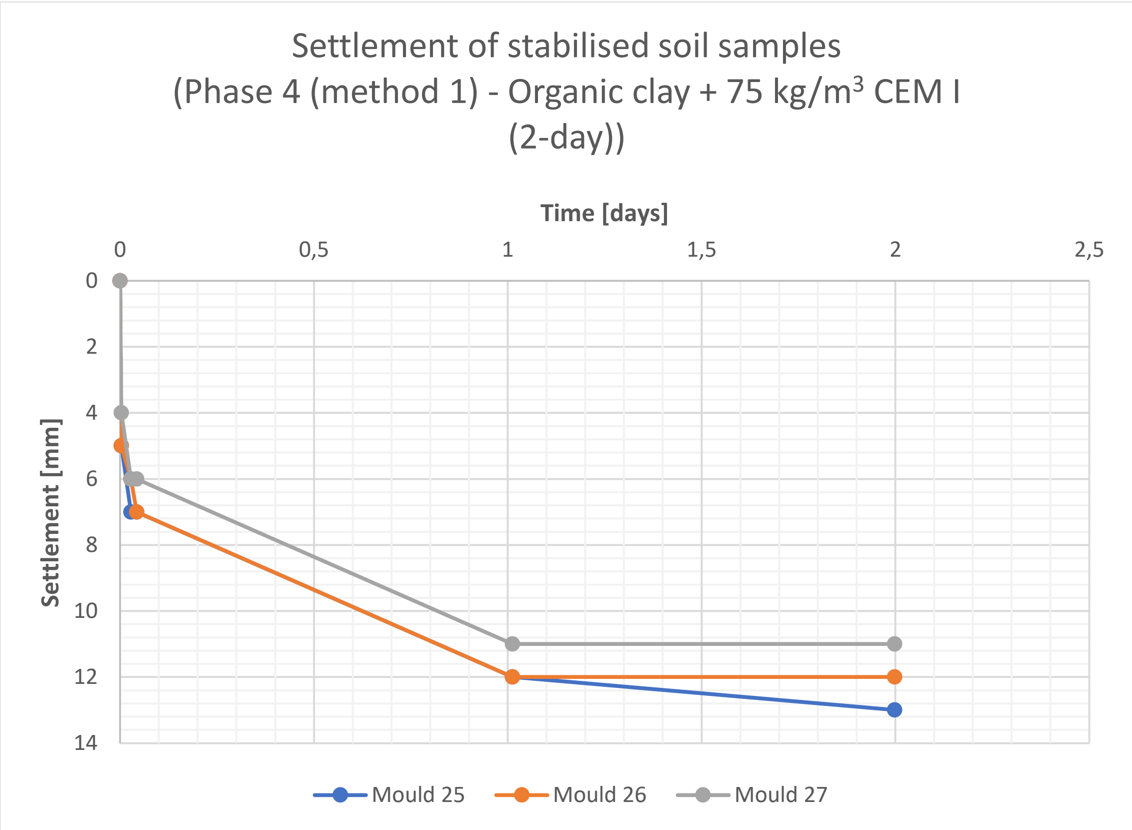


Figure F.73; The measured settlement of the organic clay samples stabilised with 75 kg/m³ of Portland cement (CEM I) in phase 4 (method 1) of the laboratory research. These samples were left to cure for 48 hours.

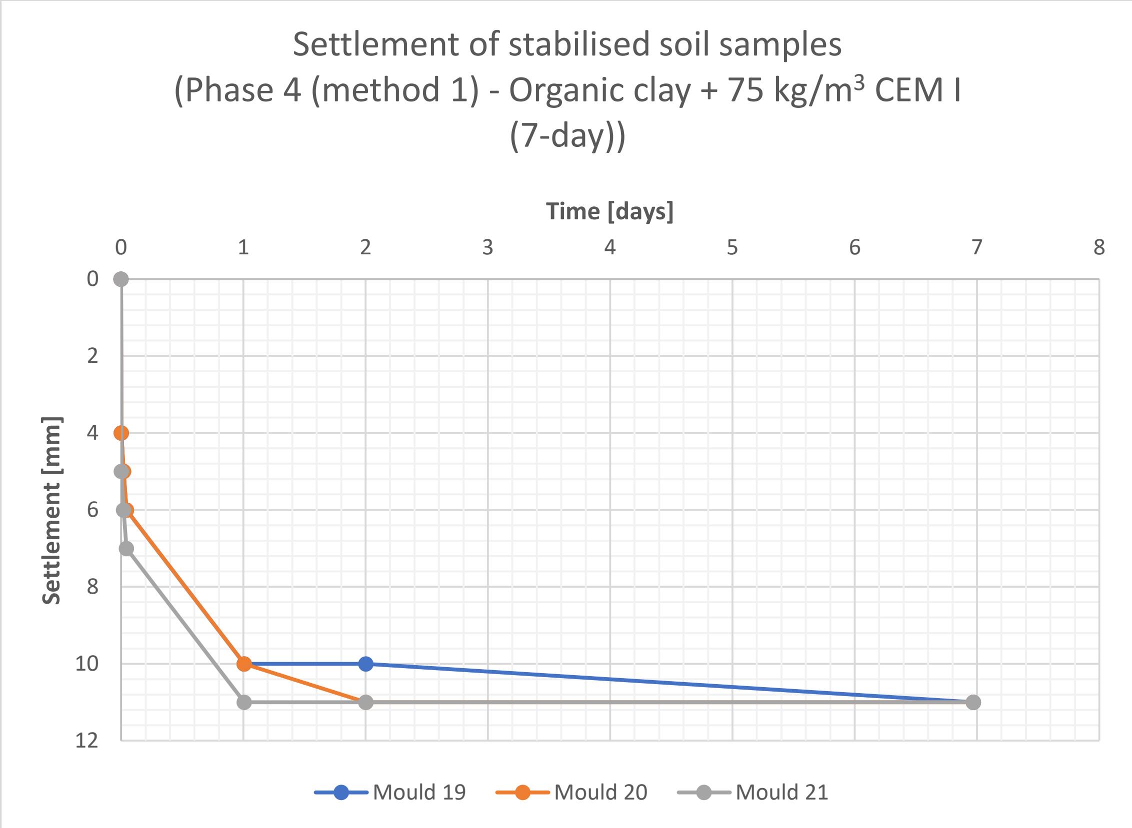


Figure F.74; The measured settlement of the organic clay samples stabilised with 75 kg/m³ of Portland cement (CEM I) in phase 4 (method 1) of the laboratory research. These samples were left to cure for 7 days.

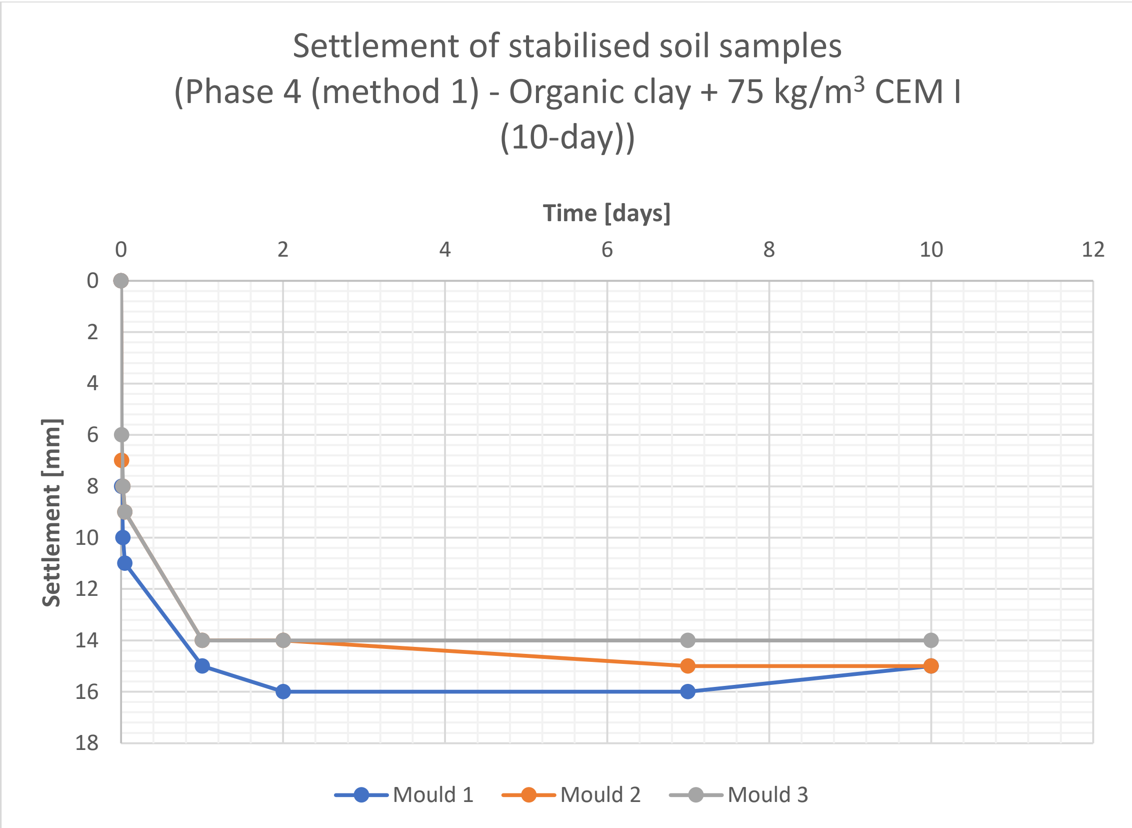


Figure F.75; The measured settlement of the organic clay samples stabilised with 75 kg/m³ of Portland cement (CEM I) in phase 4 (method 1) of the laboratory research. These samples were left to cure for 10 days.

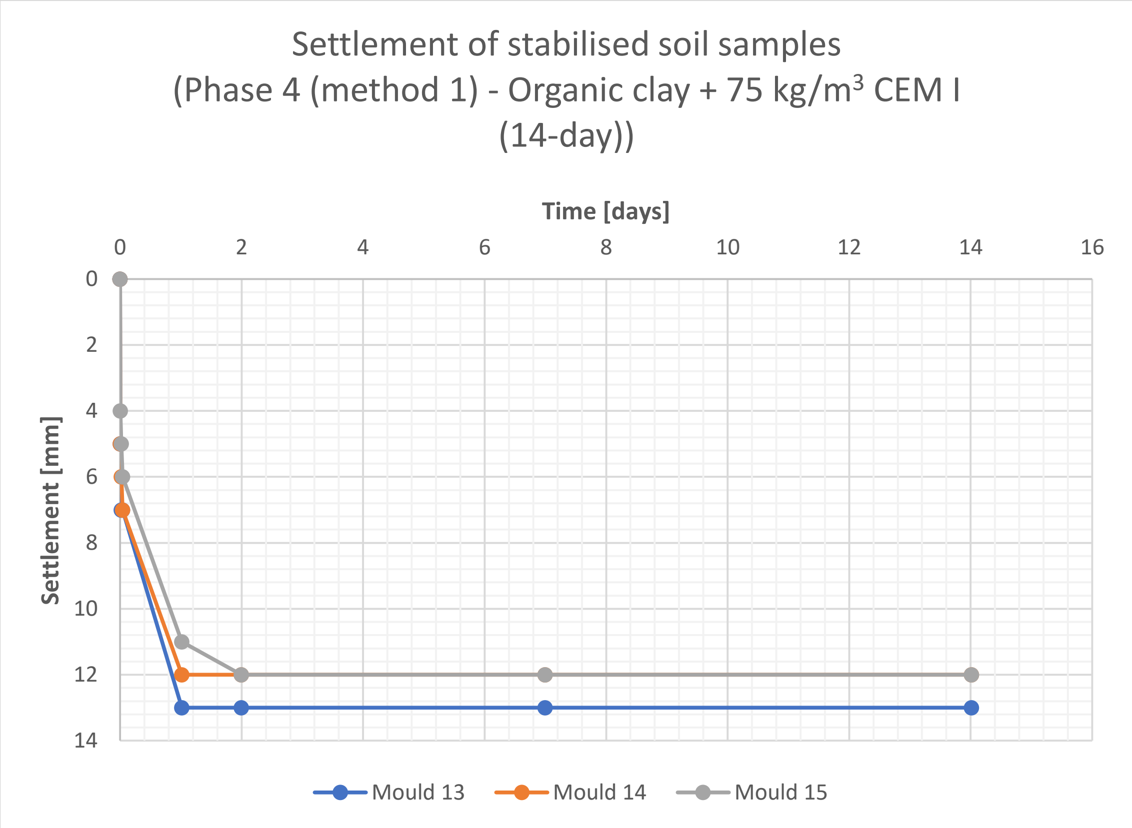


Figure F.76; The measured settlement of the organic clay samples stabilised with 75 kg/m³ of Portland cement (CEM I) in phase 4 (method 1) of the laboratory research. These samples were left to cure for 14 days.

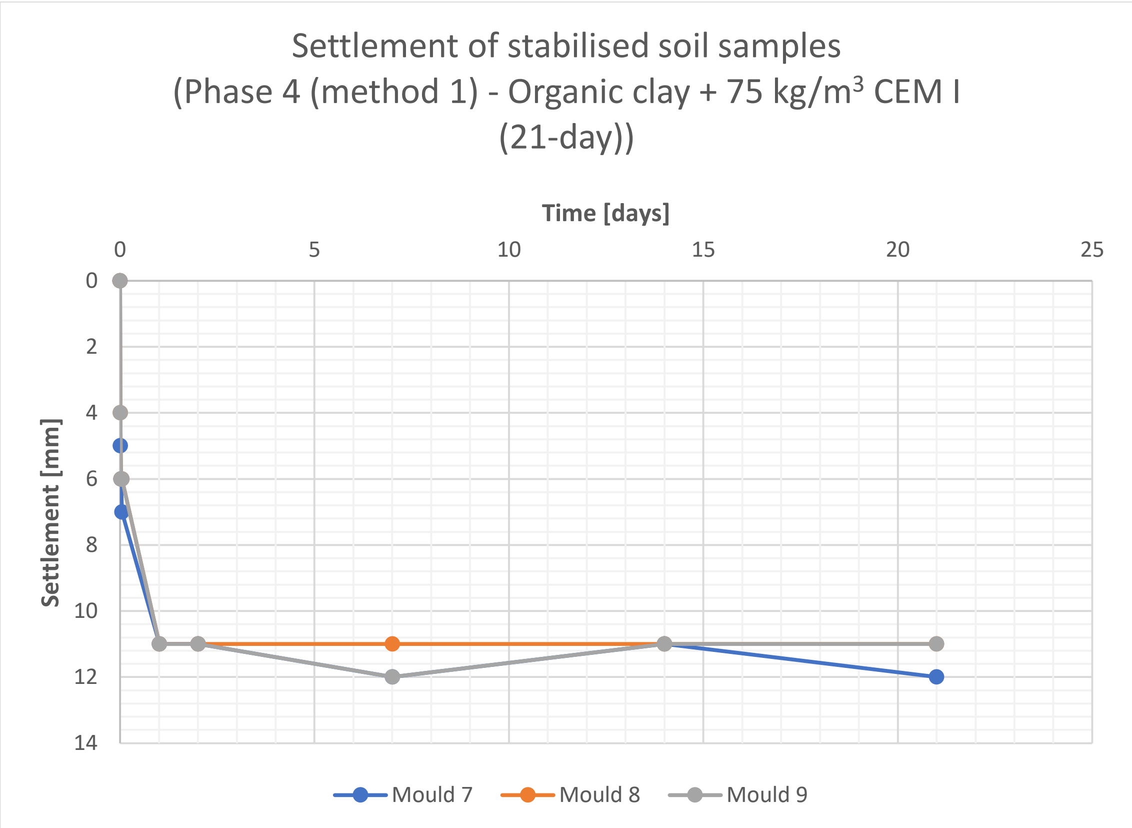


Figure F.77; The measured settlement of the organic clay samples stabilised with 75 kg/m³ of Portland cement (CEM I) in phase 4 (method 1) of the laboratory research. These samples were left to cure for 21 days.

F.4.1.2 Method 2 (single batch)

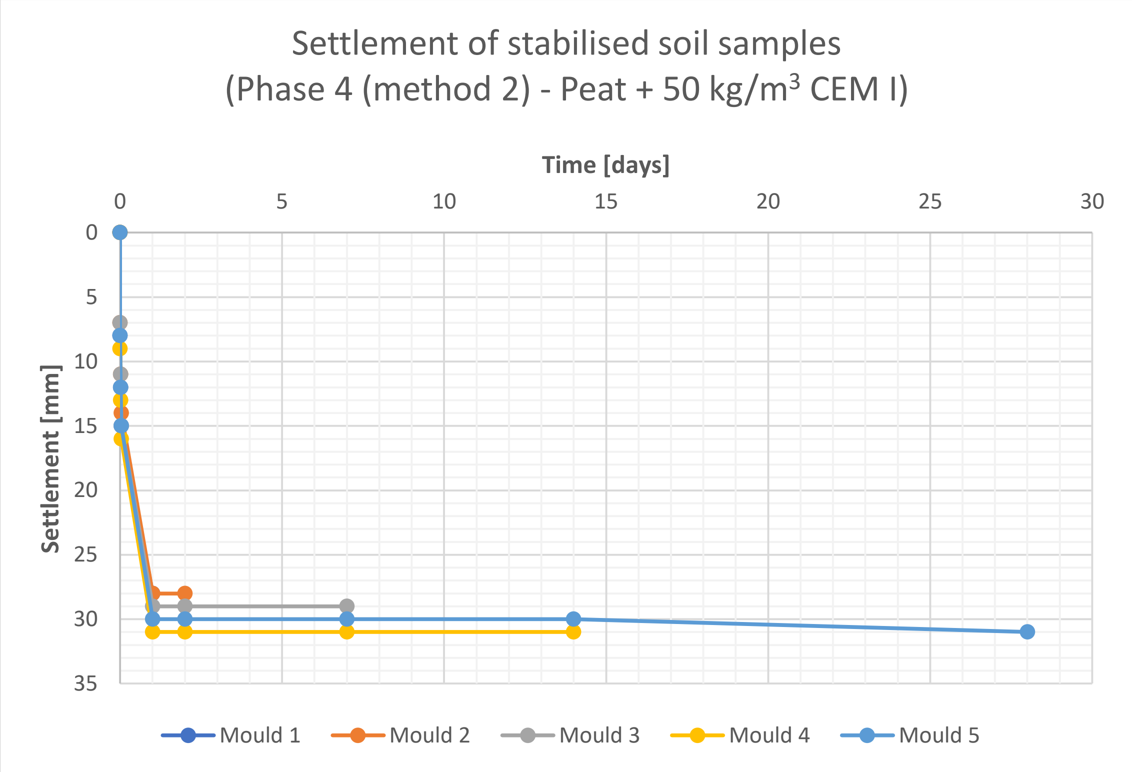


Figure F.78; The measured settlement of the peat samples stabilised with 50 kg/m³ of Portland cement (CEM I) in phase 4 (method 2) of the laboratory research. These samples were left to cure for 24 hours, 48 hours, 7 days, 14 days and 28 days respectively.

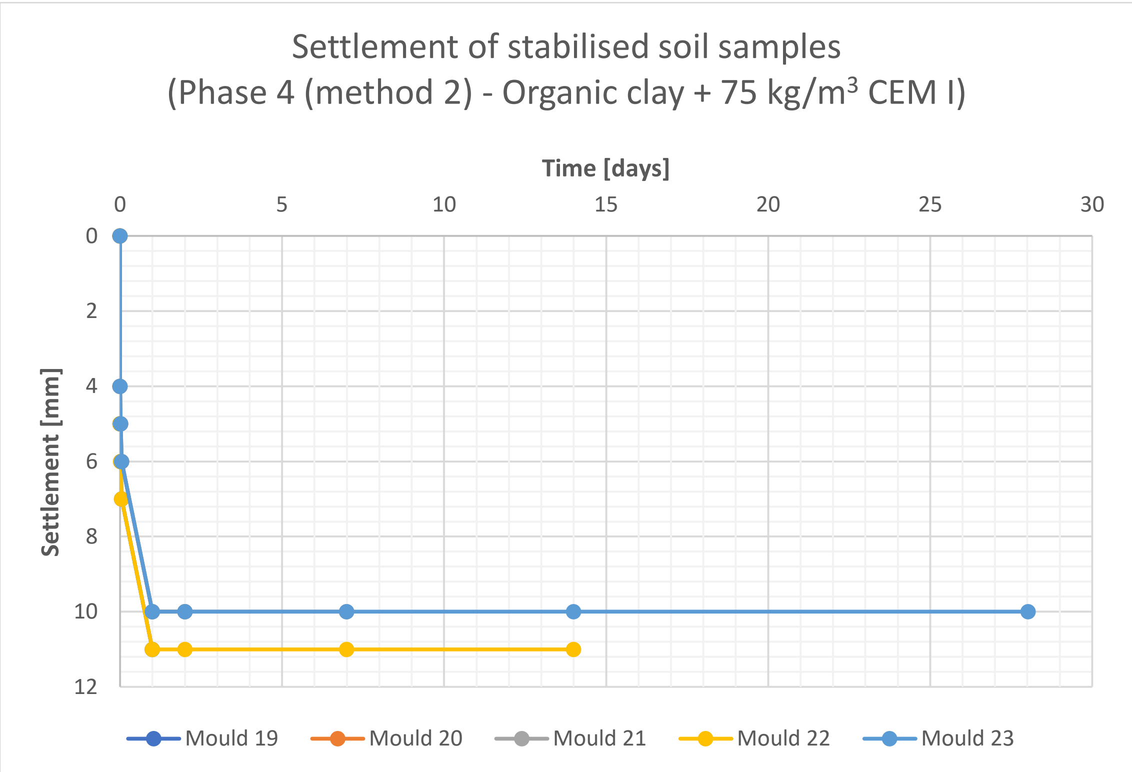


Figure F.79; The measured settlement of the organic clay samples stabilised with 75 kg/m³ of Portland cement (CEM I) in phase 4 (method 2) of the laboratory research. These samples were left to cure for 24 hours, 48 hours, 7 days, 14 days and 28 days respectively. Mould 19 and mould 22 show similar results. Mould 20, 21 and 23 also show similar results.

F.4.2 Unit weight of stabilised soil samples (laboratory stabilisation procedure)

F.4.2.1 Method 1 (multiple batches)

Table F.34; The measured unit weights of the different mixtures in phase 4 (method 1) of the laboratory research.

Mixture	Mould number	Bulk unit weight directly after filling the mould [kN/m³]	Bulk unit weight after 28 days of loading [kN/m³]	Bulk unit weight of extruded sample [kN/m³]
PEAT + 50 kg/m³ CEM I (1-day)	22	11,1	11,5	11,3
	23	11,1	11,4	11,3
	24	11,1	11,4	11,3
PEAT + 50 kg/m³ CEM I (2-day)	28	11,0	11,4	11,3
	29	11,0	11,5	11,3
	30	11,1	11,5	11,3
PEAT + 50 kg/m³ CEM I (7-day)	22	11,0	11,5	11,4
	23	11,1	11,5	11,4
	24	11,1	11,5	11,4
PEAT + 50 kg/m³ CEM I (10-day)	04	11,2	11,6	11,5
	05	11,2	11,6	11,5
	06	11,1	11,5	11,5
PEAT + 50 kg/m³ CEM I (14-day)	16	11,0	11,4	11,4
	17	11,2	11,5	11,4
	18	11,2	11,6	11,5
PEAT + 50 kg/m³ CEM I (21-day)	10	11,2	11,7	11,5
	11	11,2	11,7	11,5
	12	11,2	11,6	11,5
ORGANIC CLAY + 75 kg/m³ CEM I (1-day)	19	13,7	14,1	14,0
	20	13,8	14,1	14,0
	21	13,7	14,0	13,9
ORGANIC CLAY + 75 kg/m³ CEM I (2-day)	25	13,7	14,2	14,1
	26	13,7	14,1	14,0
	27	13,9	14,2	14,1
ORGANIC CLAY + 75 kg/m³ CEM I (7-day)	19	13,7	14,1	14,0
	20	13,7	14,1	14,0
	21	13,7	14,0	13,9
ORGANIC CLAY + 75 kg/m³ CEM I (10-day)	01	13,3	13,8	13,7
	02	13,4	13,9	13,8
	03	13,4	13,8	13,8
ORGANIC CLAY + 75 kg/m³ CEM I (14-day)	13	13,6	14,1	13,9
	14	13,6	14,0	13,9
	15	13,8	14,2	14,1
ORGANIC CLAY + 75 kg/m³ CEM I (21-day)	07	13,5	13,9	13,8
	08	13,7	14,1	13,9
	09	13,7	14,1	14,0

F.4.2.2 Method 2 (single batch)

Table F.35; The measured unit weights of the different mixtures in phase 4 (method 2) of the laboratory research.

Mixture	Mould number	Curing time	Bulk unit weight directly after filling the mould	Bulk unit weight after loading and curing	Bulk unit weight of extruded sample
	[-]	[days]	[kN/m³]	[kN/m³]	[kN/m³]
PEAT + 50 kg/m³ CEM I	01	1	10,9	11,2	11,1
	02	2	11,0	11,3	11,2
	03	7	10,9	11,2	11,1
	04	14	11,0	11,4	11,2
	05	28	11,0	11,5	11,3
ORGANIC CLAY + 75 kg/m³ CEM I	19	1	13,8	14,2	14,1
	20	2	13,9	14,2	14,1
	21	7	13,7	14,1	14,0
	22	14	13,8	14,1	14,0
	23	28	13,8	14,2	14,1

F.4.3 Unconfined compression tests

F.4.3.1 Method 1 (multiple batches)

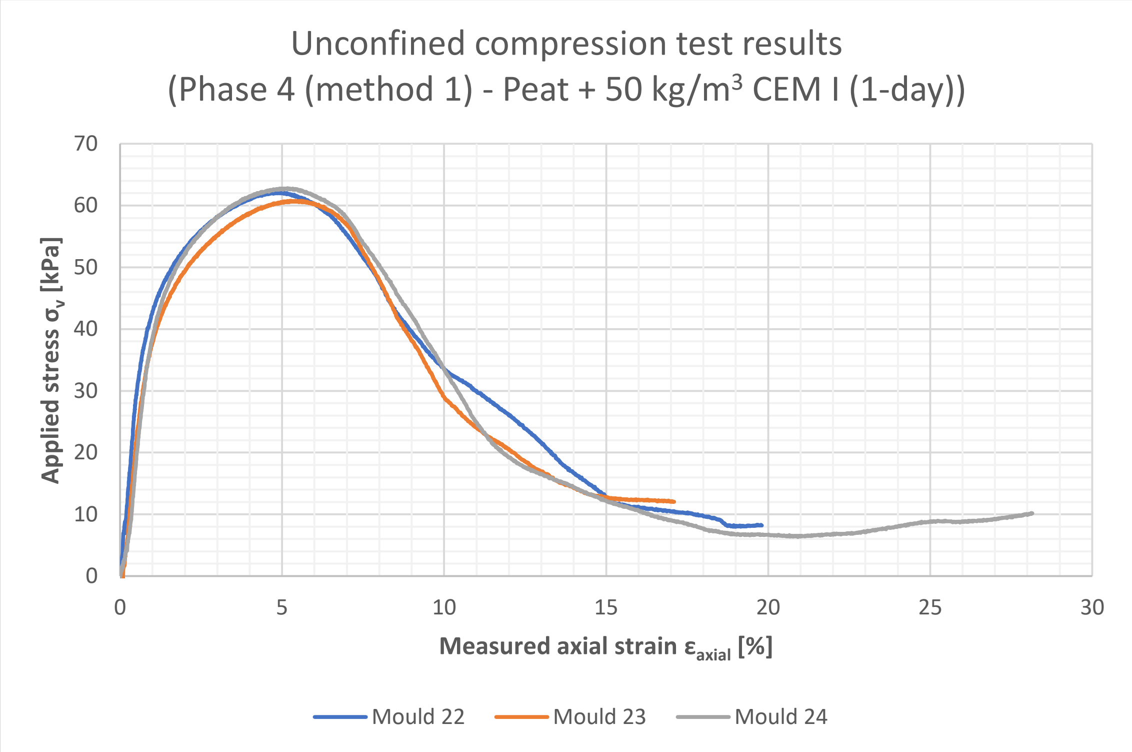


Figure F.80; The stress-strain diagrams obtained from the unconfined compression tests for the peat samples stabilised with 50 kg/m³ of Portland cement (CEM I) in phase 4 (method 1) of the laboratory research. These samples were left to cure for 24 hours.

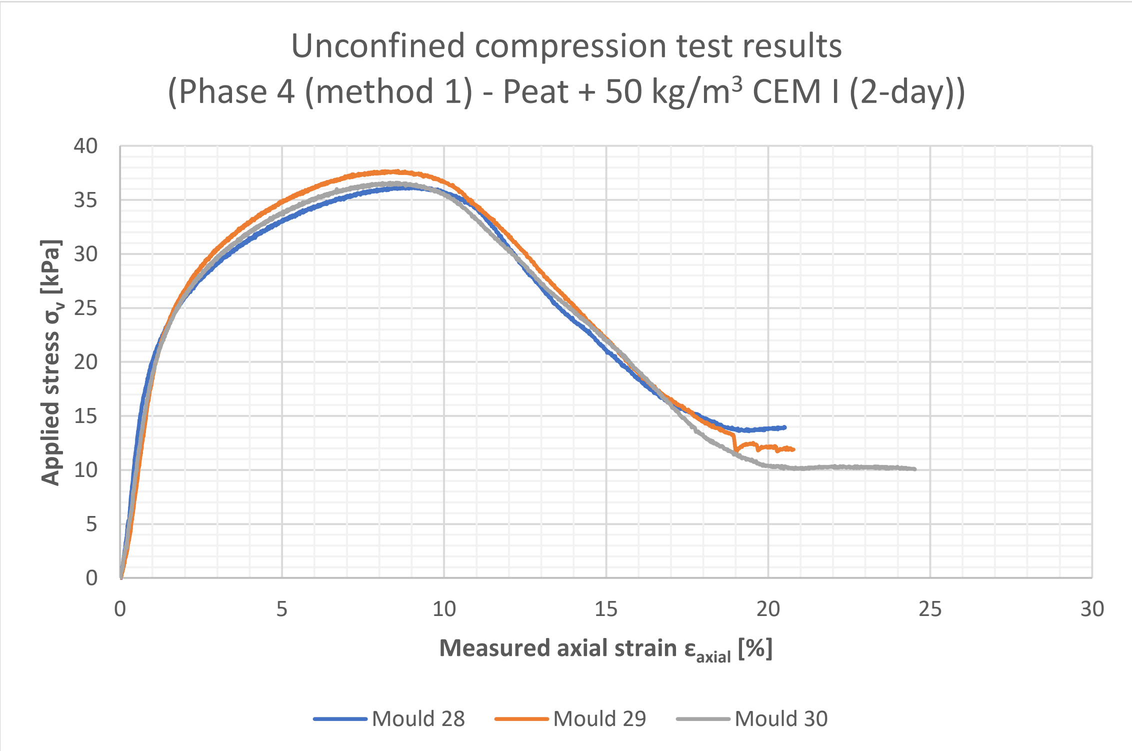


Figure F.81; The stress-strain diagrams obtained from the unconfined compression tests for the peat samples stabilised with 50 kg/m³ of Portland cement (CEM I) in phase 4 (method 1) of the laboratory research. These samples were left to cure for 48 hours.

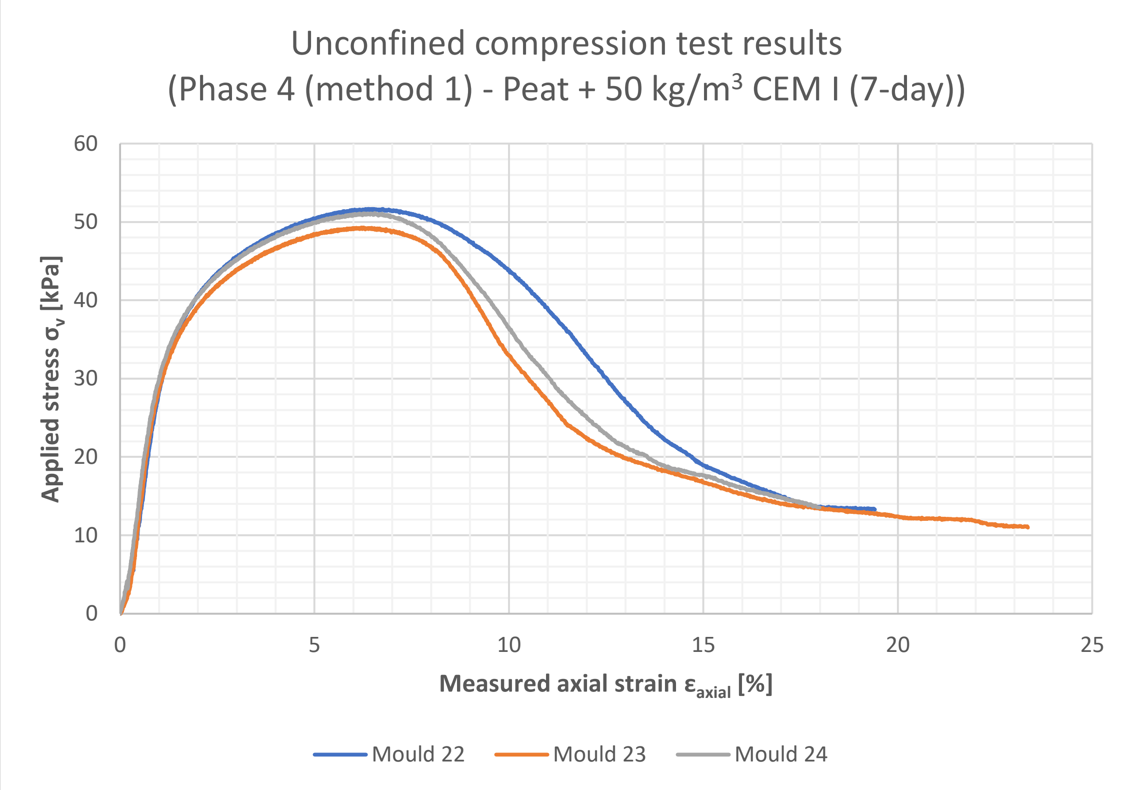


Figure F.82; The stress-strain diagrams obtained from the unconfined compression tests for the peat samples stabilised with 50 kg/m³ of Portland cement (CEM I) in phase 4 (method 1) of the laboratory research. These samples were left to cure for 7 days.

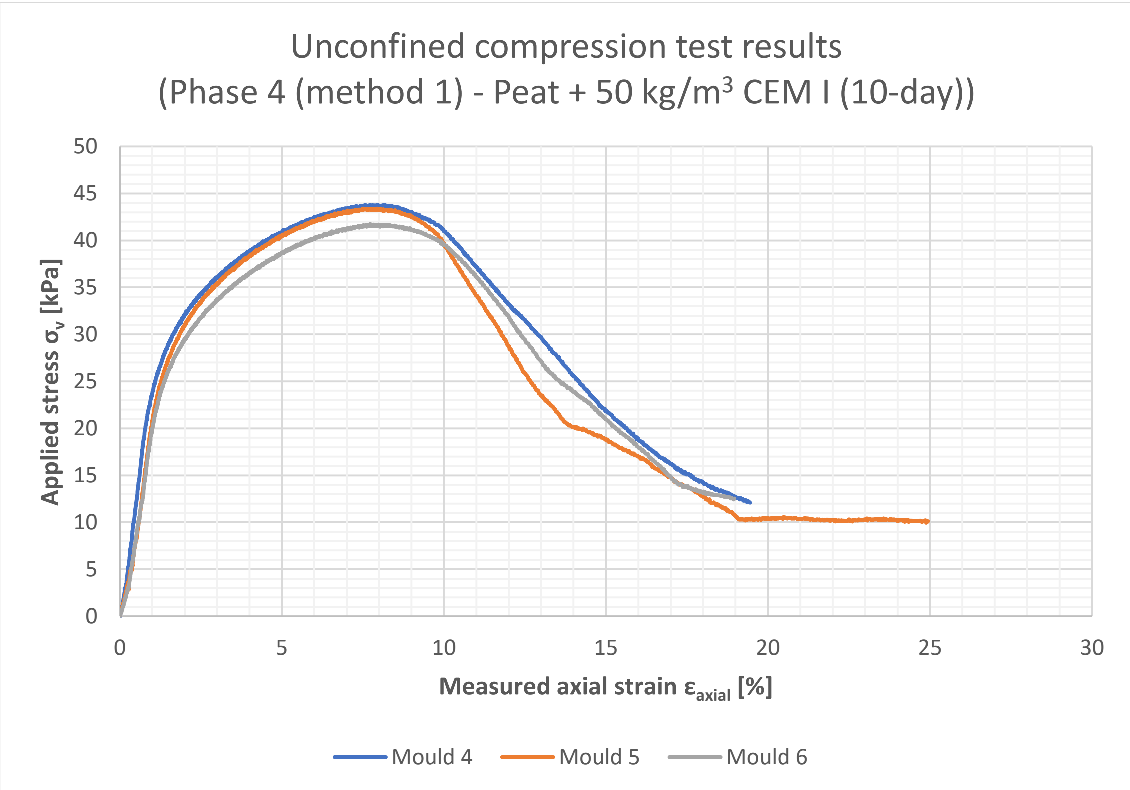


Figure F.83; The stress-strain diagrams obtained from the unconfined compression tests for the peat samples stabilised with 50 kg/m³ of Portland cement (CEM I) in phase 4 (method 1) of the laboratory research. These samples were left to cure for 10 days.

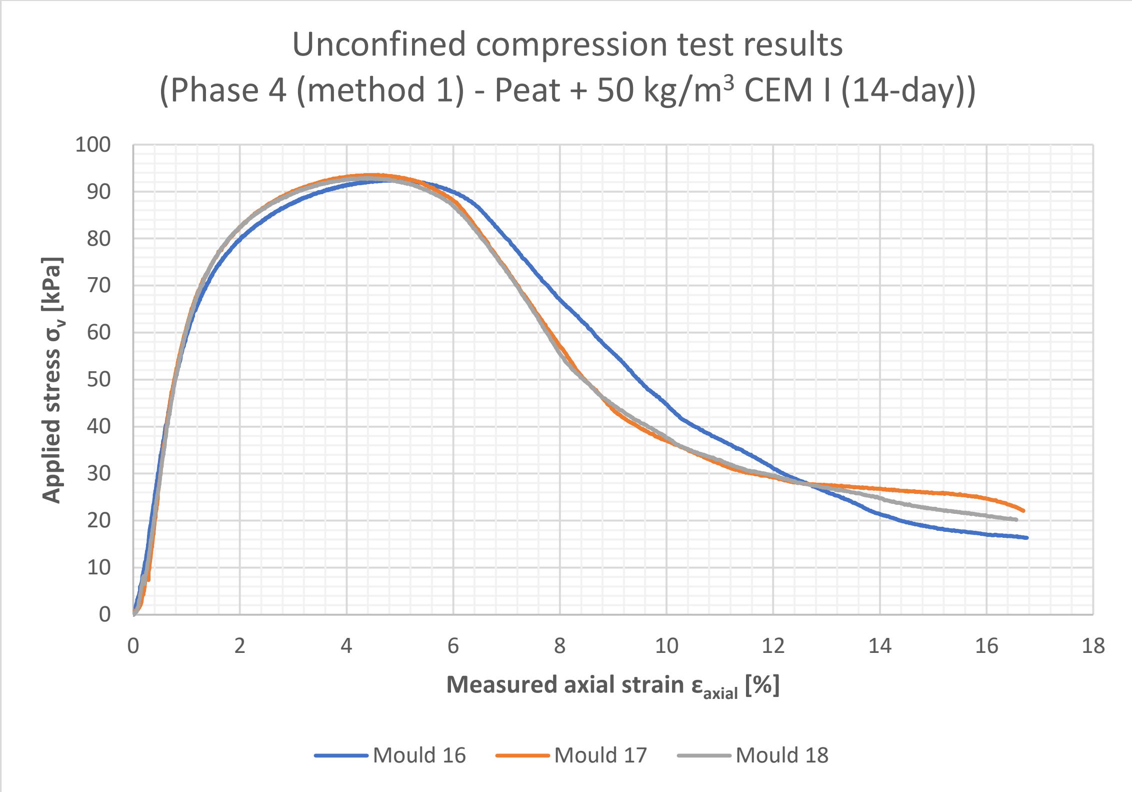


Figure F.84; The stress-strain diagrams obtained from the unconfined compression tests for the peat samples stabilised with 50 kg/m³ of Portland cement (CEM I) in phase 4 (method 1) of the laboratory research. These samples were left to cure for 14 days.

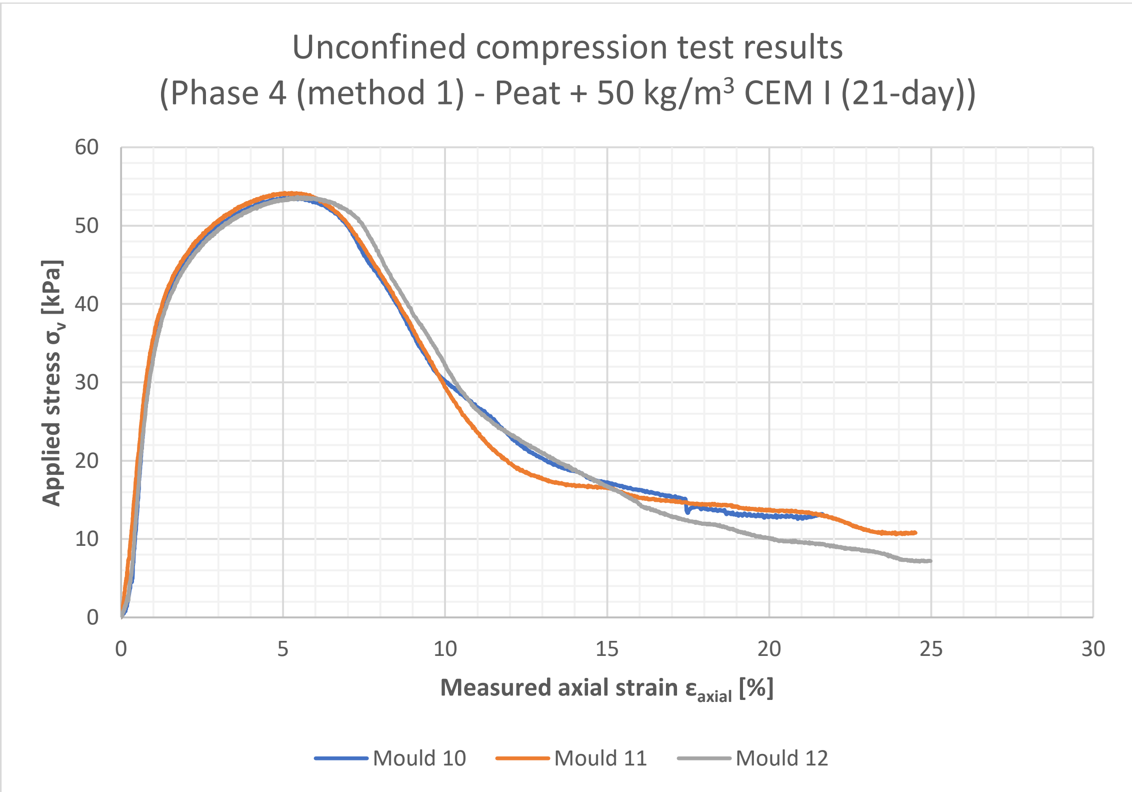


Figure F.85; The stress-strain diagrams obtained from the unconfined compression tests for the peat samples stabilised with 50 kg/m³ of Portland cement (CEM I) in phase 4 (method 1) of the laboratory research. These samples were left to cure for 21 days.

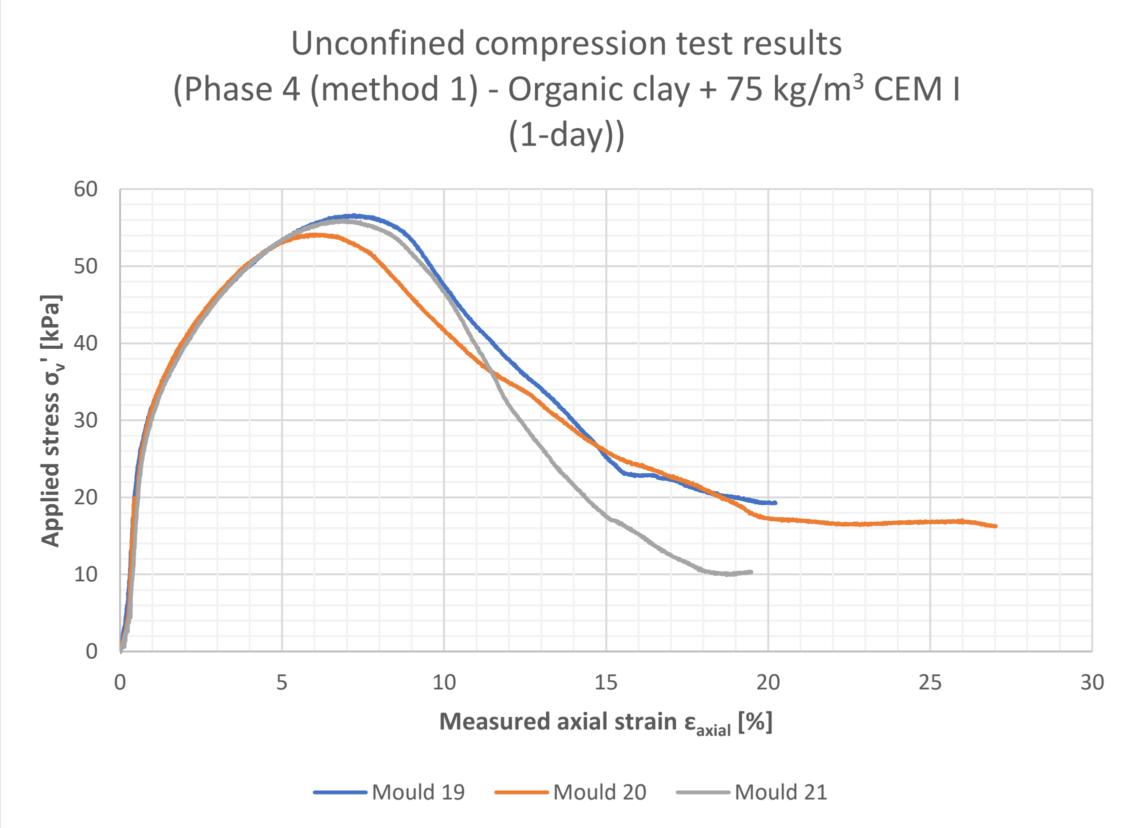


Figure F.86; The stress-strain diagrams obtained from the unconfined compression tests for the organic clay samples stabilised with 75 kg/m³ of Portland cement (CEM I) in phase 4 (method 1) of the laboratory research. These samples were left to cure for 24 hours.

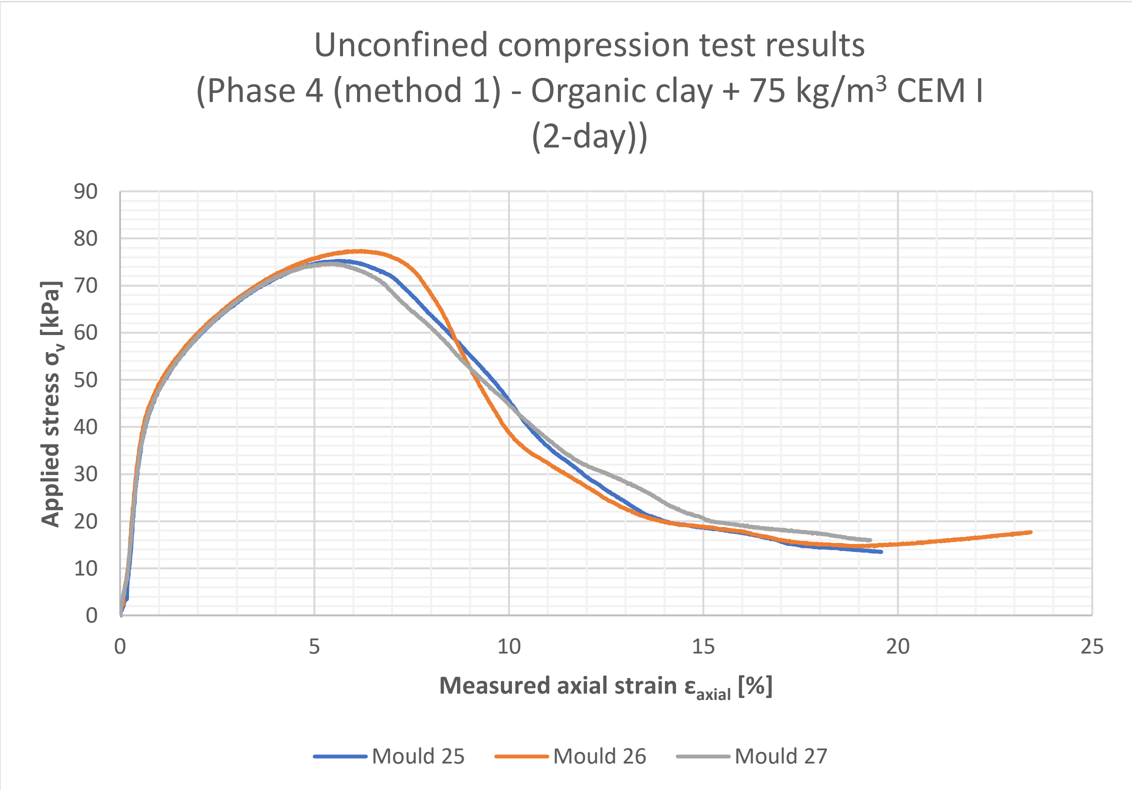


Figure F.87; The stress-strain diagrams obtained from the unconfined compression tests for the organic clay samples stabilised with 75 kg/m³ of Portland cement (CEM I) in phase 4 (method 1) of the laboratory research. These samples were left to cure for 48 hours.

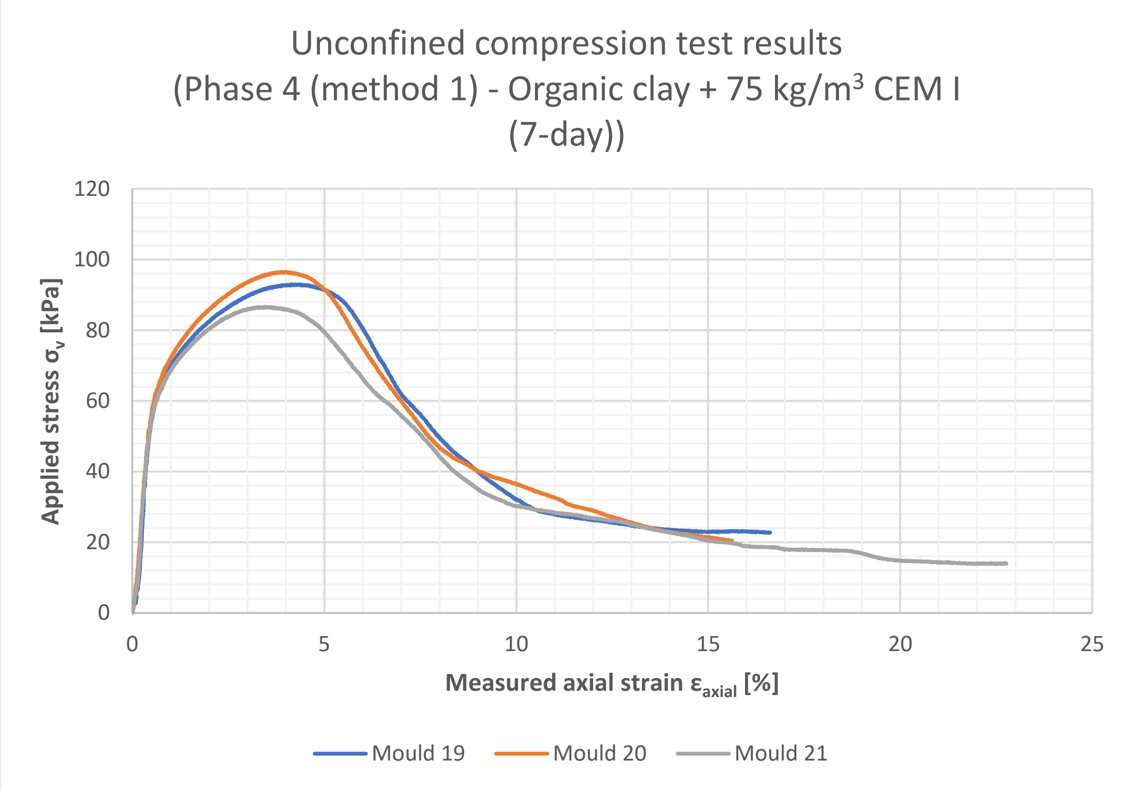


Figure F.88; The stress-strain diagrams obtained from the unconfined compression tests for the organic clay samples stabilised with 75 kg/m³ of Portland cement (CEM I) in phase 4 (method 1) of the laboratory research. These samples were left to cure for 7 days.

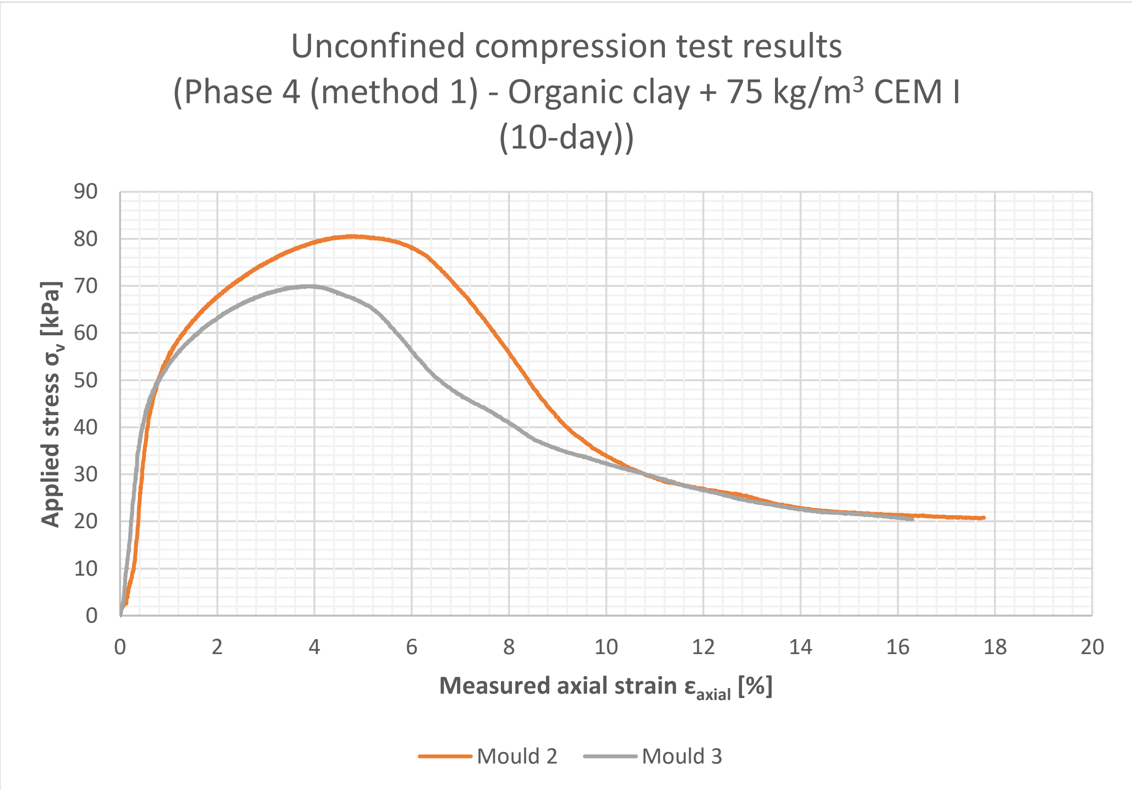


Figure F.89; The stress-strain diagrams obtained from the unconfined compression tests for the organic clay samples stabilised with 75 kg/m³ of Portland cement (CEM I) in phase 4 (method 1) of the laboratory research. These samples were left to cure for 10 days.

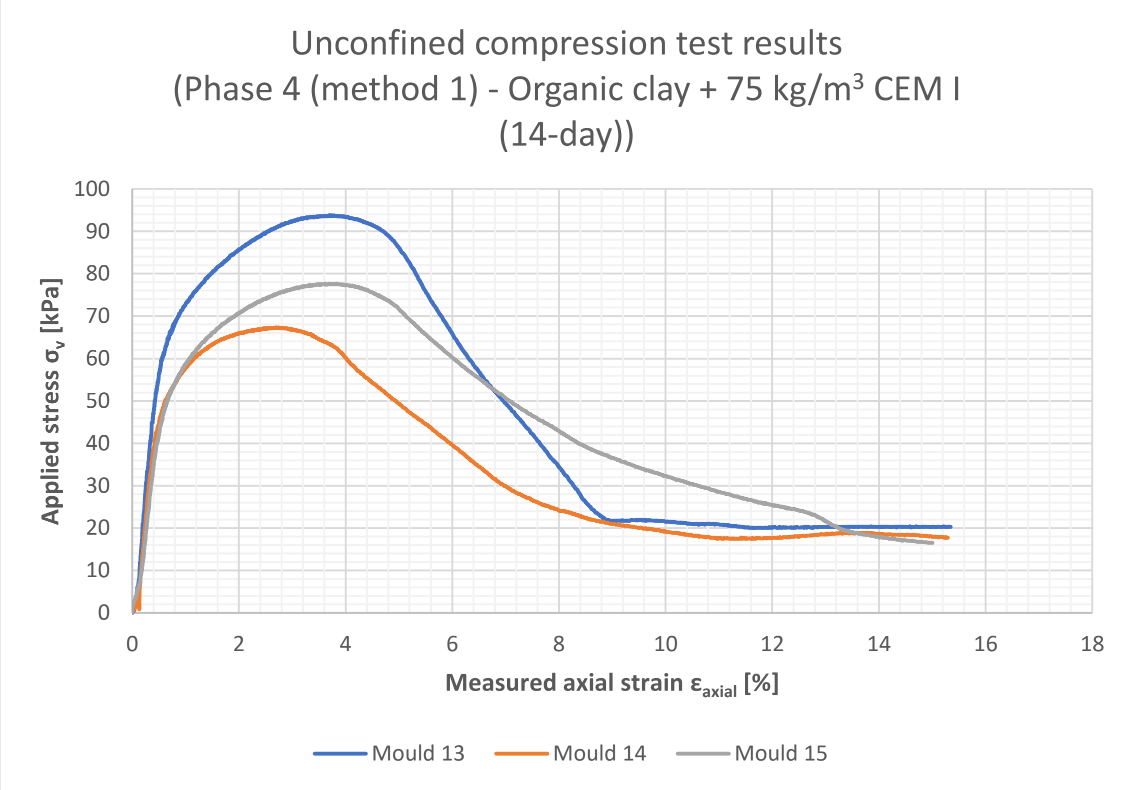


Figure F.90; The stress-strain diagrams obtained from the unconfined compression tests for the organic clay samples stabilised with 75 kg/m³ of Portland cement (CEM I) in phase 4 (method 1) of the laboratory research. These samples were left to cure for 14 days.

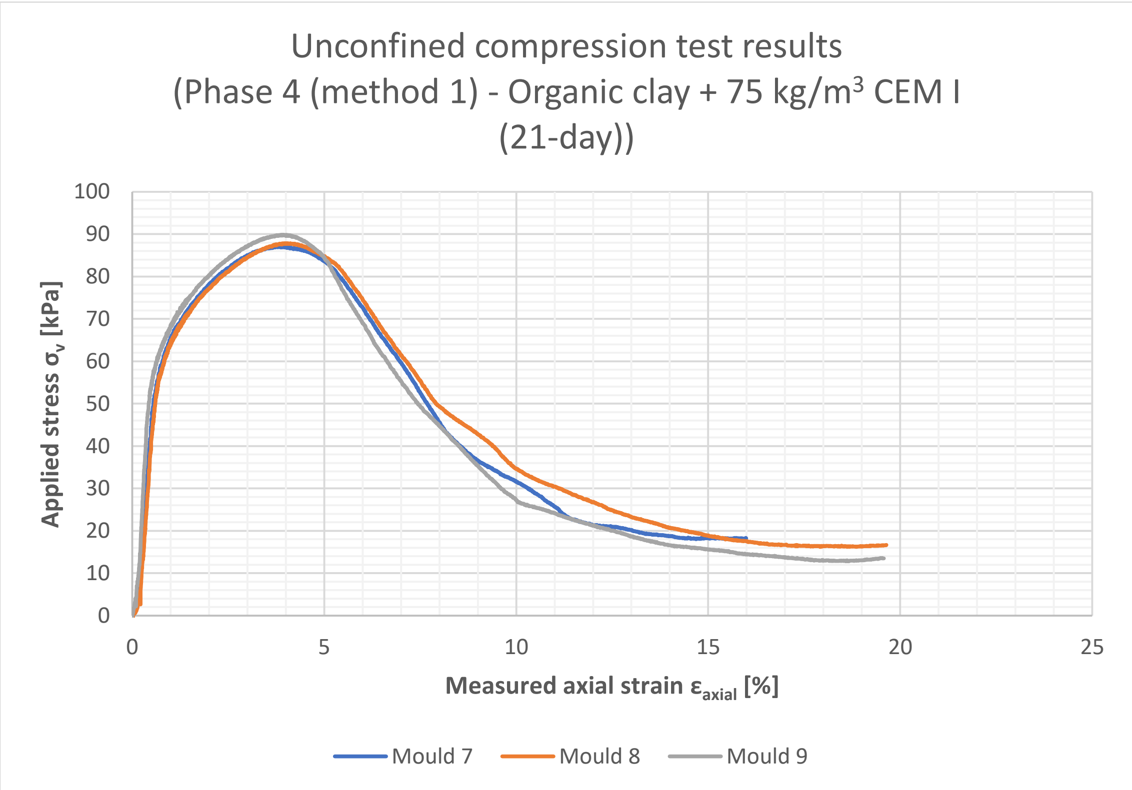


Figure F.91; The stress-strain diagrams obtained from the unconfined compression tests for the organic clay samples stabilised with 75 kg/m³ of Portland cement (CEM I) in phase 4 (method 1) of the laboratory research. These samples were left to cure for 21 days.

Table F.36; Properties of the stabilised peat and organic clay samples as measured during the unconfined compression tests of phase 4 (method 1) of the laboratory research. NM = not measured.

GENERAL INFORMATION MIXTURE					SAMPLE INFORMATION										
Soil type	**Binder**	**Binder dosage**	**Curing time**	**Mould number**	d	h	h/d	$\gamma_{stab;bulk}$	$\gamma_{stab;dry}$	w_{stab}	w_{nat} (estimated)	w/b	UCS	ε_f	Test rate
[-]	**[-]**	**[kg binder / m³ soil]**	**[days]**	**[-]**	**[mm]**	**[mm]**	**[-]**	**[kN/m³]**	**[kN/m³]**	**[%]**	**[%]**	**[-]**	**[kPa]**	**[%]**	**[N/s]**
Peat	CEM I	50	1	22	150	246	1,64	11,33	3,34	240	297	15,7	62	4,9	10
Peat	CEM I	50	1	23	150	245	1,63	11,29	3,31	241	300	15,7	61	5,3	
Peat	CEM I	50	1	24	150	245	1,63	11,34	3,40	234	290	15,7	63	5,2	
Peat	CEM I	50	2	28	150	236	1,57	11,32	3,35	238	299	15,9	36	8,9	10
Peat	CEM I	50	2	29	150	238	1,59	11,33	3,29	244	306	15,9	38	8,6	
Peat	CEM I	50	2	30	150	240	1,60	11,34	3,30	244	306	15,9	37	8,3	
Peat	CEM I	50	7	22	150	239	1,59	11,39	3,36	239	300	15,8	52	6,4	10
Peat	CEM I	50	7	23	150	237	1,58	11,36	3,45	229	287	15,8	49	6,2	
Peat	CEM I	50	7	24	150	235	1,57	11,40	3,35	241	301	15,8	51	6,5	
Peat	CEM I	50	10	04	150	240	1,60	11,48	3,56	223	273	15,4	44	8,0	10
Peat	CEM I	50	10	05	150	241	1,61	11,50	3,55	224	274	15,4	43	7,9	
Peat	CEM I	50	10	06	150	239	1,59	11,45	3,51	226	277	15,4	42	7,7	
Peat	CEM I	50	14	16	150	243	1,62	11,35	3,34	240	297	15,6	93	4,8	10
Peat	CEM I	50	14	17	150	242	1,61	11,42	3,34	242	300	15,6	94	4,6	
Peat	CEM I	50	14	18	150	248	1,65	11,47	3,33	244	303	15,6	93	4,4	
Peat	CEM I	50	21	10	150	239	1,59	11,54	3,59	222	271	15,3	54	5,4	10
Peat	CEM I	50	21	11	150	238	1,59	11,55	3,57	224	274	15,3	54	5,0	
Peat	CEM I	50	21	12	150	239	1,59	11,51	3,60	220	269	15,3	54	5,5	
Organic clay	CEM I	75	1	19	150	266	1,77	13,99	7,26	92,8	104	9,2	57	7,2	10
Organic clay	CEM I	75	1	20	150	265	1,77	14,00	7,30	91,7	103	9,2	54	6,0	
Organic clay	CEM I	75	1	21	150	265	1,77	13,91	7,20	93,3	105	9,2	56	6,9	

GENERAL INFORMATION MIXTURE					SAMPLE INFORMATION											
Soil type	**Binder**	**Binder dosage**	**Curing time**	**Mould number**	d	h	h/d	$\gamma_{stab;bulk}$	$\gamma_{stab;dry}$	w_{stab}	w_{nat} (estimated)	w/b	UCS	ε_f	Test rate
[-]	**[-]**	**[kg binder / m³ soil]**	**[days]**	**[-]**	**[mm]**	**[mm]**	**[-]**	**[kN/m³]**	**[kN/m³]**	**[%]**	**[%]**	**[-]**	**[kPa]**	**[%]**	**[N/s]**
Organic clay	CEM I	75	2	25	150	262	1,75	14,07	7,25	94,0	106	9,3	75	5,6	10
Organic clay	CEM I	75	2	26	150	265	1,77	14,02	7,25	93,5	105	9,3	77	6,2	
Organic clay	CEM I	75	2	27	150	266	1,77	14,11	7,26	94,5	106	9,3	75	5,5	
Organic clay	CEM I	75	7	19	150	264	1,76	14,01	7,15	96,0	108	9,4	93	4,2	10
Organic clay	CEM I	75	7	20	150	266	1,77	14,02	7,14	96,2	109	9,4	96	3,9	
Organic clay	CEM I	75	7	21	150	265	1,77	13,95	7,12	95,9	108	9,4	87	3,5	
Organic clay	CEM I	75	10	01	150	268	1,79	13,72	6,81	101	115	9,6	NM	NM	10
Organic clay	CEM I	75	10	02	150	270	1,80	13,82	6,84	102	115	9,6	81	4,8	
Organic clay	CEM I	75	10	03	150	264	1,76	13,76	6,87	100	113	9,6	70	3,9	
Organic clay	CEM I	75	14	13	150	268	1,79	13,95	7,12	96,0	109	9,7	94	3,7	10
Organic clay	CEM I	75	14	14	150	266	1,77	13,87	7,22	92,1	104	9,7	67	2,7	
Organic clay	CEM I	75	14	15	150	273	1,82	14,12	7,33	92,7	105	9,7	78	3,6	

GENERAL INFORMATION MIXTURE					SAMPLE INFORMATION										
Soil type	**Binder**	**Binder dosage**	**Curing time**	**Mould number**	d	h	h/d	$\gamma_{stab;bulk}$	$\gamma_{stab;dry}$	w_{stab}	w_{nat} (estimated)	w/b	UCS	ε_f	Test rate
[-]	**[-]**	**[kg binder / m³ soil]**	**[days]**	**[-]**	**[mm]**	**[mm]**	**[-]**	**[kN/m³]**	**[kN/m³]**	**[%]**	**[%]**	**[-]**	**[kPa]**	**[%]**	**[N/s]**
Organic clay	CEM I	75	21	07	150	261	1,74	13,82	7,03	96,7	109	9,4	87	3,8	10
Organic clay	CEM I	75	21	08	150	265	1,77	13,92	7,11	95,9	108	9,4	88	4,1	
Organic clay	CEM I	75	21	09	150	264	1,76	13,98	7,16	95,2	107	9,4	90	3,9	

F.4.3.2 Method 2 (single batch)

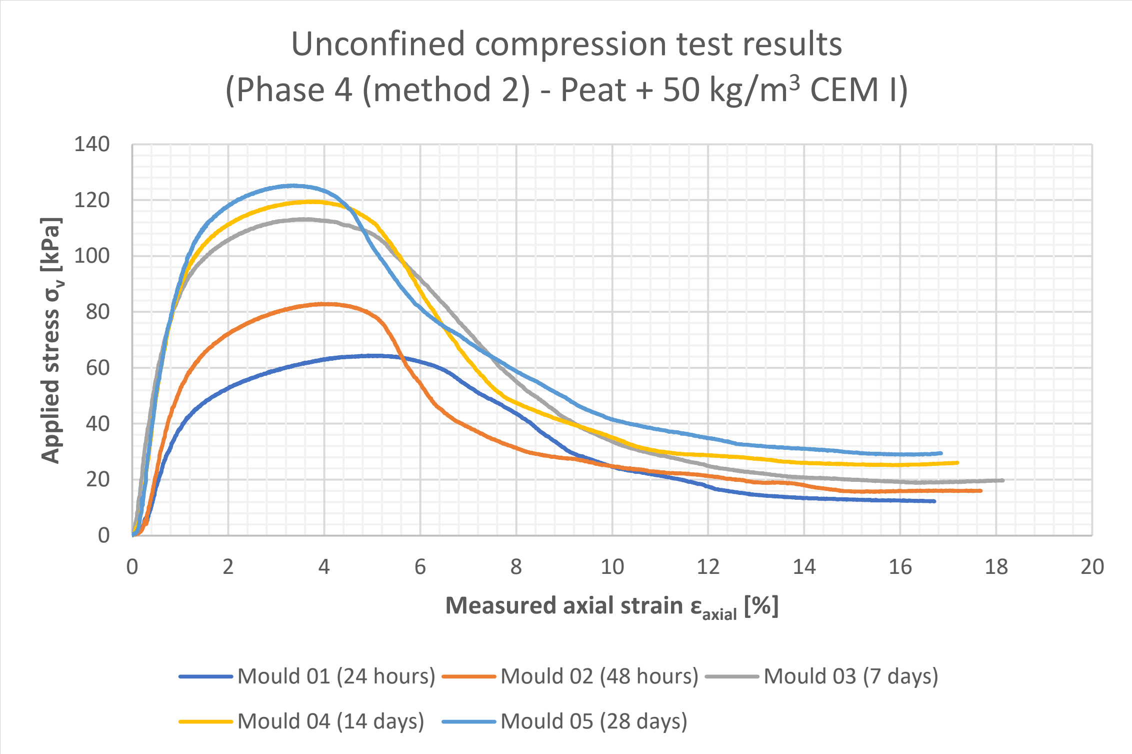


Figure F.92; The stress-strain diagrams obtained from the unconfined compression tests for the peat samples stabilised with 50 kg/m³ of Portland cement (CEM I) in phase 4 (method 2) of the laboratory research. These samples were left to cure for 24 hours, 48 hours, 7 days, 14 days and 28 days respectively.

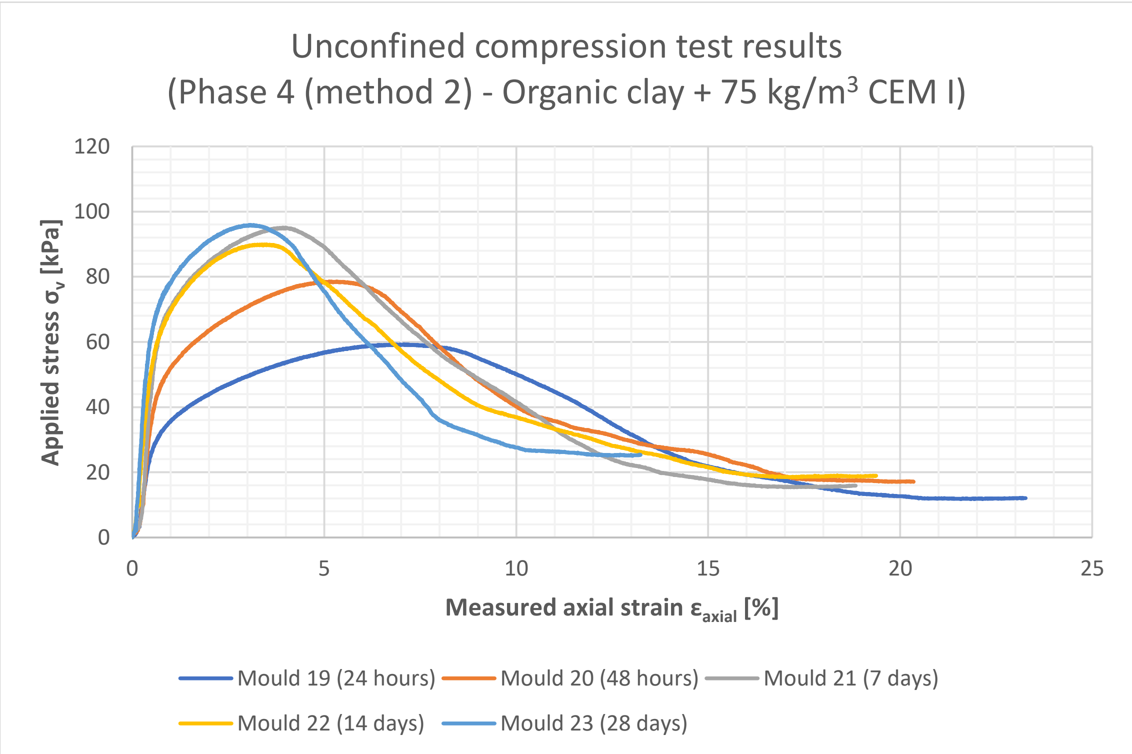


Figure F.93; The stress-strain diagrams obtained from the unconfined compression tests for the organic clay samples stabilised with 75 kg/m³ of Portland cement (CEM I) in phase 4 (method 2) of the laboratory research. These samples were left to cure for 24 hours, 48 hours, 7 days, 14 days and 28 days respectively.

Table F.37; Properties of the stabilised peat and organic clay samples as measured during the unconfined compression tests of phase 4 (method 2) of the laboratory research.

GENERAL INFORMATION MIXTURE					SAMPLE INFORMATION										
Soil type	**Binder**	**Binder dosage**	**Curing time**	**Mould number**	d	h	h/d	$\gamma_{stab;bulk}$	$\gamma_{stab;dry}$	w_{stab}	w_{nat} (estimated)	w/b	UCS	ε_f	Test rate
[-]	**[-]**	**[kg binder / m³ soil]**	**[days]**	**[-]**	**[mm]**	**[mm]**	**[-]**	**[kN/m³]**	**[kN/m³]**	**[%]**	**[%]**	**[-]**	**[kPa]**	**[%]**	**[N/s]**
Peat	CEM I	50	1	01	150	249	1,66	11,15	3,07	264	330	15,8	64	4,9	10
Peat	CEM I	50	2	02	150	252	1,68	11,20	3,10	262	328	15,8	83	4,1	
Peat	CEM I	50	7	03	150	247	1,65	11,14	3,08	262	328	15,8	113	3,5	
Peat	CEM I	50	14	04	150	248	1,65	11,24	3,12	261	326	15,8	119	3,6	
Peat	CEM I	50	28	05	150	249	1,66	11,29	3,12	262	328	15,8	125	3,4	
Organic clay	CEM I	75	1	19	150	266	1,77	14,09	7,38	91,0	102	9,2	59	7,1	10
Organic clay	CEM I	75	2	20	150	266	1,77	14,11	7,32	92,8	104	9,2	79	5,3	
Organic clay	CEM I	75	7	21	150	265	1,77	13,98	7,25	92,8	104	9,2	95	3,9	
Organic clay	CEM I	75	14	22	150	272	1,81	14,01	7,29	92,1	104	9,2	90	3,5	
Organic clay	CEM I	75	28	23	150	267	1,78	14,09	7,32	92,6	104	9,2	96	3,1	

F.5 Phase 5 – Soil parameters of stabilised soil samples

F.5.1 Settlement curves (laboratory stabilisation procedure)

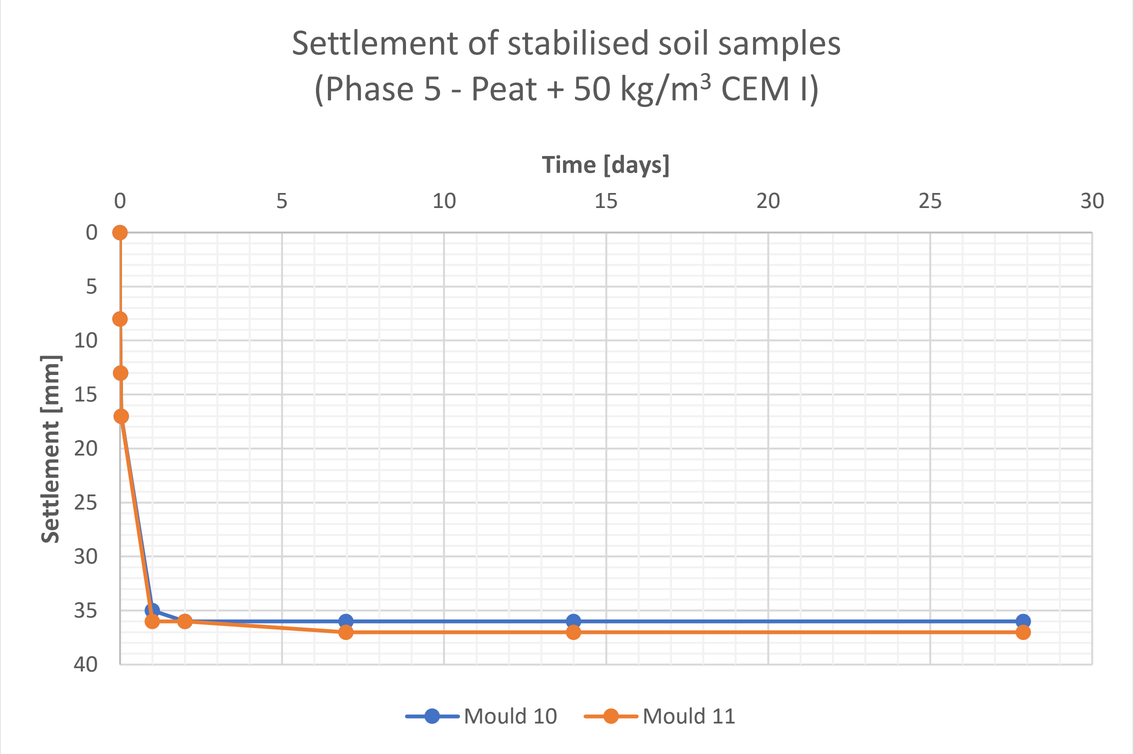


Figure F.94; The measured settlement of the peat samples stabilised with 50 kg/m³ of Portland cement (CEM I) in phase 5 of the laboratory research. These samples were left to cure for 28 days.

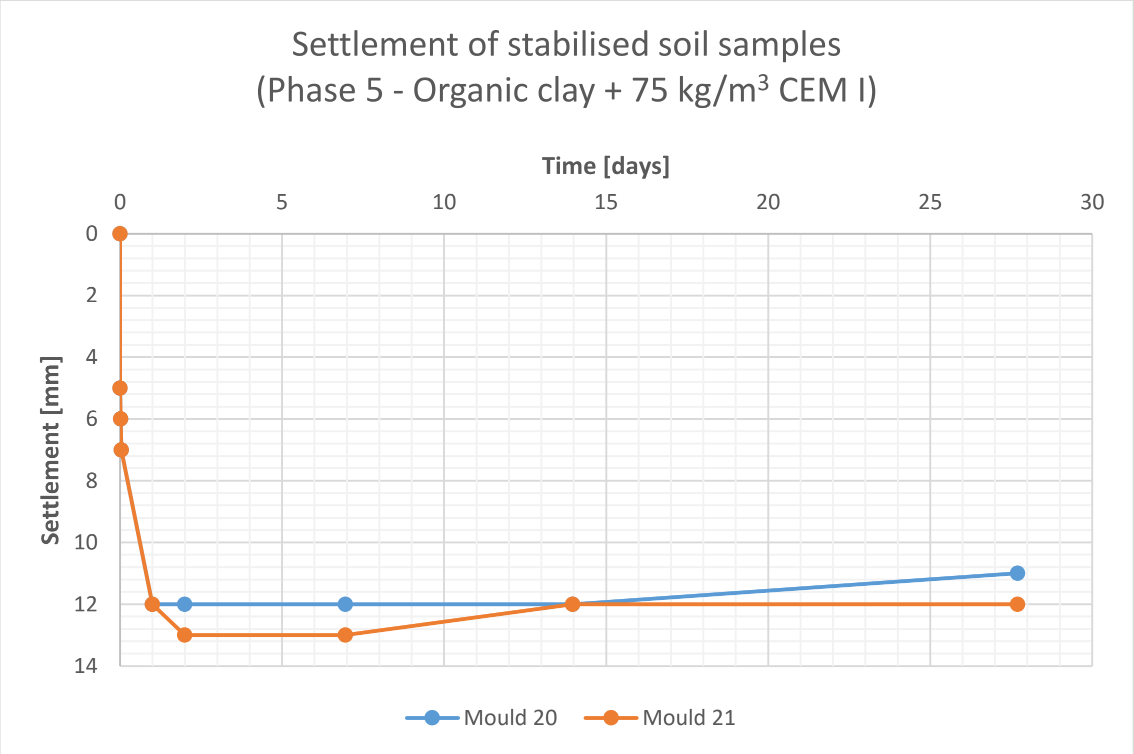


Figure F.95; The measured settlement of the organic clay samples stabilised with 75 kg/m³ of Portland cement (CEM I) in phase 5 of the laboratory research. These samples were left to cure for 28 days.

F.5.2 Unit weight of stabilised soil samples (laboratory stabilisation procedure)

Table F.38; The measured unit weights of the different mixtures in phase 5 of the laboratory research.

Mixture	Mould number	Bulk unit weight directly after filling the mould [kN/m³]	Bulk unit weight after 28 days of loading [kN/m³]	Bulk unit weight of extruded sample [kN/m³]
PEAT + 50 kg/m³ CEM I (28-day)	10	11,03	11,40	11,27
	11	11,01	11,42	11,28
ORGANIC CLAY + 75 kg/m³ CEM I (28-day)	20	13,83	14,19	14,09
	21	Not measured	Not measured	Not measured

F.5.3 Unconfined compression tests

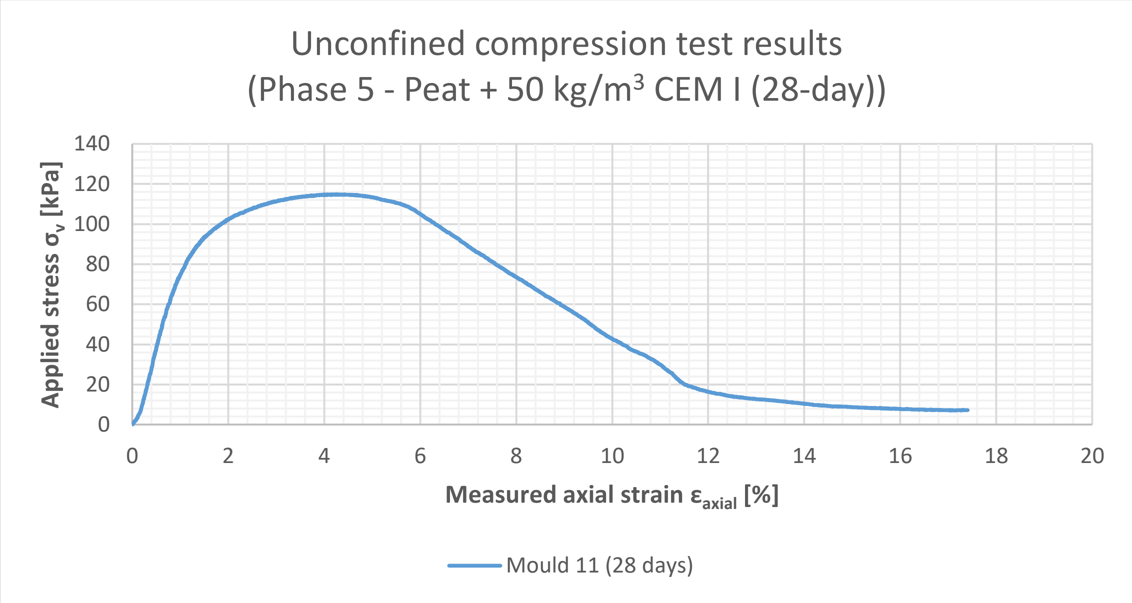


Figure F.96; The stress-strain diagrams obtained from the unconfined compression tests for the peat sample stabilised with 50 kg/m³ of Portland cement (CEM I) in phase 5 of the laboratory research. This sample was left to cure for 28 days.

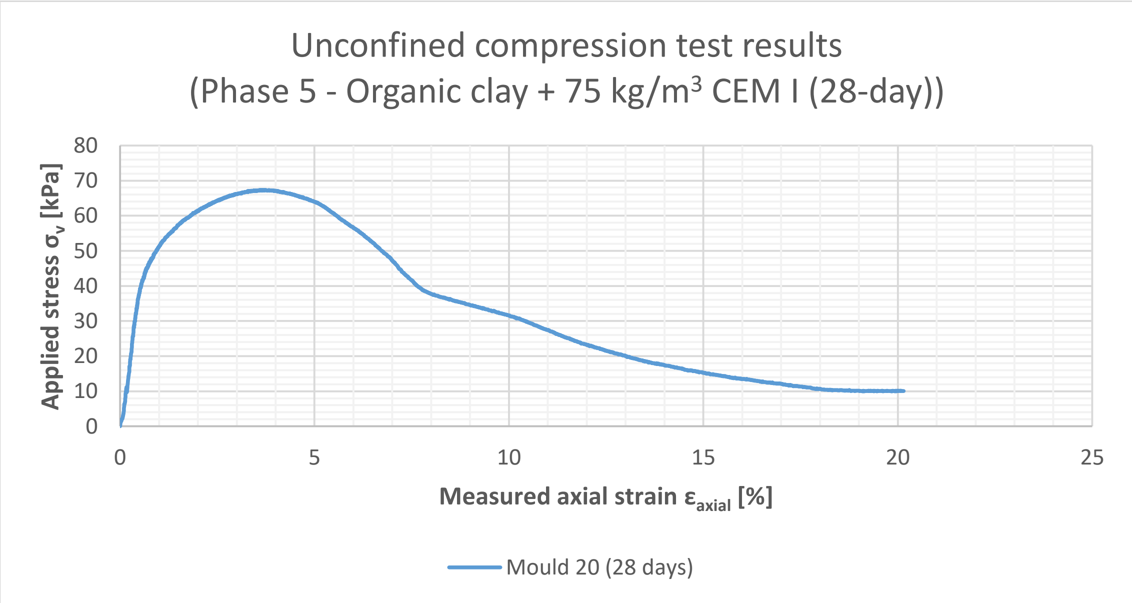


Figure F.97; The stress-strain diagrams obtained from the unconfined compression tests for the organic clay sample stabilised with 75 kg/m³ of Portland cement (CEM I) in phase 5 of the laboratory research. This sample was left to cure for 28 days.

Table F.39; Properties of the stabilised peat and organic clay samples as measured during the unconfined compression tests of phase 5 of the laboratory research.

GENERAL INFORMATION MIXTURE					SAMPLE INFORMATION										
Soil type	**Binder**	**Binder dosage**	**Curing time**	**Mould number**	d	h	h/d	$\gamma_{stab;bulk}$	$\gamma_{stab;dry}$	w_{stab}	w_{nat} (estimated)	w/b	UCS	ε_f	Test rate
[-]	**[-]**	**[kg binder / m³ soil]**	**[days]**	**[-]**	**[mm]**	**[mm]**	**[-]**	**[kN/m³]**	**[kN/m³]**	**[%]**	**[%]**	**[-]**	**[kPa]**	**[%]**	**[N/s]**
Peat	CEM I	50	28	11	150	238	1,59	11,28	3,18	255	319	15,8	115	4,3	10
Organic clay	CEM I	75	28	20	150	262	1,75	14,09	7,26	94,2	106	9,2	67	3,6	

F.5.4 CIU Triaxial tests

Table F.40; Tested soil-binder mixture and mixture composition.

Soil type	Binder type	Dosage	Curing time	Mixture composition			
				Soil solids	Soil water	Binder solids	Added water
		[kg binder / m³ soil]	[days]	[% m/m]	[% m/m]	[% m/m]	[% m/m]
Organic clay	Portland cement (CEM I)	75	28	44,4	33,7	5,5	16,5

Table F.41; Visual description of the (undisturbed) stabilised organic clay soil samples as carried out by the laboratory technician of Fugro NL Land B.V. prior to the isotropically consolidated undrained triaxial test.

Soil type	Visual description
Stabilised organic clay	Prepared (clay)

Table F.42; Properties of the (undisturbed) stabilised organic clay samples as measured during the CIU triaxial test.

Soil type	Load step	Taken from mould	$\gamma_{bulk;i}$	$\gamma_{dry;i}$	$w_{stab;i}$	$w_{nat;i}$ (assumed)	$w_{stab;f}$	$w_{nat;f}$ (assumed)
	[kPa]	[-]	[kN/m³]	[kN/m³]	[%]	[%]	[%]	[%]
Stabilised organic clay	1	21	14,2	7,3	96,2	108	98,4	106
	2	21	14,3	7,4	92,6	104	92,6	104
	3	21	14,4	7,4	94,1	106	84,5	95,0

Table F.43; Data from the isotropically consolidated undrained triaxial test on the (undisturbed) stabilised organic clay samples after the consolidation stage was finished.

Soil type	Load step	σ_{1C}	σ_{3C}	B	ε_{1C}	$\varepsilon_{vol;C}$	Strain rate
	[kPa]	[kPa]	[kPa]	[-]	[%]	[%]	[%/h]
Stabilised organic clay	1	15,0	15,0	0,97	2,20	0,89	2,0
	2	60,0	60,0	0,97	2,40	2,64	2,0
	3	120	120	0,97	5,60	7,15	1,9

Table F.44; Measured strain and stress parameters from the isotropically consolidated undrained triaxial test at failure of the (undisturbed) stabilised organic clay samples.

Soil type	Load step	Adopted failure criterion	q	p'	s'	t	Δu	σ'_1	σ'_3	ε_f
		[-]	[kPa]	[kPa]	[kPa]	[kPa]	[kPa]	[kPa]	[kPa]	[%]
Stab. organic clay	1	Max. q	64,85	38,66	49,52	32,58	-2,40	82,10	16,94	22,45
	2	Max. q	100,9	52,79	69,61	50,45	40,54	120,1	19,16	10,17
	3	Max. q	151,8	77,08	102,4	75,91	91,31	178,3	26,48	13,48

Table F.45; Measured strength parameters from the isotropically consolidated undrained triaxial test at failure of the (undisturbed) stabilised organic clay samples.

Soil type	Load step	Adopted failure criterion	c'	ϕ'	S_u	$E_{u;50}$
			[kPa]	[°]	[kPa]	[MPa]
Stabilised organic clay	1	Max. q	0,00	46,50	32,6	1,7
	2	Max. q			50,5	16,8
	3	Max. q			75,9	25,4

Table F.46; The measured values of the drained shear strength parameters and the stress parameters of the (undisturbed) stabilised organic clay samples at various strain levels measured during the isotropically consolidated undrained triaxial test.

Applied normal stress [kPa]	**Parameter**	**Unit**	**Value of parameter at axial strain (ε_{axial} [%])**								
			0,2	**0,5**	**1,0**	**2,0**	**3,0**	**4,0**	**5,0**	**6,0**	**7,0**
	ϕ'	[°]	1,82	3,02	3,01	2,81	2,45	1,48	0,61	0,00	0,00
	c'	[kPa]	16,93	24,53	32,92	40,89	44,87	47,33	48,54	49,03	48,64
16,0	s'	[kPa]	14,59	15,05	16,45	19,79	23,85	28,06	31,71	34,97	37,72
	t	[kPa]	5,21	7,83	10,18	13,91	17,71	21,16	24,06	26,34	27,85
	q	[kPa]	10,43	15,65	20,35	27,81	35,41	42,32	48,13	52,68	55,69
	p'	[kPa]	12,86	12,44	13,05	15,15	17,94	21,01	23,69	26,19	28,43
60,0	s'	[kPa]	57,04	52,87	50,01	50,53	53,46	57,06	59,78	62,26	64,39
	t	[kPa]	19,82	26,87	32,13	37,51	41,20	43,96	45,43	46,60	47,62
	q	[kPa]	39,65	53,75	64,27	75,03	82,39	87,93	90,86	93,21	95,23
	p'	[kPa]	50,44	43,92	39,30	38,03	39,72	42,41	44,64	46,73	48,51
120	s'	[kPa]	103,84	96,10	87,65	82,13	81,89	83,48	85,76	88,40	91,20
	t	[kPa]	31,27	41,62	49,01	54,75	58,62	61,86	64,56	67,06	69,14
	q	[kPa]	62,54	83,23	98,03	109,51	117,25	123,72	129,12	134,13	138,28
	p'	[kPa]	93,42	82,22	71,32	63,88	62,35	62,86	64,24	66,05	68,15

Applied normal stress [kPa]	**Parameter**	**Unit**	**Value of parameter at axial strain (ε_{axial} [%])**								
			8,0	**9,0**	**10,0**	**11,0**	**12,0**	**13,0**	**14,0**	**15,0**	**16,0**
	ϕ'	[°]	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00
	c'	[kPa]	48,40	47,99	47,67	47,24	46,95	46,65	46,35	46,07	45,89
16,0	s'	[kPa]	39,63	41,35	42,63	43,73	44,72	45,64	46,35	47,12	47,64
	t	[kPa]	28,77	29,59	30,13	30,57	31,04	31,50	31,81	32,15	32,31
	q	[kPa]	57,53	59,17	60,27	61,14	62,09	63,00	63,63	64,29	64,62
	p'	[kPa]	30,04	31,48	32,59	33,54	34,38	35,14	35,75	36,40	36,87
60,0	s'	[kPa]	66,47	68,25	69,42	69,77	69,77	69,94	69,96	69,76	69,36
	t	[kPa]	48,78	49,76	50,34	50,14	49,71	49,66	49,49	49,04	48,56
	q	[kPa]	97,56	99,51	100,67	100,28	99,42	99,31	98,98	98,09	97,12
	p'	[kPa]	50,21	51,66	52,64	53,06	53,20	53,38	53,46	53,42	53,17
120	s'	[kPa]	93,40	95,46	97,47	99,25	100,61	101,92	102,48	102,53	102,55
	t	[kPa]	70,87	72,10	73,37	74,31	75,13	75,70	75,70	75,48	75,35
	q	[kPa]	141,75	144,19	146,74	148,63	150,26	151,39	151,39	150,95	150,70
	p'	[kPa]	69,78	71,42	73,01	74,48	75,57	76,68	77,24	77,37	77,43

Applied normal stress	Parameter	Unit	Value of parameter at axial strain (ε_{axial} [%])								
			17,0	**18,0**	**19,0**	**20,0**	**21,0**	**22,0**	**23,0**	**24,0**	**25,0**
	ϕ'	**[°]**	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00
[kPa]	c'	**[kPa]**	45,79	45,75	45,64	45,55	45,31	45,11	44,85	44,66	44,46
16,0	s'	**[kPa]**	48,08	48,41	48,66	48,81	49,09	49,42	49,51	49,59	49,67
	t	**[kPa]**	32,43	32,49	32,47	32,46	32,46	32,52	32,51	32,44	32,39
	q	**[kPa]**	64,86	64,97	64,94	64,92	64,91	65,04	65,02	64,88	64,77
	p'	**[kPa]**	37,27	37,58	37,84	37,99	38,27	38,58	38,67	38,78	38,87
60,0	s'	**[kPa]**	68,95	68,80	68,51	68,31	68,09	68,01	67,54	67,28	66,95
	t	**[kPa]**	48,07	47,91	47,55	47,29	46,83	46,58	46,06	45,69	45,26
	q	**[kPa]**	96,15	95,83	95,10	94,58	93,66	93,15	92,12	91,38	90,51
	p'	**[kPa]**	52,93	52,83	52,66	52,55	52,48	52,48	52,19	52,05	51,86
120	s'	**[kPa]**	101,93	101,55	101,29	100,96	100,75	100,40	99,93	99,46	98,98
	t	**[kPa]**	74,94	74,71	74,50	74,22	73,88	73,44	72,74	72,19	71,63
	q	**[kPa]**	149,88	149,41	149,00	148,44	147,76	146,87	145,48	144,38	143,26
	p'	**[kPa]**	76,95	76,64	76,46	76,22	76,12	75,92	75,68	75,40	75,10

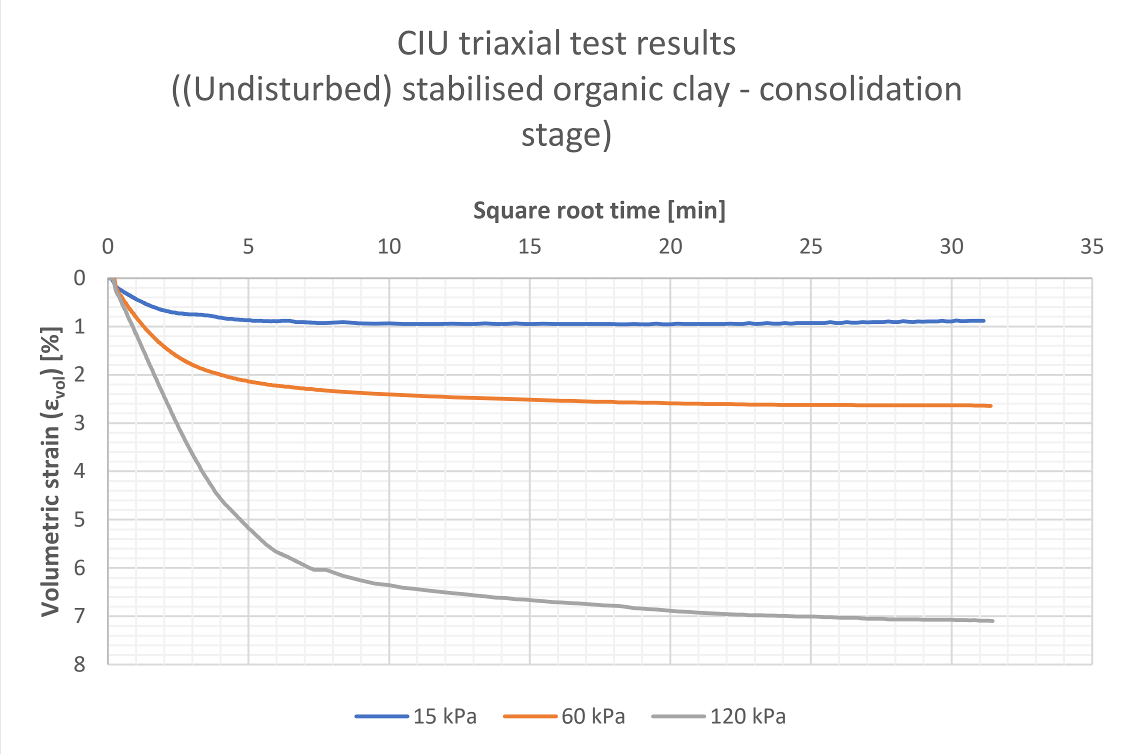


Figure F.98; The measured volumetric strain of the (undisturbed) stabilised organic clay samples during the consolidation stage of the isotropically consolidated undrained triaxial test for three different consolidation stresses.

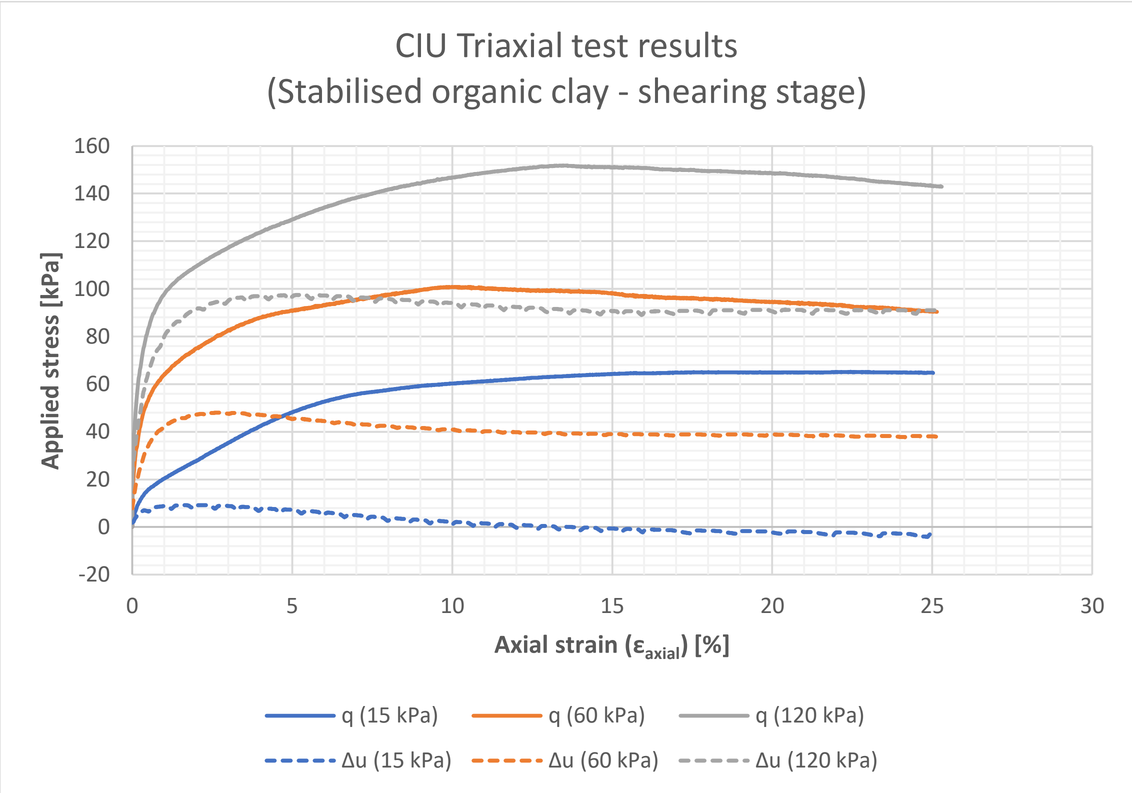


Figure F.99; The measured stress-strain responses of the (undisturbed) stabilised organic clay samples during the shearing stage of the isotropically consolidated undrained triaxial test at three different consolidation stresses. The deviator stress (q) and the change in pore pressure (Δu) during the shearing stage are indicated in the graph.

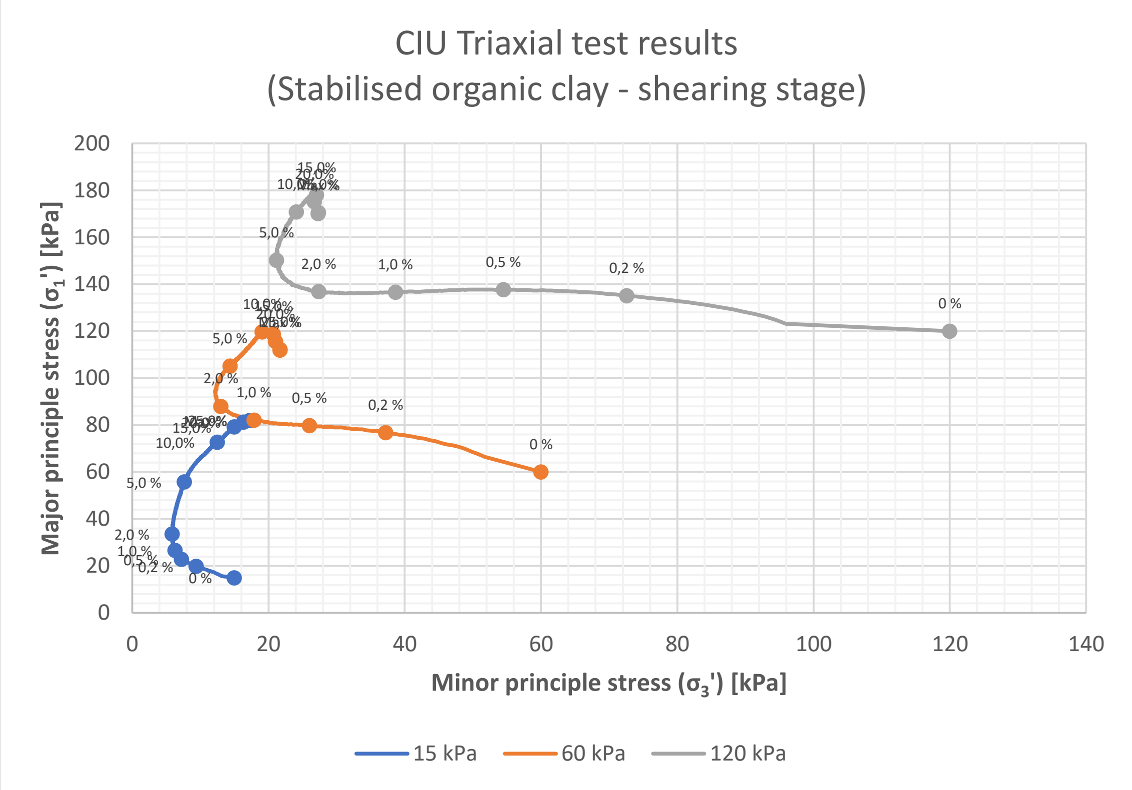


Figure F.100; The measured stress paths of the (undisturbed) stabilised organic clay samples during the shearing stage of the isotropically consolidated undrained triaxial test at three different consolidation stresses. The axial strains during shearing are indicated in the graph.

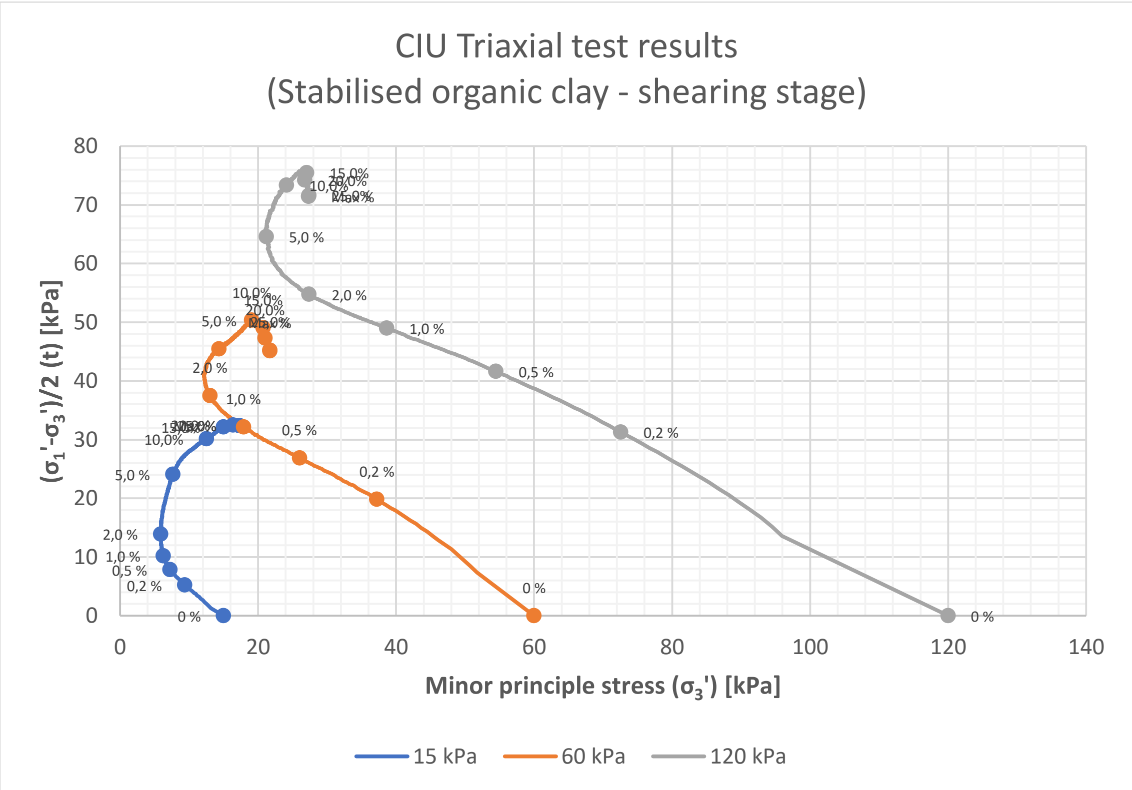


Figure F.101; The measured stress paths of the (undisturbed) stabilised organic clay samples during the shearing stage of the isotropically consolidated undrained triaxial test at three different consolidation stresses. The axial strains during shearing are indicated in the graph.

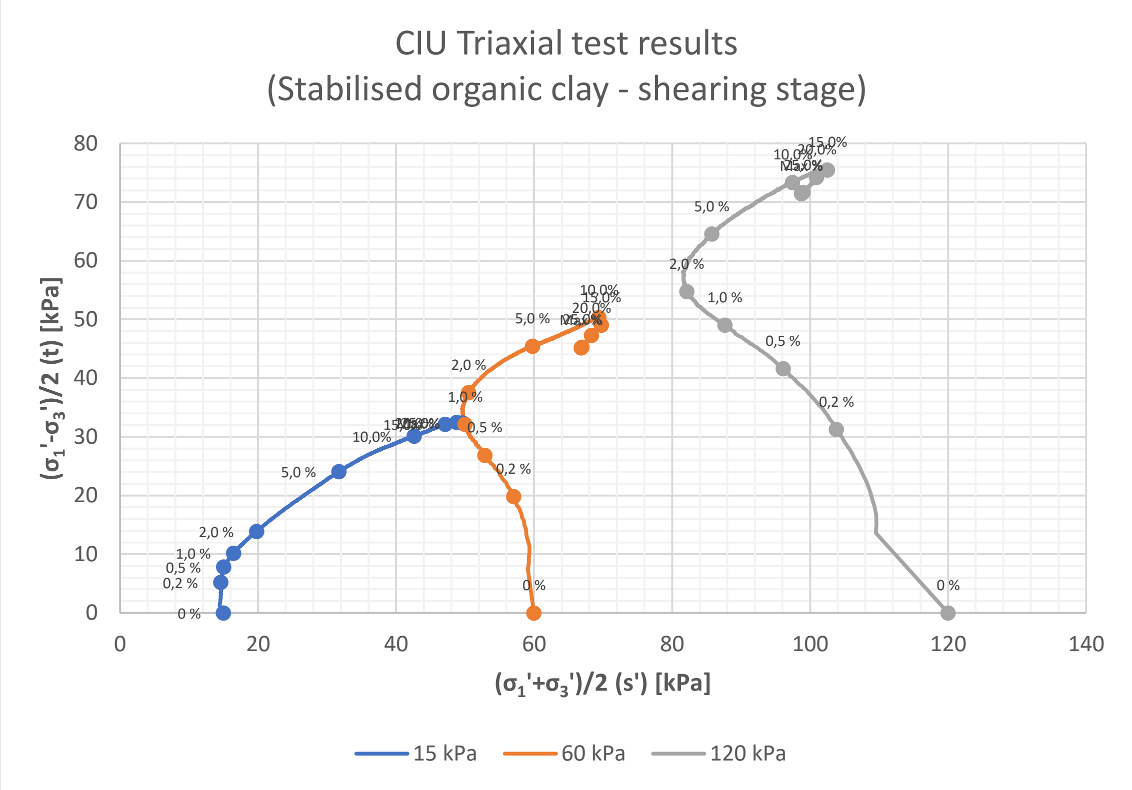


Figure F.102; The measured stress paths of the (undisturbed) stabilised organic clay samples during the shearing stage of the isotropically consolidated undrained triaxial test at three different consolidation stresses. The axial strains during shearing are indicated in the graph.

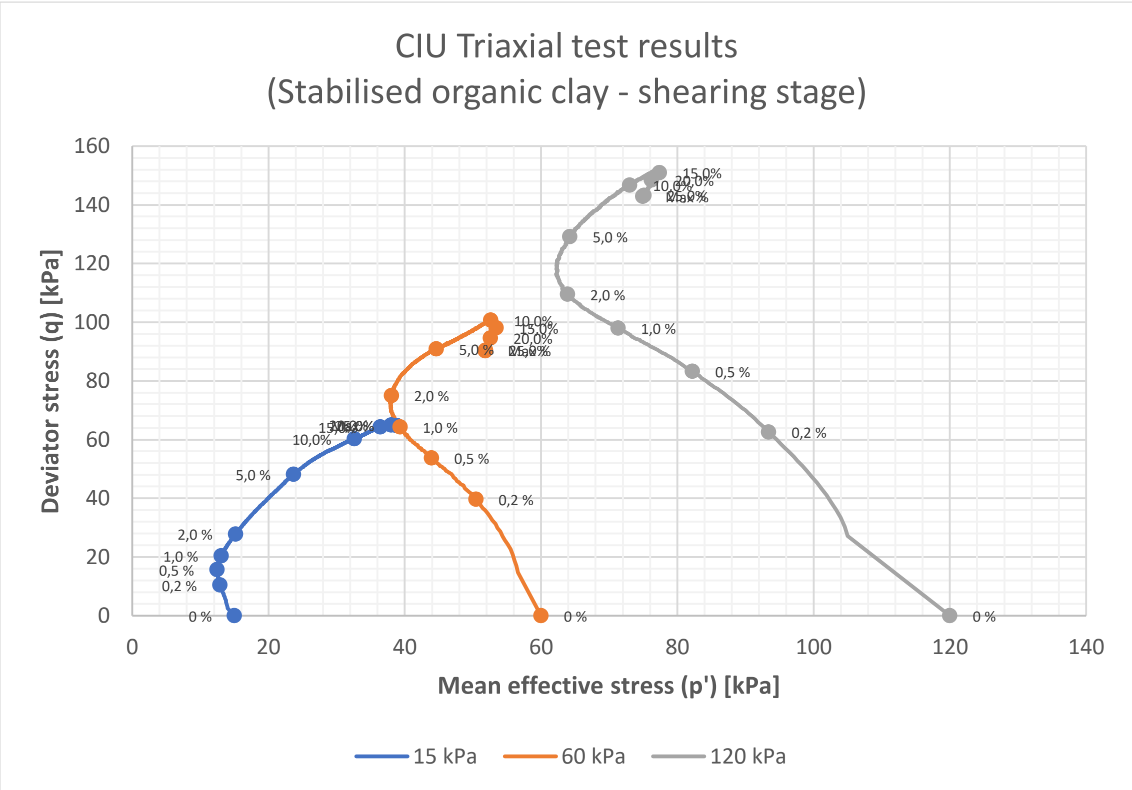


Figure F.103; The measured stress paths of the (undisturbed) stabilised organic clay samples during the shearing stage of the isotropically consolidated undrained triaxial test at three different consolidation stresses. The axial strains during shearing are indicated in the graph.

Applied normal stress [kPa]		
15,0		
60,0		
120		

Figure F.104; Photographs of the (undisturbed) stabilised organic clay samples after failure in the isotropically consolidated undrained triaxial test at three different consolidation stresses.

F.5.5 Shear box tests

Table F.47; Tested soil-binder mixture and mixture composition.

Soil type	Binder type	Dosage	Curing time	Mixture composition			
				Soil solids	Soil water	Binder solids	Added water
		[kg binder / m³ soil]	[days]	[% m/m]	[% m/m]	[% m/m]	[% m/m]
Peat	Portland cement (CEM I)	50	28	19,2	76,0	4,8	0,0

Table F.48; Properties of the (undisturbed) stabilised peat samples as measured for the shear box tests. NM = not measured.

Soil type	Applied normal stress	Taken from mould	d	h	$\gamma_{bulk;i}$	$\gamma_{dry;i}$	$w_{stab;i}$	$w_{nat;i}$	ρ_s (measured)
	[kPa]	[-]	[mm]	[mm]	[kN/m³]	[kN/m³]	[%]	[%]	[Mg/m³]
Stabilised peat	16,0	10	21,00	63,00	10,90	2,91	275	338	NM
	60,0	10	21,00	63,00	10,60	2,87	269	337	NM
	120	10	21,00	63,00	10,62	2,87	270	337	NM
	135	10	21,00	63,00	10,78	2,94	266	333	NM

Table F.49; Information about the shear box tests on the (undisturbed) stabilised peat samples.

Soil type	Applied normal stress	Sheared	Test rate	Strain reversal
	[kPa]	[-]	[mm/min]	[-]
Stabilised peat	16,0	Submerged	0,050	No
	60,0	Submerged	0,050	No
	120	Submerged	0,050	No
	135	Submerged	0,050	No

Table F.50; Data from the shear box test at failure of the (undisturbed) stabilised peat samples.

Soil type	Applied normal stress	Adopted failure criterion	$\tau_{\max}$	u_f	γ_f	c'	ϕ'
	[kPa]	[-]	[kPa]	[mm]	[%]	[kPa]	[°]
Stab. peat	16,0	Max. τ	42,0	4,64	22,5	27,36	35,2
	60,0	Max. τ	64,5	15,5	79,6		
	120	Max. τ	112	14,2	78,3		
	135	Max. τ	125	15,3	83,7		

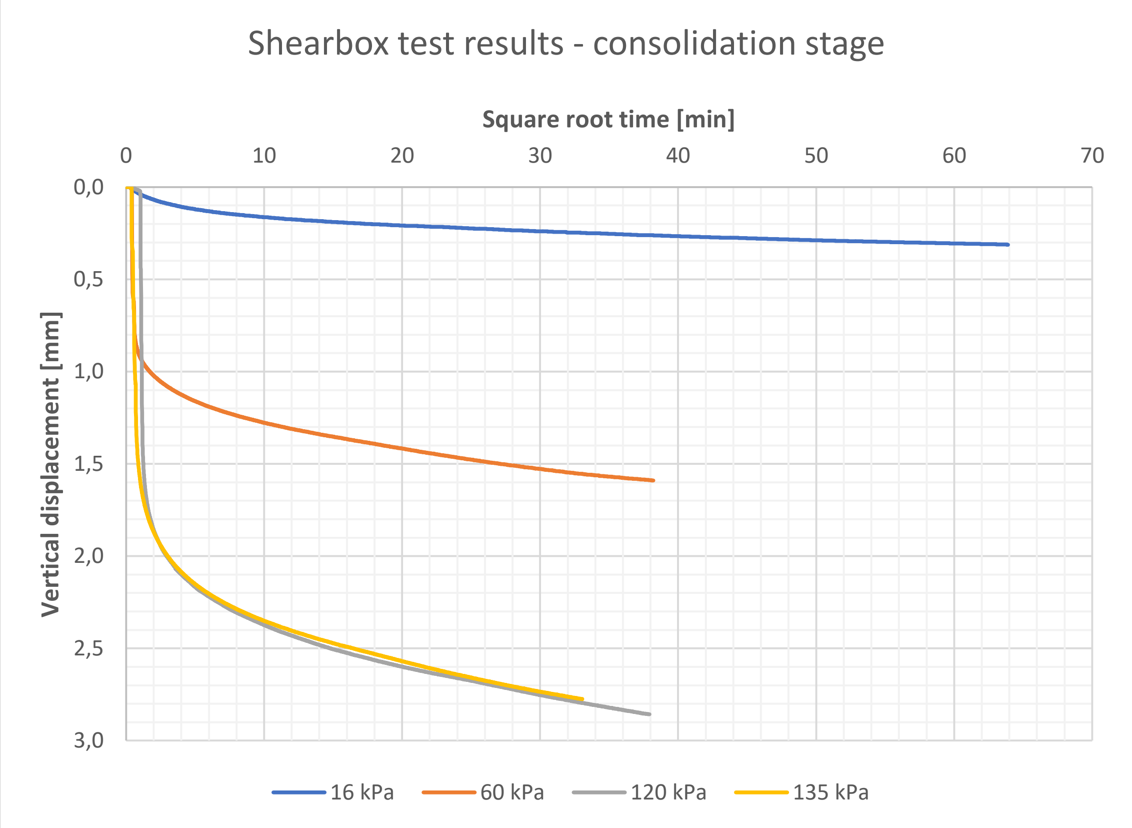


Figure F.105; The measured settlement of the (undisturbed) stabilised peat samples during the consolidation stage of the shear box tests at four different consolidation pressures.

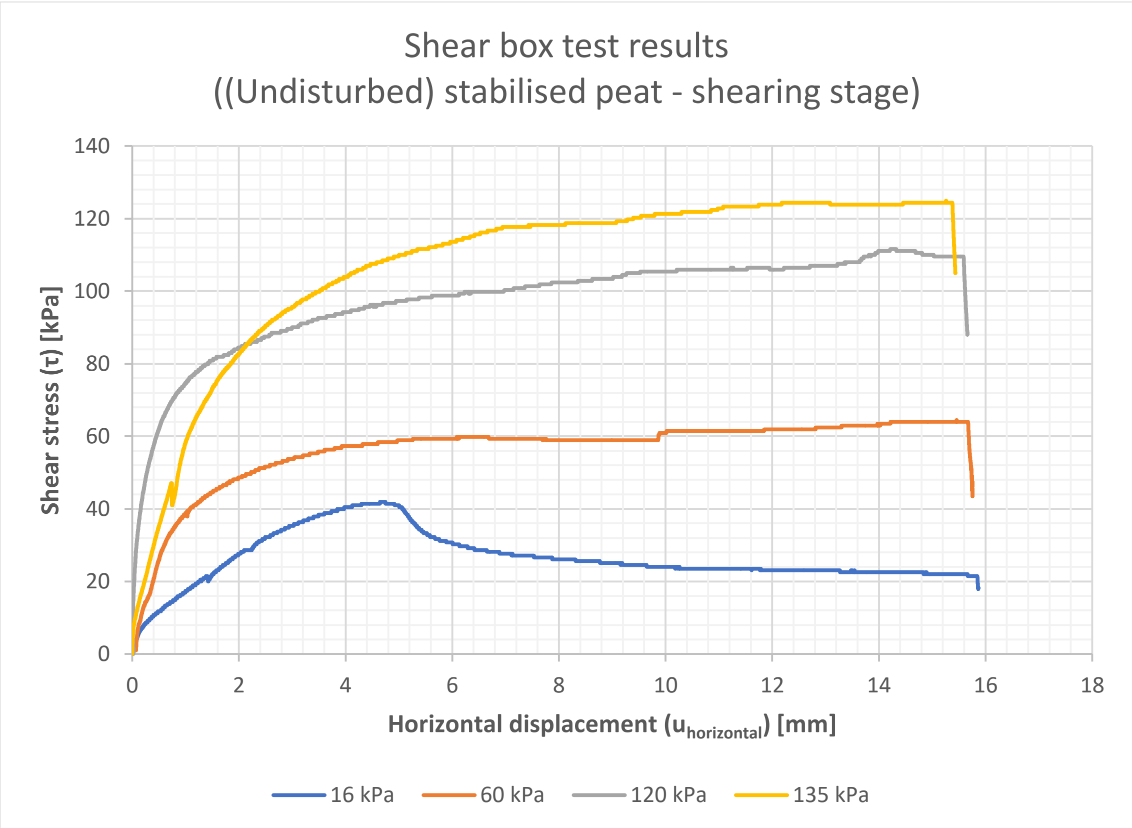


Figure F.106; The shear stress-displacement response of the (undisturbed) stabilised peat samples during the shearing stage of the shear box tests at four different consolidation pressures.

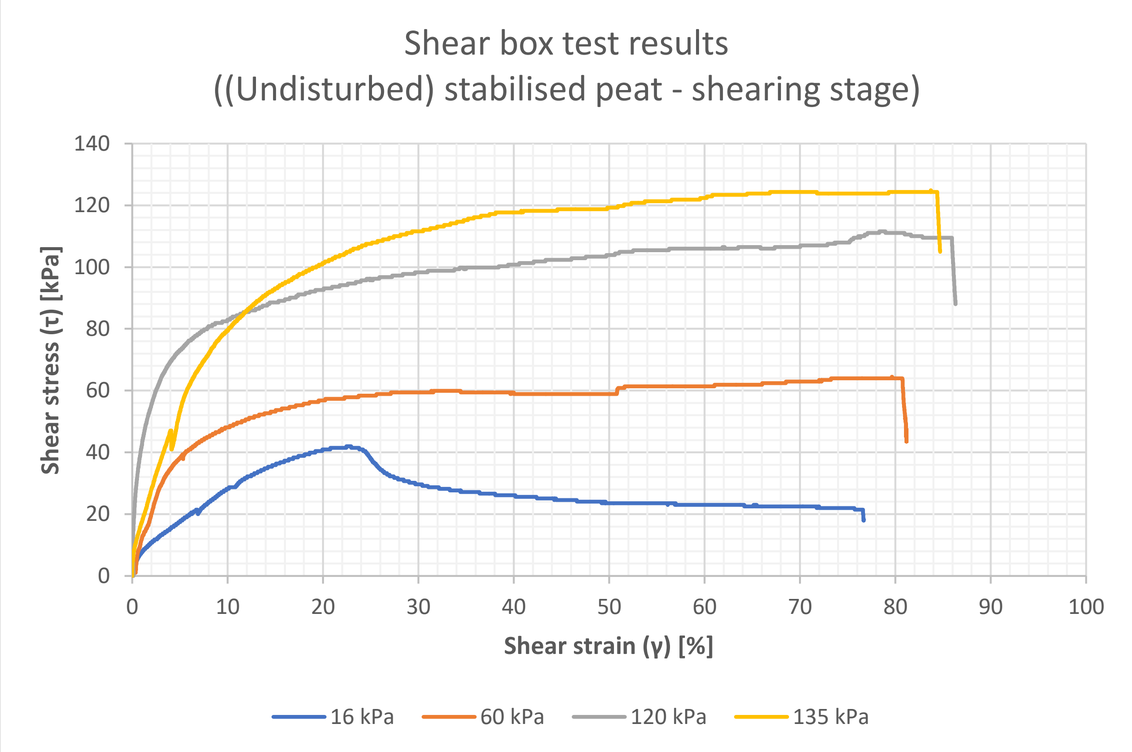


Figure F.107; The shear stress-displacement response of the (undisturbed) stabilised peat samples during the shearing stage of the shear box tests at four different consolidation pressures.

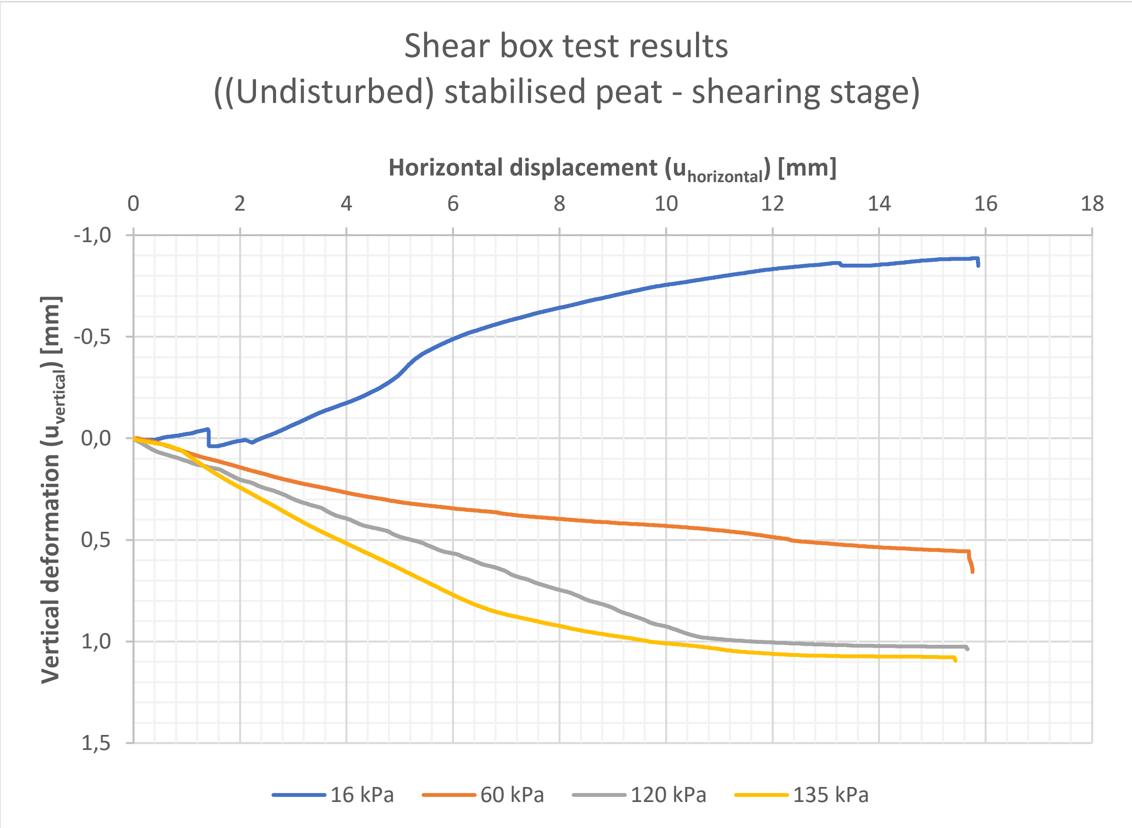


Figure F.108; The measured vertical and horizontal displacements of the (undisturbed) stabilised peat samples during the shearing stage of the shear box tests at four different consolidation pressures.

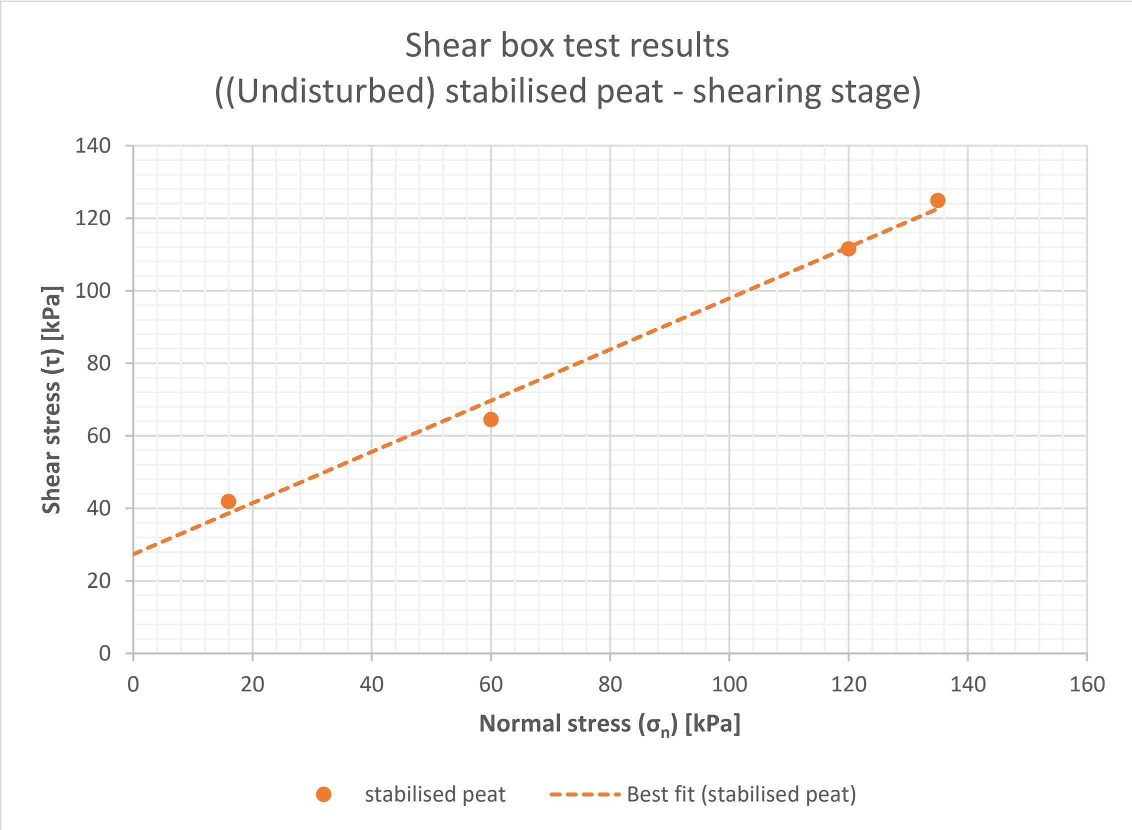


Figure F.109; The Mohr-Coulomb failure lines for the sheared (undisturbed) stabilised peat samples at peak stress.

Table F.51; The derived values of the drained shear strength parameters of the (undisturbed) stabilised peat samples at various strain levels using the measured stresses during the shear box test. N/A = not applicable.

Applied normal stress	Parameter	Unit	Value of parameter at shear strain (γ [%])								
			0,2	**0,5**	**1,0**	**2,0**	**5,0**	**10,0**	**15,0**	**20,0**	**25,0**
	ϕ'	[°]	0,00	1,60	4,49	7,07	13,94	20,85	26,15	29,97	26,33
[kPa]	c'	**[kPa]**	5,73	8,40	10,59	14,54	20,91	25,11	26,65	27,44	30,45
16,0	τ	**[kPa]**	1,54	5,12	7,68	10,75	17,40	28,15	35,83	40,95	37,87
60,0	τ	**[kPa]**	1,02	6,14	12,28	19,96	38,39	48,11	53,23	56,81	58,35
120	τ	**[kPa]**	19,96	31,73	42,48	55,28	73,19	82,92	88,55	92,64	96,22
135	τ	**[kPa]**	8,70	12,28	17,40	28,15	53,23	79,33	93,15	101,34	107,48

Applied normal stress	Parameter	Unit	Value of parameter at shear strain (γ [%])								
			30,0	**35,0**	**40,0**	**45,0**	**50,0**	**55,0**	**60,0**	**65,0**	**70,0**
	ϕ'	[°]	18,67	15,53	13,75	12,06	10,93	11,51	10,79	10,19	10,55
[kPa]	c'	**[kPa]**	34,11	35,87	36,90	37,75	38,34	38,75	39,16	39,62	39,63
16,0	τ	**[kPa]**	29,69	27,13	26,10	24,57	23,54	23,54	23,03	22,52	22,52
60,0	τ	**[kPa]**	59,37	59,37	58,86	58,86	58,86	61,42	61,42	61,93	62,95
120	τ	**[kPa]**	98,27	99,81	100,83	102,36	103,90	105,44	105,95	106,46	106,46
135	τ	**[kPa]**	111,58	115,16	117,72	118,74	119,25	121,30	122,33	123,86	124,37

Applied normal stress	Parameter	Unit	Value of parameter at shear strain (γ [%])					
			75,0	**80,0**	**85,0**	**90,0**	**95,0**	**100,0**
	ϕ'	[°]	10,55	N/A	N/A	N/A	N/A	N/A
[kPa]	c'	**[kPa]**	39,78	N/A	N/A	N/A	N/A	N/A
16,0	τ	**[kPa]**	22,01	N/A	N/A	N/A	N/A	N/A
60,0	τ	**[kPa]**	63,98	63,98	N/A	N/A	N/A	N/A
120	τ	**[kPa]**	107,99	111,07	109,53	N/A	N/A	N/A
135	τ	**[kPa]**	123,86	124,37	N/A	N/A	N/A	N/A

F.5.6 Oedometer tests

Table F.52; Tested soil-binder mixtures and mixture compositions.

Soil type	Binder type	Dosage	Curing time	Mixture composition			
				Soil solids	Soil water	Binder solids	Added water
		[kg binder / m³ soil]	[days]	[% m/m]	[% m/m]	[% m/m]	[% m/m]
Organic clay	Portland cement (CEM I)	75	28	44,4	33,7	5,5	16,5
Peat	Portland cement (CEM I)	50	28	19,2	76,0	4,8	0,0

Table F.53; Visual description of the stabilised peat and organic clay soil sample as carried out by the laboratory technician of Fugro NL Land B.V. prior to the oedometer test.

Soil type	Visual description
Stabilised peat	Peat, stabilised
Stabilised organic clay	Clay, stabilised

Table F.54; Initial properties of the stabilised peat and organic clay sample as measured before the oedometer tests.

Soil type	Sample code	Taken from mould	d	h	$\gamma_{stab.;bulk;i}$	$\gamma_{stab.;dry;i}$	$w_{stab;i}$	e_0	ρ_s (assumed)
	[-]	[-]	[mm]	[mm]	[kN/m³]	[kN/m³]	[%]	[-]	[Mg/m³]
Stab. Peat	B3	10	50,1	20,0	10,7	2,9	266	2,569	2,04
Stab. org. clay	B4	21	50,0	20,0	13,8	7,1	95,5	2,575	2,57

Table F.55; Applied loading steps in the oedometer tests on the stabilised peat and organic clay samples.

Soil type	Loading steps [kPa]								
	1	2	3	4	5	6	7	8	9
Stab. Peat	4	7	15	30	60	120	60	120	240
Stab. org. clay	4	7	15	30	60	120	60	120	240

Table F.56; Derived values of the oedometer stiffness from the results of the oedometer tests on the stabilised peat and organic clay samples.

Soil type	Sample code	Oedometer stiffness (E_{oed}) [MN/m²] at load step								
[-]	[-]	1	2	3	4	5	6	7	8	9
Stab. Peat	B3	-	-	1,7	1,6	1,5	1,1	-	-	1,1
Stab. organic clay	B4	-	-	3,0	2,7	2,4	2,0	-	-	1,9

Table F.57; Derived compression parameters from the results of the oedometer tests on the stabilised soil samples.

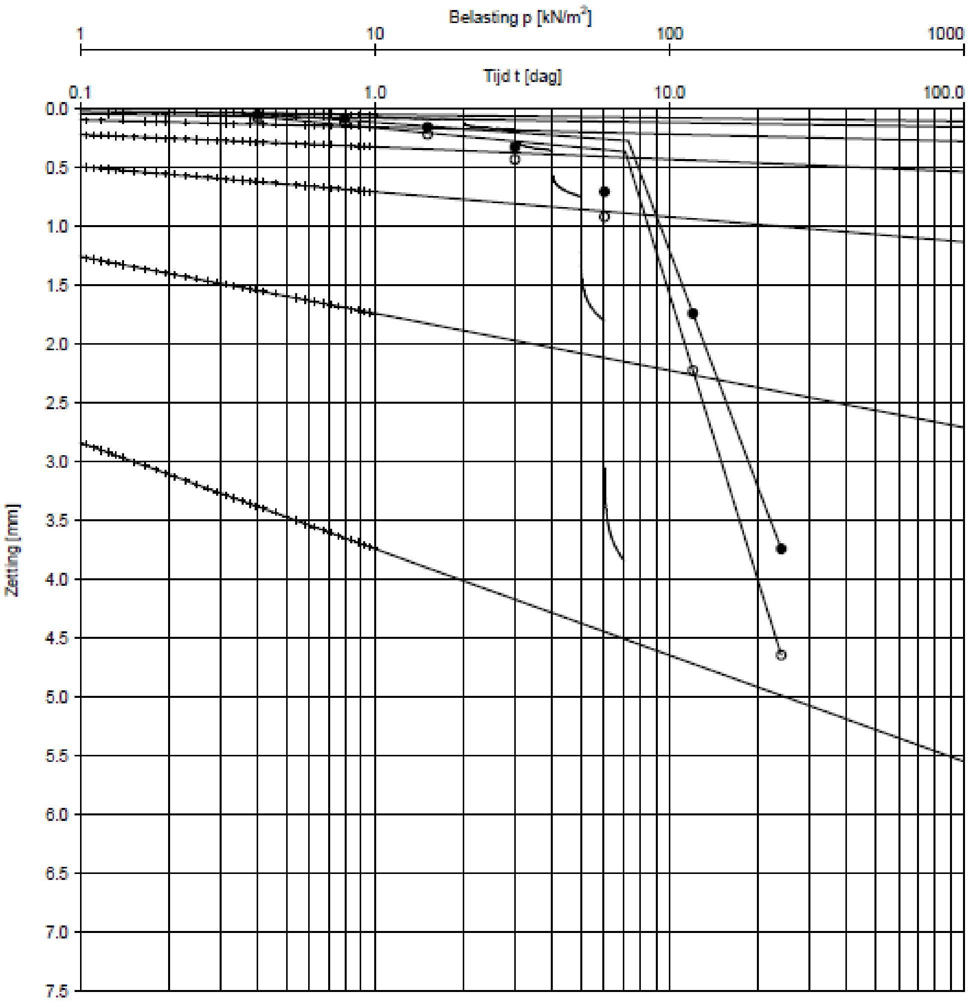
Applied method	Value of parameter						
Anglo-Saxon method (linear strain)	Soil type	Sample code	$CR\ (<\sigma'_p)$	$CR\ (>\sigma'_p)$	SR (step 6-7)	RR (step 7-8)	σ'_p
	[-]	[-]	[-]	[-]	[-]	[-]	[kPa]
	Stab. peat	B3	0,0068	0,3402	0,0126	0,0374	71
	Stab. organic clay	B4	0,0023	0,2029	0,0048	0,0112	77
Anglo-Saxon method (void ratio)	Soil type	Sample code	$C_c\ (<\sigma'_p)$	$C_c\ (>\sigma'_p)$	C_{sw} (step 6-7)	C_r (step 7-8)	σ'_p
	[-]	[-]	[-]	[-]	[-]	[-]	[kPa]
	Stab. peat	B3	0,0465	2,3367	0,0868	0,2570	71
	Stab. organic clay	B4	0,0082	0,7255	0,0173	0,0399	77
Koppejan method	Soil type	Sample code	C_p	C_s	C'_p	C'_s	σ'_p
	[-]	[-]	[-]	[-]	[-]	[-]	[kPa]
	Stab. peat	B3	251,3	823,3	6,9	32,9	73
	Stab. organic clay	B4	248,5	1167	11,4	122,3	78

Table F.58; Derived parameters from the settlement analyses of the oedometer test on stabilised peat sample B3.

Load step	Load	Taylor method			Casagrande method				
		$c_{v;10}$	m_v	$k_{v;10}$	$c_{v;10}$	m_v	$k_{v;10}$	$C_{\alpha;NEN}$	$C_{\alpha;HEAD}$
[-]	[kPa]	[m²/s]	[m²/MN]	[m/s]	[m²/s]	[m²/MN]	[m/s]	[-]	[-]
1	**4**	-	-	-	-	-	-	-	-
2	**8**	-	-	-	-	-	-	-	-
3	**15**	$1{,}8\cdot10^{-7}$	$5{,}8\cdot10^{-1}$	$1{,}1\cdot10^{-9}$	N/A	$5{,}8\cdot10^{-1}$	N/A	$1{,}9\cdot10^{-3}$	$1{,}9\cdot10^{-3}$
4	**30**	$5{,}7\cdot10^{-7}$	$6{,}1\cdot10^{-1}$	$3{,}4\cdot10^{-9}$	N/A	$6{,}1\cdot10^{-1}$	N/A	$2{,}8\cdot10^{-3}$	$2{,}7\cdot10^{-3}$
5	**60**	$9{,}6\cdot10^{-7}$	$6{,}6\cdot10^{-1}$	$6{,}3\cdot10^{-9}$	N/A	$6{,}6\cdot10^{-1}$	N/A	$6{,}6\cdot10^{-3}$	$6{,}5\cdot10^{-3}$
6	**120**	$4{,}7\cdot10^{-7}$	$9{,}1\cdot10^{-1}$	$4{,}3\cdot10^{-9}$	N/A	$9{,}1\cdot10^{-1}$	N/A	$1{,}5\cdot10^{-2}$	$1{,}5\cdot10^{-2}$
7	**60**	-	-	-	-	-	-	-	-
8	**120**	-	-	-	-	-	-	-	-
9	**240**	$1{,}7\cdot10^{-7}$	$8{,}8\cdot10^{-1}$	$1{,}4\cdot10^{-9}$	N/A	$8{,}8\cdot10^{-1}$	N/A	$2{,}4\cdot10^{-2}$	$2{,}1\cdot10^{-2}$

Table F.59; Derived parameters from the settlement analyses of the oedometer test on stabilised org. clay sample B4.

Load step	Load	Taylor method			Casagrande method				
		$c_{v;10}$	m_v	$k_{v;10}$	$c_{v;10}$	m_v	$k_{v;10}$	$C_{\alpha;NEN}$	$C_{\alpha;HEAD}$
[-]	[kPa]	[m²/s]	[m²/MN]	[m/s]	[m²/s]	[m²/MN]	[m/s]	[-]	[-]
1	**4**	-	-	-	-	-	-	-	-
2	**8**	-	-	-	-	-	-	-	-
3	**15**	$5{,}1\cdot10^{-7}$	$3{,}3\cdot10^{-1}$	$1{,}7\cdot10^{-9}$	N/A	$3{,}3\cdot10^{-1}$	N/A	$8{,}6\cdot10^{-4}$	$8{,}6\cdot10^{-4}$
4	**30**	$4{,}6\cdot10^{-7}$	$3{,}7\cdot10^{-1}$	$1{,}7\cdot10^{-9}$	N/A	$3{,}7\cdot10^{-1}$	N/A	$1{,}1\cdot10^{-3}$	$1{,}1\cdot10^{-3}$
5	**60**	$4{,}3\cdot10^{-7}$	$4{,}3\cdot10^{-1}$	$1{,}8\cdot10^{-9}$	$4{,}0\cdot10^{-7}$	$4{,}3\cdot10^{-1}$	$1{,}7\cdot10^{-9}$	$1{,}8\cdot10^{-3}$	$1{,}8\cdot10^{-3}$
6	**120**	$2{,}0\cdot10^{-7}$	$4{,}9\cdot10^{-1}$	$1{,}0\cdot10^{-9}$	$1{,}5\cdot10^{-7}$	$4{,}9\cdot10^{-1}$	$7{,}3\cdot10^{-10}$	$4{,}1\cdot10^{-3}$	$4{,}0\cdot10^{-3}$
7	**60**	-	-	-	-	-	-	-	-
8	**120**	-	-	-	-	-	-	-	-
9	**240**	$1{,}9\cdot10^{-7}$	$5{,}2\cdot10^{-1}$	$1{,}0\cdot10^{-9}$	$1{,}0\cdot10^{-7}$	$5{,}2\cdot10^{-1}$	$5{,}3\cdot10^{-10}$	$6{,}2\cdot10^{-3}$	$5{,}9\cdot10^{-3}$



Boring	: B3		C	=	113.2	
Monster	: Veen gestab.		C'	=	3.8	
Preparatie	: Trimmen		σ'_p	=	73	kN/m²
Diepte	: 0.00 m t.o.v. maaiveld					
Grondsoort	: GESTABILISEERD veen (50 kg/m3 cem i)		C_p	=	251.3	
Proefstukorientatie	: Verticaal		C_s	=	823.3	
γ	:	10.7 kN/m³				
γ_{dr}	:	2.9 kN/m³	C'_p	=	6.9	
w	:	266.3 % [m/m]	C'_s	=	32.9	
			H_0	=	20.0	mm
			D	=	50.1	mm

Uitgevoerd conform NEN 5118: 1991 / A1: 1997 nl

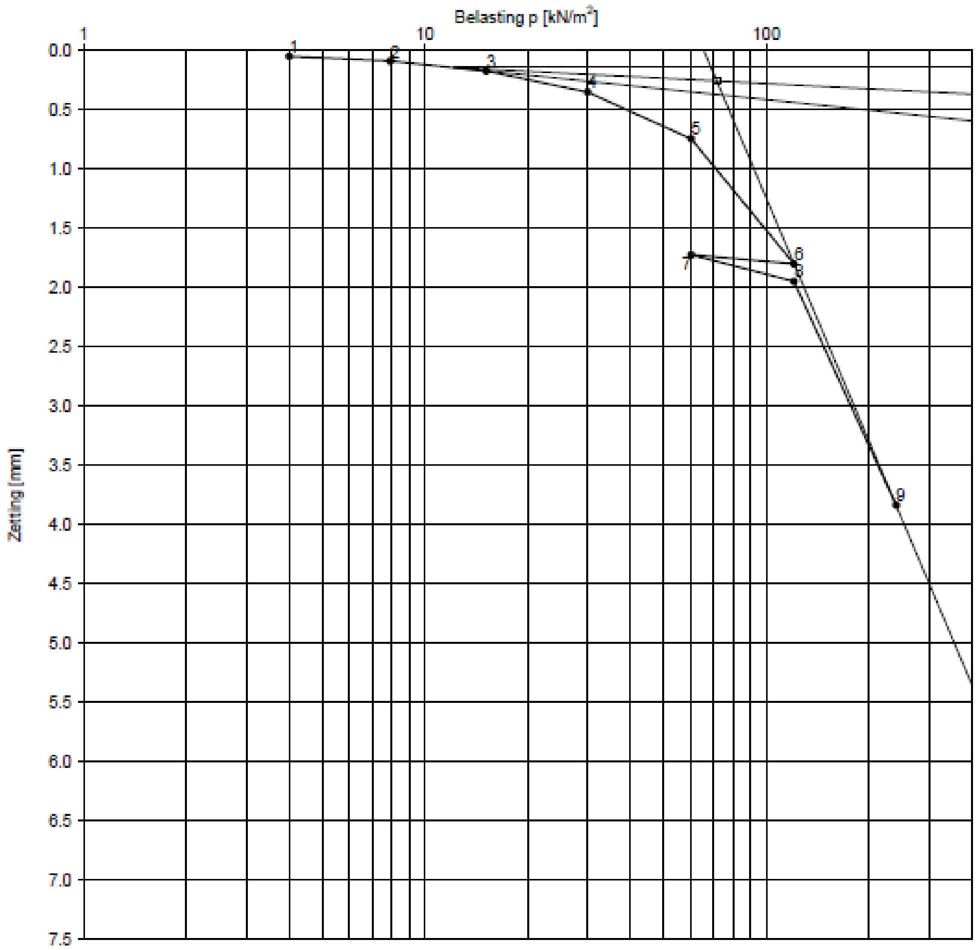
FugroNL Land B.V. Arnhem OED 605.0016 /11:10:57/P:\VEEN GESTAB.OED Opg.:NVV dd:11-Apr-2019

Samendrukkingsproef methode KOPPEJAN

Opdr. 1718-0800-000

Afstudeeropdracht: Onderzoek Massastabilisatie 2018-2019

Figure F.110; Compression-stress plot from the results of the oedometer test on <u>stabilised peat sample B3</u> for the Koppejan method.



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Boring : B3
Monster : Veen gestab.
Preparatie : Trimmen
Diepte : 0.00 m t.o.v. maaiveld
Grondsoort : GESTABILISEERD veen (50 kg/m3 cem i)
Proefstukorientatie : Verticaal

γ : 10.7 kN/m³
γ_{dr} : 2.9 kN/m³
w : 266.3 % [m/m]

Grensspanning
σ'_p = 71 kN/m²
Primaire samendrukkings getal CR
belasting < σ'_p = 0.0068
belasting > σ'_p = 0.3402
SR Zwelgetal 6-7 = 0.0126
RR Herbelastingsgetal 7-8 = 0.0374
H_0 = 20.0 mm
D = 50.1 mm

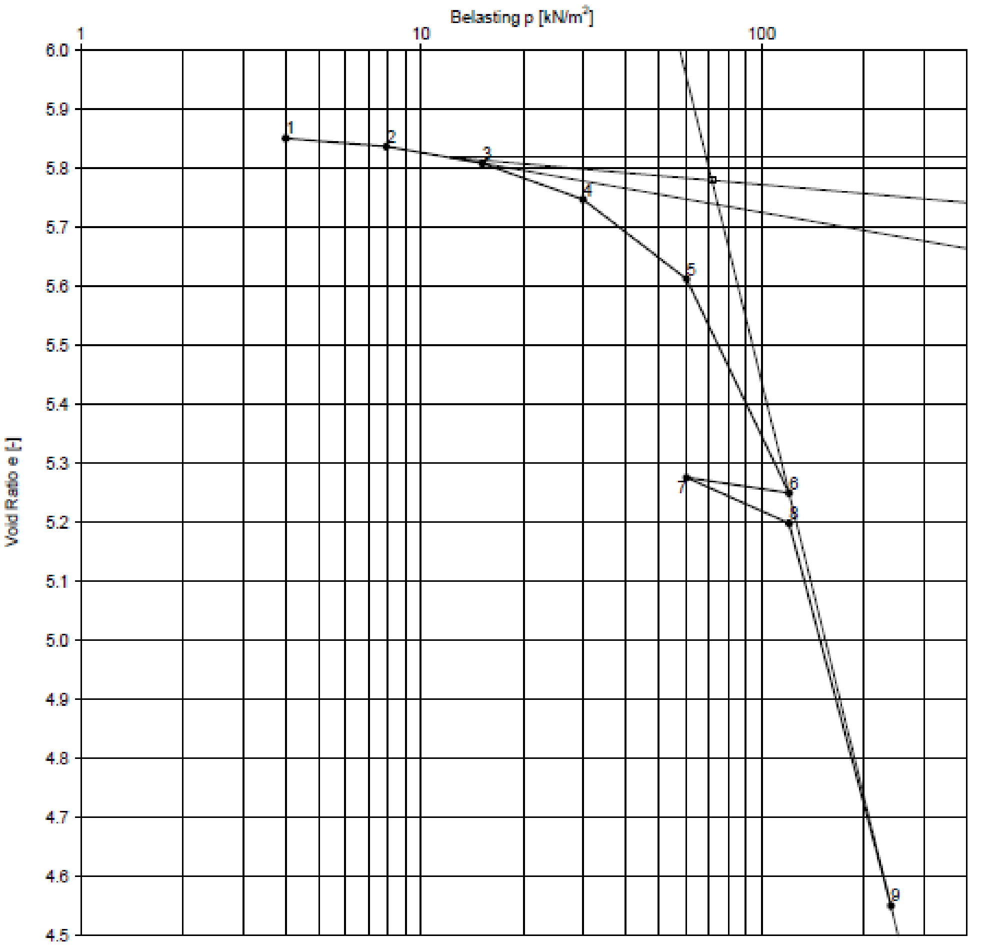
Uitgevoerd conform NEN 5118: 1991 / A1: 1997 nl

Samendrukkingsproef resultaten z-log p

Opdr. 1718-0800-000

Afstudeeropdracht: Onderzoek Massastabilisatie 2018-2019

Figure F.111; Compression-stress plot from the results of the oedometer test on <u>stabilised peat sample B3</u> for the Anglo-Saxon method (linear strain).



Boring	: B3	Grensspanning			
Monster	: Veen gestab.	σ'_p	=	71	kN/m²
Preparatie	: Trimmen	Primaire samendrukkings index C_c			
Diepte	: 0.00 m t.o.v. maaiveld	belasting < σ'_p	=	0.0465	
Grondsoort	: GESTABILISEERD veen (50 kg/m3 cem i)	belasting > σ'_p	=	2.3367	
		C_{sw} Zwelindex 6-7	=	0.0868	
Proefstukorientatie	: Verticaal	C_r Herbelastingsindex 7-8	=	0.2570	
		H_0	=	20.0	mm
γ	: 10.7 kN/m³	D	=	50.1	mm
γ_{dr}	: 2.9 kN/m³				
w	: 266.3 % [m/m]				
Void ratio e_0	: 5.869 (bij ρ = 2.04 t/m³)				

Uitgevoerd conform NEN 5118: 1991 / A1: 1997 nl

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Samendrukkingsproef resultaten e-log p

Opdr. 1718-0800-000

Afstudeeropdracht: Onderzoek Massastabilisatie 2018-2019

Figure F.112; Compression-stress plot from the results of the oedometer test on *__stabilised peat sample B3__* *for the Anglo-Saxon method (void ratio).*

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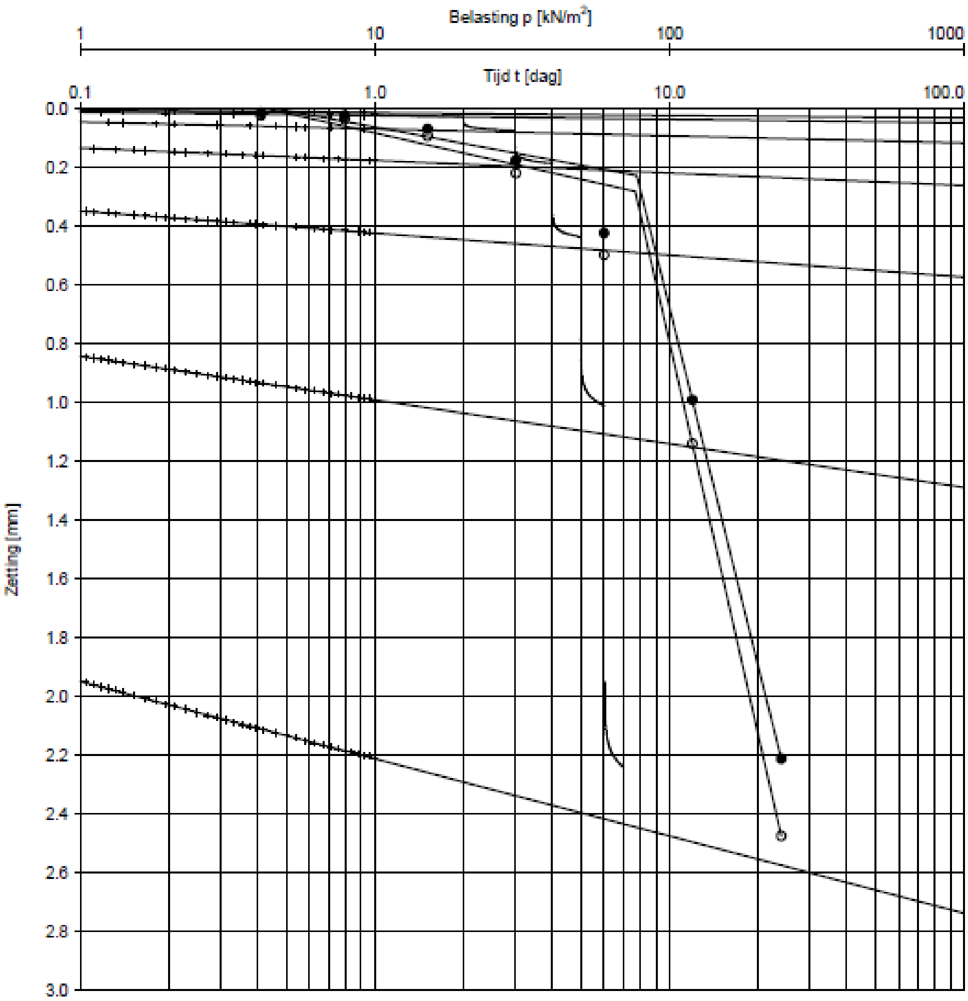

Boring	: B4	C	=	134.2	
Monster	: Klei gestab.	C'	=	8.3	
Preparatie	: Trimmen	σ'_p	=	78	kN/m²
Diepte	: 0.00 m t.o.v. maaiveld				
Grondsoort	: GESTABILISEERDE klei (75 kg/m3 cem i)	C_p	=	248.5	
Proefstukorientatie	: Verticaal	C_s	=	1167	
γ	: 13.8 kN/m³	C'_p	=	11.4	
γ_{dr}	: 7.1 kN/m³	C'_s	=	122.3	
w	: 95.5 % [m/m]				
		H_0	=	20.0	mm
		D	=	50.0	mm

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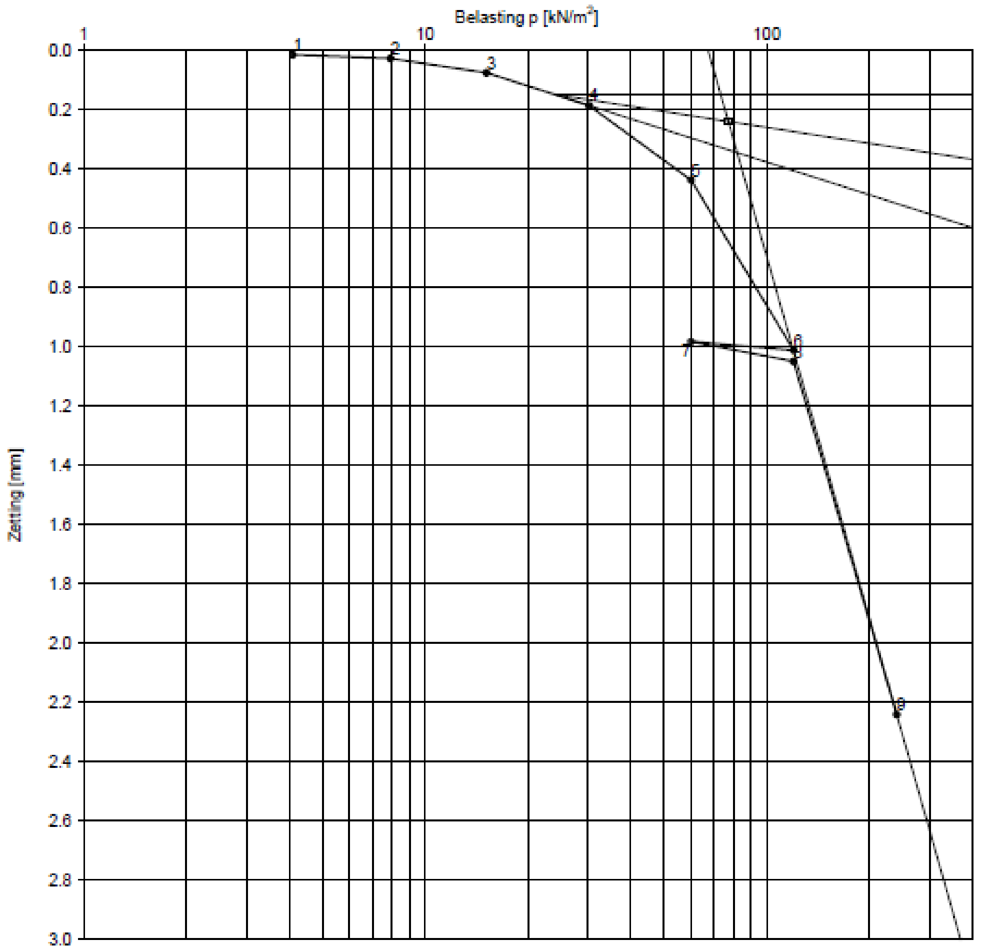
Uitgevoerd conform NEN 5118: 1991 / A1: 1997 nl

Samendrukkingsproef methode KOPPEJAN

Opdr. 1718-0800-000

Afstudeeropdracht: Onderzoek Massastabilisatie 2018-2019

Figure F.113; Compression-stress plot from the results of the oedometer test on <u>*stabilised organic clay sample B4*</u> *for the Koppejan method.*



Boring	: B4	Grensspanning			
Monster	: Klei gestab.	σ'_p	=	77	kN/m²
Preparatie	: Trimmen	Primaire samendrukkings getal CR			
Diepte	: 0.00 m t.o.v. maaiveld	belasting < σ'_p	=	0.0023	
Grondsoort	: GESTABILISEERDE klei (75 kg/m3 cem i)	belasting > σ'_p	=	0.2029	
		SR Zwelgetal 6-7	=	0.0048	
Proefstukorientatie	: Verticaal	RR Herbelastingsgetal 7-8	=	0.0112	
		H_0	=	20.0	mm
γ	: 13.8 kN/m³	D	=	50.0	mm
γ_{dr}	: 7.1 kN/m³				
w	: 95.5 % [m/m]				

FugroNL Land B.V. Arnhem OED 6.05.0016 /11:13:15P KLB GESTAB.OED Opg.:NW dd:11-Apr-2019

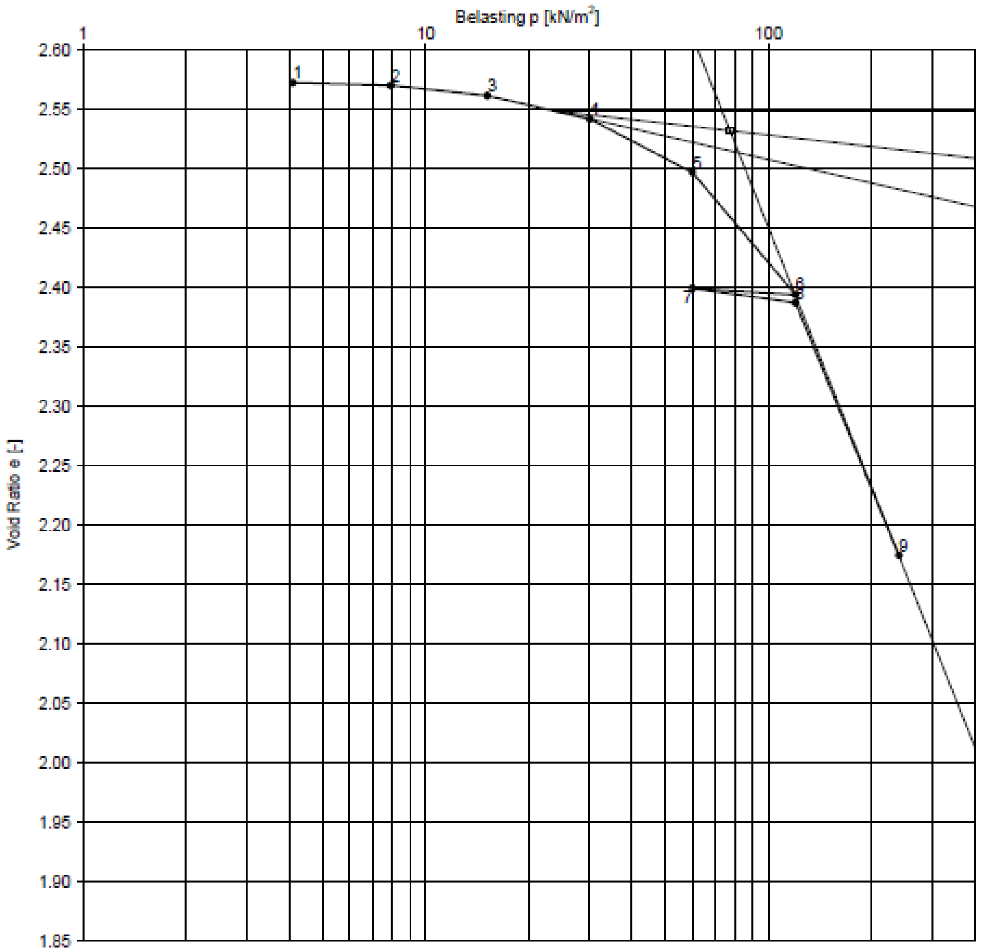
Uitgevoerd conform NEN 5118: 1991 / A1: 1997 nl

Samendrukkingsproef resultaten z-log p

Opdr. 1718-0800-000

Afstudeeropdracht: Onderzoek Massastabilisatie 2018-2019

Figure F.114; Compression-stress plot from the results of the oedometer test on <u>stabilised organic clay sample B4</u> for the Anglo-Saxon method (linear strain).



Boring	:	B4
Monster	:	Klei gestab.
Preparatie	:	Trimmen
Diepte	:	0.00 m t.o.v. maaiveld
Grondsoort	:	GESTABILISEERDE klei (75 kg/m3 cem i)
Proefstukorientatie	:	Verticaal
γ	:	13.8 kN/m³
γ_{dr}	:	7.1 kN/m³
w	:	95.5 % [m/m]
Void ratio e_0	:	2.575 (bij ρ = 2.57 t/m³)

Grensspanning

σ'_p	=	77	kN/m²
Primaire samendrukkings index C_c			
belasting < σ'_p	=	0.0082	
belasting > σ'_p	=	0.7255	
C_{sw} Zwelindex 6-7	=	0.0173	
C_r Herbelastingsindex 7-8	=	0.0399	
H_0	=	20.0	mm
D	=	50.0	mm

Uitgevoerd conform NEN 5118: 1991 / A1: 1997 nl

Samendrukkingsproef resultaten e-log p

Opdr. 1718-0800-000

Afstudeeropdracht: Onderzoek Massastabilisatie 2018-2019

Figure F.115; Compression-stress plot from the results of the oedometer test on _stabilised organic clay sample B4_ *for the Anglo-Saxon method (void ratio).*

Appendix G - Binder specifications

G.1 Portland cement

In this research, Portland cement CEM I 52,5 R was used from the producer ENCI Heidelberg Cement Group. Information on the chemical and physical properties of this specific Portland cement is provided in table g.1 through table g.5

Table G.1; The composition of the used Portland cement CEM I 52,5 R. The components are presented in percent of the sum of the main and side components (ENCI Heidelberg Group, 2018).

Composition of CEM I 52,5 R		
Component	**Average value**	**Unit**
Clinker	100	% m/m
Filler	0	% m/m

Table G.2; The additions to the main composition of the used Portland cement CEM I 52,5 R. The components are presented as additions in percent of the cement (ENCI Heidelberg Group, 2018).

Additions to CEM I 52,5 R		
Component	**Average value**	**Unit**
Binder time regulator	5,3	% m/m
Grinding aids	0,019	% m/m
Reducing agent	0,28	% m/m

Table G.3; The chemical composition of the used Portland cement CEM I 52,5 R (ENCI Heidelberg Group, 2018).

Chemical composition of CEM I 52,5 R		
Chemical compound	**Average value**	**Unit**
CaO	64	%
SiO_2	20	%
Al_2O_3	5,06	%
Fe_2O_3	3	%
SO_3	3,24	%
Insoluble residue	0,62	%
Loss on ignition	1	%
Chloride	0,05	%
Chrome (IV)	<0,0002	%
Na_2O equivalent	-	%

Table G.4; The physical characteristics of the used Portland cement CEM I 52,5 R (ENCI Heidelberg Group, 2018).

Physical characteristics of CEM I 52,5 R		
Parameter	**Average value**	**Unit**
Specific surface area (Blaine number)	5308	cm^2/g
Absolute density (particle density)	3140	kg/m^3
Bulk density	1040	kg/m^3

Table G.5; The binding characteristics of the used Portland cement CEM I 52,5 R (ENCI Heidelberg Group, 2018).

Binding characteristics of CEM I 52,5 R			
Parameter	**Average value**	**Minimum value**	**Unit**
Start of binding	106	45	minutes
End of binding	149	-	minutes

G.2 Blast-furnace slag cement

In this research, blast-furnace slag CEM III/B 42,5 N-LH/HS was used from the producer ENCI Heidelberg Cement Group. Information on the chemical and physical properties of this specific blast-furnace slag cement is provided in table g.6 through table g.9

Table G.6; The composition of the used blast-furnace slag cement CEM III/B 42,5 N-LH/HS. The components are presented in percent of the sum of the main and side components (ENCI Heidelberg Cement Group, 2018).

Composition of CEM III/B 42,5 N-LH/HS		
Component	**Average value**	**Unit**
Clinker	28	% m/m
Blast-furnace slag	72	% m/m
Filler	0	% m/m

Table G.7; The additions to the main composition of the used blast-furnace slag cement CEM III/B 42,5 N-LH/HS. The components are presented as additions in percent of the cement (ENCI Heidelberg Cement Group, 2018).

Additions to CEM III/B 42,5 N-LH/HS		
Component	**Average value**	**Unit**
Binder time regulator	6,7	% m/m
Grinding aids	0,097	% m/m
Reducing agent	0	% m/m

Table G.8; The chemical composition of the used blast-furnace slag cement CEM III/B 42,5 N-LH/HS (ENCI Heidelberg Cement Group, 2018).

Chemical composition of CEM III/B 42,5 N-LH/HS		
Chemical compound	**Average value**	**Unit**
CaO	46	%
SiO_2	29	%
Al_2O_3	9,46	%
Fe_2O_3	1	%
SO_3	3,01	%
Insoluble residue	0,88	%
Loss on ignition	1	%
Chloride	0,08	%
Chrome (IV)	<0,0002	%
Na_2O equivalent	0,52	%

Table G.9; The physical characteristics of the used blast-furnace slag cement CEM III/B 42,5 N-LH/HS (ENCI Heidelberg Cement Group, 2018).

Physical characteristics of CEM III/B 42,5 N-LH/HS		
Parameter	**Average value**	**Unit**
Specific surface area (Blaine number)	4879	cm^2/g
Absolute density (particle density)	2970	kg/m^3
Bulk density	1040	kg/m^3

Table G.10; The binding characteristics of the used blast-furnace slag cement CEM III/B 42,5 N-LH/HS (ENCI Heidelberg Cement Group, 2018).

Binding characteristics of CEM I 52,5 R			
Parameter	**Average value**	**Minimum value**	**Unit**
Start of binding	227	60	minutes
End of binding	280	-	minutes

G.3 Flue gas desulphurisation gypsum

In this research, flue gas desulphurisation gypsum (FGD-gypsum) was used from the producer Vliegasunie. Information on the chemical and physical properties of this specific gypsum is provided in table g.11 and table g.12.

Table G.11; The chemical composition of the used FGD-gypsum (Haanappel, 2018).

Chemical composition of FGD-gypsum		
Component	**Average value**	**Unit**
$CaSO_4 \cdot 2H_2O$	96,08	% m/m
$CaCO_3$	1,92	% m/m
Residual components (mostly SiO_2 and Al_2O_3)	2,00	% m/m

Table G.12; The physical characteristics of the used FGD-gypsum (Vliegasunie, 2016).

Physical characteristics of FGD-gypsum		
Parameter	**Average value**	**Unit**
Absolute density (particle density)	2960	kg/m^3
Bulk density	1300 - 1500	kg/m^3

G.4 Blast-furnace slag

In this research, blast-furnace slag was used from the producer Ecocem Benelux B.V.. Information on the chemical and physical properties of this specific blast-furnace slag is provided in table g.13 and table g.14.

Table G.13; The chemical composition of the used blast-furnace slag (Ecocem Benelux B.V., 2018)..

Chemical composition of blast-furnace slag		
Chemical compound	**Average value**	**Unit**
CaO	38,77	% m/m
SiO_2	33,46	% m/m
MgO	8,89	% m/m
Al_2O_3	12,64	% m/m
Fe_2O_3	0,47	% m/m
Mn_2O_3	0,31	% m/m
Cl^-	0,028	% m/m
S^{2-}	0,89	% m/m
SO_3	0,04	% m/m
Na_2O	0,32	% m/m
K_2O	0,56	% m/m
Na_2O equivalent	0,69	% m/m
Loss on ignition	0,22	% m/m
Insoluble residue	0,40	% m/m
Glass content	100	% m/m

Table G.14; The physical characteristics of the used blast-furnace slag (Ecocem Benelux B.V., 2018).

Physical characteristics of blast-furnace slag		
Parameter	**Average value**	**Unit**
Specific surface area (Blaine number)	394 – 439	m^2/kg
Absolute density (particle density)	2890	kg/m^3
Bulk density	-	kg/m^3

List of references

Ecocem Benelux B.V. (2018). *Product information sheet blast-furnace slag.* Moerdijk: Ecocem Benelux B.V.

ENCI Heidelberg Cement Group. (2018). *Product sheet blast-furnace slag cement (CEM III/B 42,5 N-LH/SR).* Maastricht: Enci Heidelberg Group.

ENCI Heidelberg Group. (2018). *Product sheet Portland cement (CEM I 52,5 R).* Maastricht: ENCI Heidelberg Group.

Haanappel, N. (2018, October 11th). FGD-gypsum composition (e-mail).

Vliegasunie. (2016). *Safety information sheet FGD-gypsum.* Culemborg: Vliegasunie.

Appendix H - Implementation analyses results

H.1 Parameter determination

During the implementation analyses, the stabilised soil was assigned a different value of the effective strength parameters (i.e. effective cohesion and effective angle of internal friction) depending on the amount of time the specific stabilised soil had cured thus far. Also, depending on the scenario examined, the stabilised soil may also be assigned a different unit weight. The determination of the parameters that were assigned to the various stabilised soils present in the model are presented in this section.

H.1.1 Effective strength parameters

During the laboratory research, the properties of a peat stabilised with 50 kg Portland cement per cubic metre undisturbed peat and an organic clay stabilised with 75 kg Portland cement per cubic metre undisturbed organic clay were examined. The strength properties of these stabilised soil samples were measured, which included the development of the unconfined compressive strength in time up to 28 days of curing and the effective strength parameters (i.e. effective cohesion and effective angle of internal friction) after 28 days of curing. Since it was not possible to measure the effective strength parameters for these stabilised soils in time, both the strength development and the final strength measurements were used to estimate the development of the effective strength parameters in time.

It was assumed that the percentage increase in the drained shear strength was directly proportional to the percentage increase in the undrained shear strength as measured from unconfined compression tests. Under this assumption, this also meant that the percentage increase in the effective cohesion and the tangent of the effective angle of internal friction were assumed to be directly proportional to the percentage increase in the undrained shear strength when evaluating the drained shear strength with the Mohr-Coulomb equation. This assumption resulted in equations (H-1) and (H-2).

$$\tau_t = \alpha \cdot \tau_{28-day} = \alpha \cdot c'_{28-day} + \sigma'_n \cdot \alpha \cdot \tan(\phi'_{28-day}) \quad \textbf{(H-1)}$$

$$\alpha = \frac{UCS_t}{UCS_{28-day}} = \frac{S_{u;t;\sigma_3=0}}{S_{u;28-day;\sigma_3=0}} \quad \textbf{(H-2)}$$

<u>*where:*</u>

τ_t	*- drained shear strength at any curing time (t)*	*[kPa]*
τ_{28-day}	*- drained shear strength after 28 days of curing*	*[kPa]*
α	*- scaling factor to account for percentage increase in strength ($\alpha = 1{,}0$ at 28 days of curing and $\alpha < 1{,}0$ at earlier curing times)*	*[-]*
c'_{28-day}	*- effective cohesion after 28 days of curing*	*[kPa]*
σ'_n	*- effective normal stress*	*[kPa]*
ϕ'_{28-day}	*- effective angle of internal friction after 28 days of curing*	*[°]*
UCS_t	*- unconfined compressive strength at any curing time (t)*	*[kPa]*
UCS_{28-day}	*- unconfined compressive strength after 28 days of curing*	*[kPa]*
$S_{u;t;\sigma_3=0}$	*- undrained shear strength at any curing time (t) as determined from unconfined compression tests*	*[kPa]*
$S_{u;28-day;\sigma_3=0}$	*- undrained shear strength after 28 days of curing as determined from unconfined compression tests*	*[kPa]*

When subsequently determining the scaling factor from the development in the unconfined compressive strength as measured during the laboratory research, the scaling factors as presented in table h.1 were obtained. The 14-day scaling factor of the stabilised organic clay was assumed, as a reduction in strength was not considered possible. The scaling factors at all other days up to 28 days of curing were linearly interpolated between measurements.

Table H.1; The scaling factors applied during the implementation analyses as determined from the curing curves.

Stabilised soil type	Scaling factor (α) at curing time [-]					Unit
	1 day	2 days	7 days	14 days	28 days	
Peat	0,514	0,662	0,903	0,953	1,000	[-]
Organic clay	0,618	0,819	0,992	0,996 (assumed)	1,000	[-]

Apart from the measured unconfined compressive strengths, the effective strength parameters were also measured for the examined stabilised soil samples after 28 days of curing. For the implementation analyses, the effective strength parameters of the stabilised organic clay as measured from isotropic consolidation undrained triaxial compression tests (CIUc triaxial tests) were determined at 2% axial strain. On the other hand, the effective strength parameters of the stabilised peat as measured from shearbox tests were determined at 5% shear strain. The obtained values of the effective strength parameters for these stabilised soils are presented in table h.2.

Both the determination of the effective strength parameters at 2% axial strain and 5% shear strain are in line with the Dutch STOWA guideline for the assessment of the safety of regional flood defences (Stichting Toegepast Onderzoek Waterbeheer, 2015) (van Duinen, 2012). However, according to the Deltares laboratory protocol, the strength of peat should be determined using direct simple shear test (DSS test) (Greeuw, van Essen, & van Duinen, 2016). Since there were no DSS setups available to the author during the research, shearbox tests were applied as a substitute to the DSS test instead. The strength of the peat (and for comparison purposes also the stabilised peat) as measured in the shearbox test was subsequently determined at 5% shear strain to match the strength interpretation for DSS tests.

Table H.2; The measured values of the effective strength parameters of the examined stabilised peat and organic clay after 28 days of curing.

Soil parameter	Stabilised peat	Stabilised organic clay	Unit
Effective cohesion (c')	13,94	2,81	[kPa]
Effective angle of internal friction (ϕ')	20,91	40,89	[°]

The measured values of the effective strength parameters were mean values and were required to be converted to design values in order to be allowed to use in the implementation analyses. Due to the lack of additional measurements to allow for proper derivation of the mean and standard deviation of the effective strength parameters, predetermined values of the coefficient of variation of the effective strength parameters were applied. The coefficient of variation of the cohesion of both stabilised soils was selected equal to 0,4. This value is twice the coefficient of variation of the cohesion from table 2.b of Dutch standard NEN 9997-1 (i.e. Eurocode 7 + Dutch national appendix), in which the factor two was applied based on findings in literature on the variability in stabilised soils. However, the coefficient of variation of the tangent of the angle of internal friction of the stabilised soils was assumed equal to the coefficient of variation of the tangent of the angle of internal friction of the untreated soils. These coefficients of variation differed slightly for both soil types and were obtained from the database on soil parameters of soils that were found in the management region of Water Board *Hoogheemraadschap De Stichtse Rijnlanden*. Coefficients of variation of the cohesion of these soils were also available in this database, but these values were either very large (exceeding 3,0) or negative and thus unusable.

Additionally, partial material factors were selected from module C of the guideline for the assessment of the safety of (Dutch) regional flood defences (Stichting Toegepast Onderzoek Waterbeheer, 2015). These partial material factors were selected rather high to include additional safety, since a suitable value of the partial material factor was not known for mass stabilised soils.

The partial material factors and coefficients of variation for the stabilised peat and the stabilised organic clay that were applied in this research are presented in table h.3. These coefficients of variation and partial material factors are equal to those applied earlier in this research to determine the required mean strength of the stabilised soils (see chapter 3 of the main report).

Table H.3; The applied coefficients of variation and partial material factors for the cohesion and the tangent of the internal friction for the examined stabilised soils.

Soil type	$COV_{c'}$	$COV_{tan(\phi')}$	$\gamma_{mat;c'}$	$\gamma_{mat;tan(\phi')}$
	[-]	**[-]**	**[-]**	**[-]**
Stabilised peat	0,4	0,03	1,5	1,2
Stabilised organic clay	0,4	0,09	1,5	1,2

The measured values of the effective strength parameters as presented in table h.2 were subsequently converted to design values using equation (H-3) and the coefficients of variation and partial material factors from table h.3. Equation (H-3) assumes a normal distribution of the effective strength parameters with a t-student factor of 1,64. After conversion of the values presented in table h.2, the design values of the effective strength parameters of both stabilised soils as shown in table h.4 were obtained.

$$X_d = \frac{X_m - 1{,}64 \cdot (COV_X \cdot X_m)}{\gamma_{mat(X)}} \quad \textbf{(H-3)}$$

<u>*where:*</u>

X_d	*- design value of soil parameter*	*[any]*
X_m	*- mean (measured) value of soil parameter*	*[any]*
COV_X	*- coefficient of variation of soil parameter*	*[-]*
$\gamma_{mat(X)}$	*- partial material factor for soil parameter*	*[-]*

Table H.4; The design values of the effective strength parameters of the stabilised soils after 28 days of curing.

Soil parameter	Stabilised peat	Stabilised organic clay	Unit
Effective cohesion (c')	3,20	0,64	[kPa]
Effective angle of internal friction (ϕ')	16,84	31,60	[°]

The obtained design values of the effective cohesion and the tangent of the effective angle of internal friction ($\tan(\phi')$) of both stabilised soils were subsequently scaled using the scaling factors from table h.1. This resulted in an estimation of the design values of the effective cohesion and the tangent of the effective angle of internal friction ($\tan(\phi')$) during curing. The estimation of the effective angle of internal friction in time was obtained by taking the arctangent of the scaled tangent of the effective angle of internal friction ($\tan(\phi')$). The obtained estimation of the design values of the effective cohesion and the effective angle of internal friction for both stabilised soils in time are presented in figure h.1. The numbers corresponding to each curing time in figure h.1 are presented in table h.5.

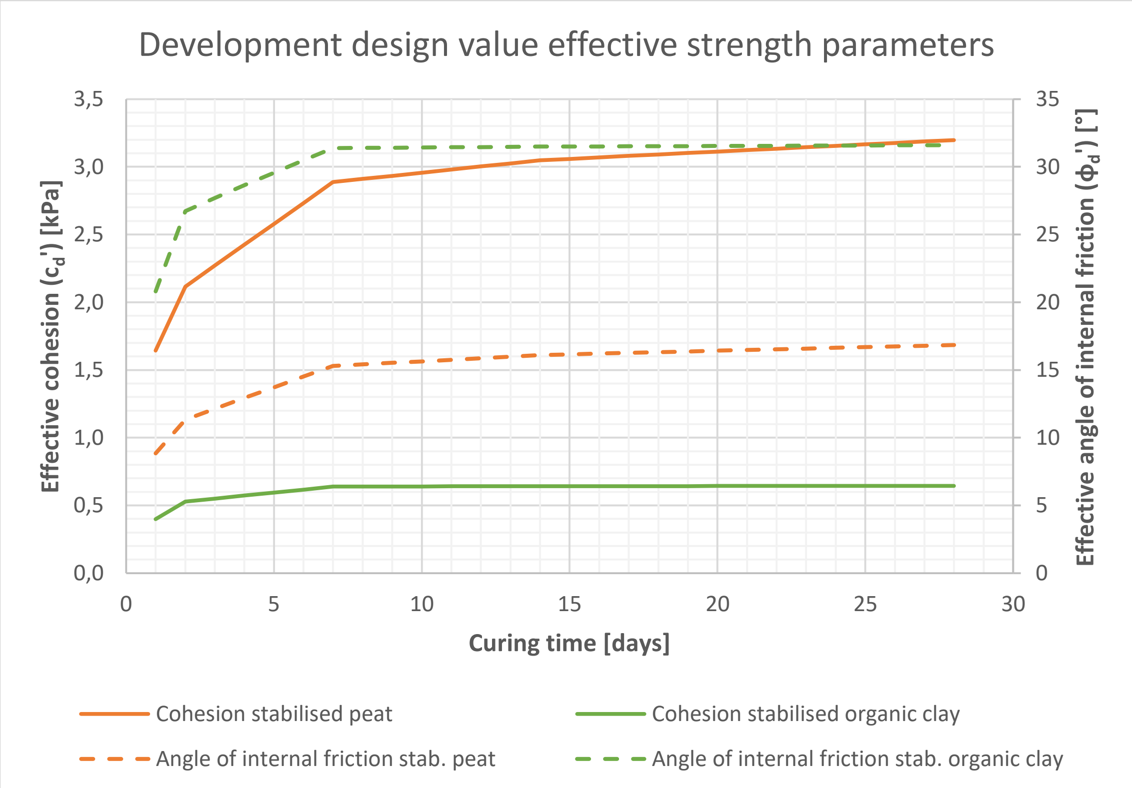


Figure H.1; The applied development of the effective strength parameters in time for the stabilised peat and the stabilised organic clay during the implementation analyses.

Table H.5; Scaling factors and derived design values of the effective strength parameters in time for both the examined stabilised peat and examined stabilised organic clay.

Curing time	**Stabilised peat**			**Stabilised organic clay**		
	Scaling factor	c'_d	ϕ'_d	**Scaling factor**	c'_d	ϕ'_d
[days]	**[-]**	**[kPa]**	**[°]**	**[-]**	**[kPa]**	**[°]**
1	0,514	1,64	8,84	0,618	0,40	20,80
2	0,662	2,12	11,33	0,819	0,53	26,73
3	0,710	2,27	12,13	0,853	0,55	27,69
4	0,758	2,42	12,93	0,888	0,57	28,64
5	0,807	2,58	13,72	0,922	0,59	29,57
6	0,855	2,73	14,51	0,957	0,62	30,48
7	0,903	2,89	15,29	0,991	0,64	31,38
8	0,910	2,91	15,41	0,991	0,64	31,38
9	0,917	2,93	15,52	0,991	0,64	31,38
10	0,925	2,96	15,64	0,991	0,64	31,38
11	0,932	2,98	15,75	0,991	0,64	31,38
12	0,939	3,00	15,87	0,991	0,64	31,38
13	0,946	3,02	15,98	0,991	0,64	31,38
14	0,953	3,05	16,10	0,991	0,64	31,38
15	0,957	3,06	16,15	0,992	0,64	31,39
16	0,960	3,07	16,21	0,993	0,64	31,41
17	0,963	3,08	16,26	0,993	0,64	31,42
18	0,967	3,09	16,31	0,994	0,64	31,44
19	0,970	3,10	16,36	0,995	0,64	31,45
20	0,973	3,11	16,42	0,995	0,64	31,47
21	0,977	3,12	16,47	0,996	0,64	31,49
22	0,980	3,13	16,52	0,996	0,64	31,50
23	0,983	3,14	16,58	0,997	0,64	31,52
24	0,987	3,15	16,63	0,998	0,64	31,53
25	0,990	3,16	16,68	0,998	0,64	31,55
26	0,993	3,18	16,74	0,999	0,64	31,56
27	0,997	3,19	16,79	0,999	0,64	31,58
28	1,000	3,20	16,84	1,000	0,64	31,60

H.1.2 Bulk unit weight

During the laboratory research, stabilised soil samples were prepared by first mixing soil, binder and possibly additional water in a predetermined mass ratio to a homogenous mass. After this, the soil-binder mixture was divided over a number of moulds and compacted to remove air present in the mixture. Subsequently, the mixtures were loaded in order to simulate in-situ stresses, causing the mixtures to compress. After curing for a certain amount of time under the applied load, the mixtures were extruded from the mould and the dimensions and the mass of the extruded samples were recorded. These measurements allowed for a determination of the unit weight of the stabilised soil sample. By combining these measurements with the measured compression and the recorded mass of the mixture prior to loading, the unit weight of the mixture prior to loading was derived. Both the unit weight of the mixture before and after loading were important to record, as they told something about the change in unit weight of the soil due to stabilisation and during curing.

However, in order to determine which values of the unit weight should be used in the implementation analyses, the measured unit weights of the mixtures in the mould before and after loading were analysed. Graphs showing the unit weights of the peat and organic clay samples stabilised with respectively 50 and 75 kg Portland cement per cubic metre undisturbed soil as measured during the laboratory research are presented in figure h.2 and figure h.3 respectively. The unit weight of the stabilised soil samples prior to loading are indicated at zero days of curing. The unit weight of the stabilised soil samples after loading are indicated at the amount of time after which the samples were extruded from the moulds.

It should be noted that for the stabilised peat the unit weights of the samples produced during phase 3 of the laboratory research are included in figure h.2, whereas the unit weight of the stabilised organic clay samples produced during phase 3 of the laboratory research are not included in figure h.3. These unit weights were not included as a result of large differences in recorded compression between the samples. These differences were the result of some weights getting clamped in the moulds, preventing the mixtures from compressing properly. Since it could not be determined with certainty which samples were compressed properly, all measured unit weights of the stabilised organic clay samples after extrusion were deemed unreliable and were therefore not included in figure h.3. This included the unit weight prior to loading, as the recorded compression used to derive this unit weight was unreliable.

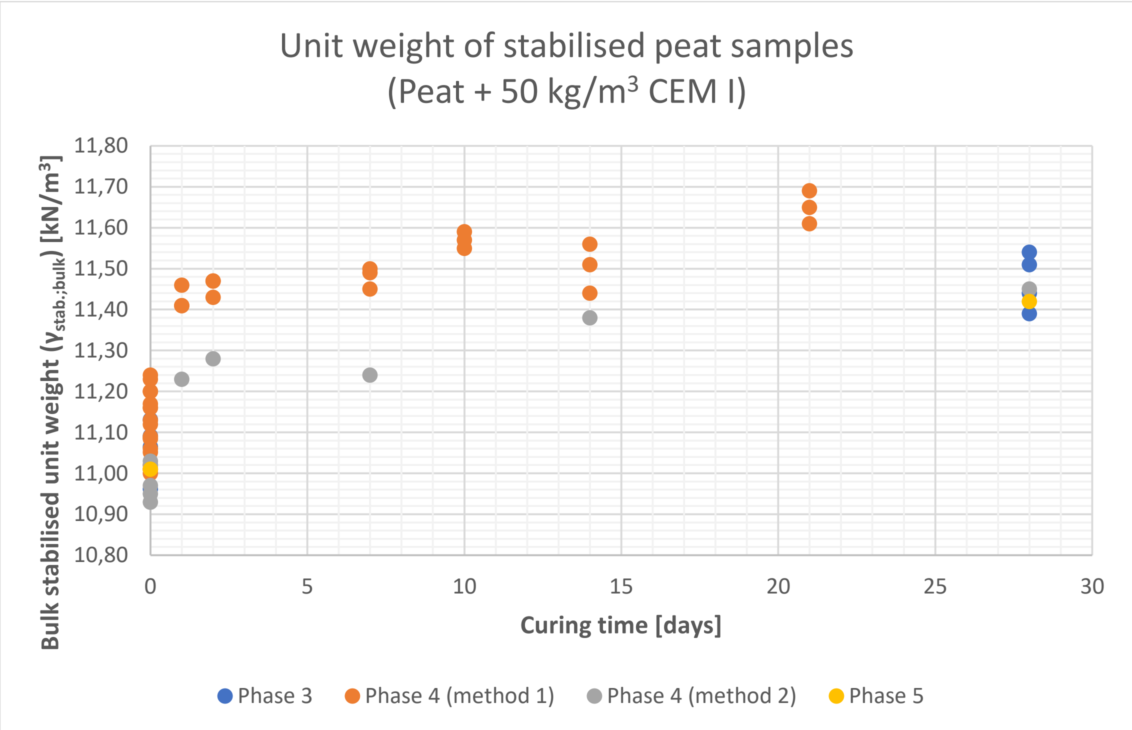


Figure H.2; The measured unit weights of the peat samples stabilised with 50 kg Portland cement / m^3 undisturbed soil prior to loading (curing time = 0 days) and after loading as measured during multiple phases of the laboratory research.

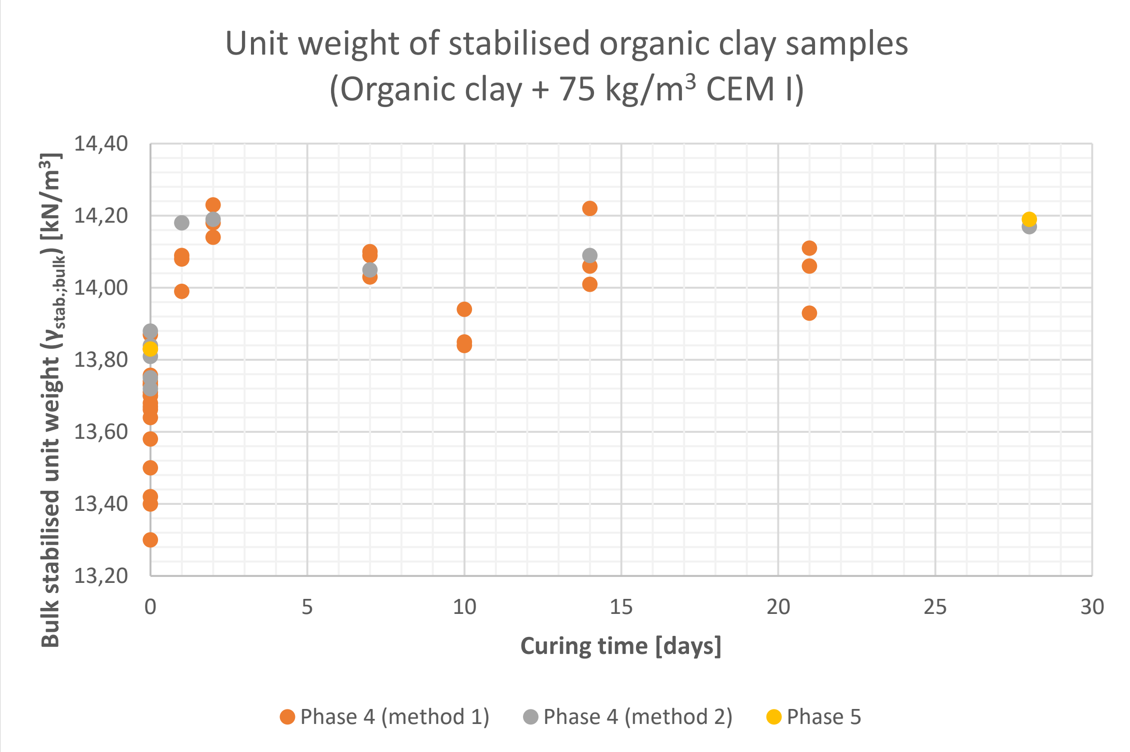


Figure H.3; The measured unit weights of the organic clay samples stabilised with 75 kg Portland cement / m³ undisturbed soil prior to loading (curing time = 0 days) and after loading as measured during multiple phases of the laboratory research.

From both figure h.2 and figure h.3 it can be seen that there are some differences in both the measured unit weights before and after loading. The observed differences in the measured unit weights before loading are the result of differences in compaction during the filling of the moulds. The observed differences in the measured unit weights after loading are the result of differences in compression between different batches of the same mixture. These differences in compression are caused by minor differences in the applied load or by differences in composition between different batches of soil-binder mixture, possibly caused by compositional differences in the undisturbed soil samples used for stabilisation due to soil heterogeneity or by minor differences in the applied mixing procedure.

During method 2 of phase 4 (see the grey dots in figure h.2 and figure h.3), one batch of stabilised peat and one batch of stabilised organic clay were produced. Each sample produced from those batches was left to cure under the same load for a different amount of time. Since each sample was produced from the same batch, differences in results due to compositional differences in the used soil samples or differences in the applied mixing procedure are prevented. When examining the unit weights after compression recorded for these samples, it can be seen that the unit weight of the examined stabilised peat samples seemed to increase with increasing curing time. An exception seemed to have been measured at 2 days of curing, in which a reduction in the unit weight was recorded. Such a trend was not recorded for the unit weights of the examined stabilised organic clay samples. For the stabilised organic clay samples the unit weight seemed constant between 1 and 2 days of curing, followed by a reduction to 7 days of curing which was followed by an increase in the unit weight between 7 and 28 days of curing. For both the examined stabilised peat and stabilised organic clay samples during method 2 of phase 4, the difference in the lowest and largest recorded unit weight is about 0,15 kN/m³.

Despite the variations in the recorded unit weight of both the stabilised peat and stabilised organic clay samples during method 2 of phase 4, it was not expected to measure these variations in the unit weight. During the loading of the mixtures, the compression of the mixtures in the moulds was measured. The recorded compression of both the examined stabilised peat and stabilised organic clay samples are presented in figure h.4 and figure h.5 respectively.

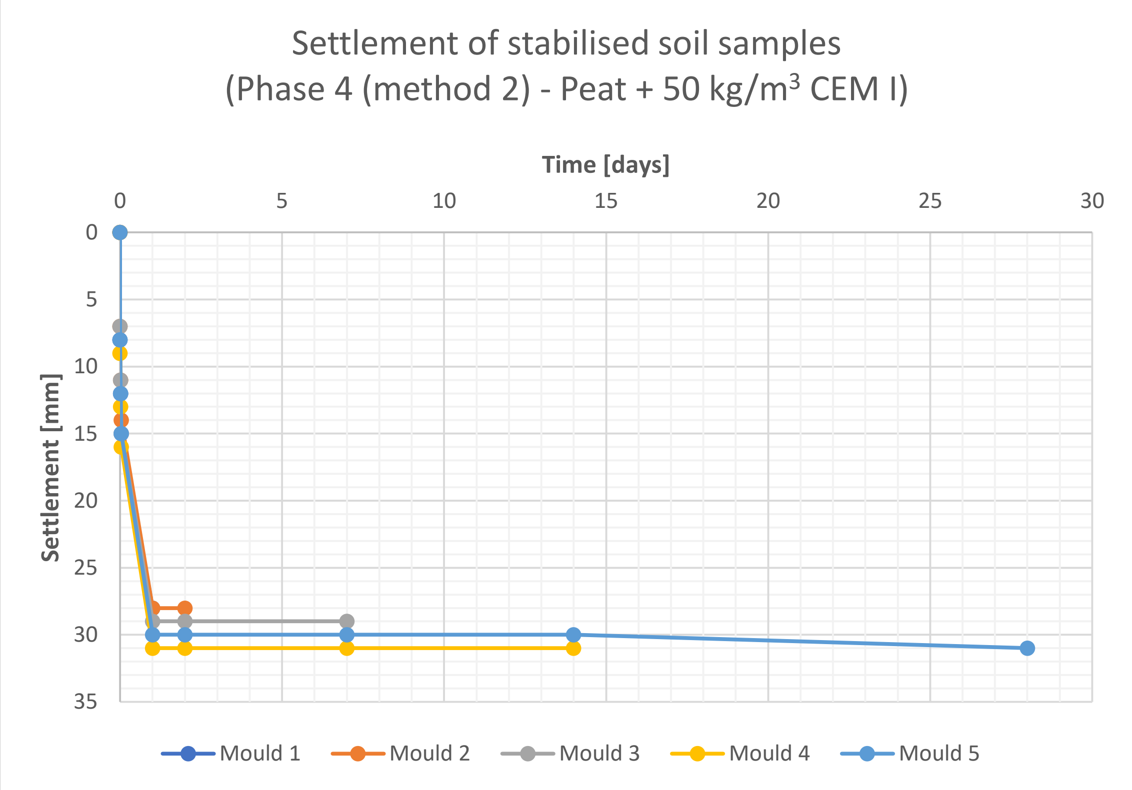


Figure H.4; The measured settlement of the peat samples stabilised with 50 kg/m³ of Portland cement (CEM I) in phase 4 (method 2) of the laboratory research. These samples were left to cure for 24 hours, 48 hours, 7 days, 14 days and 28 days respectively.

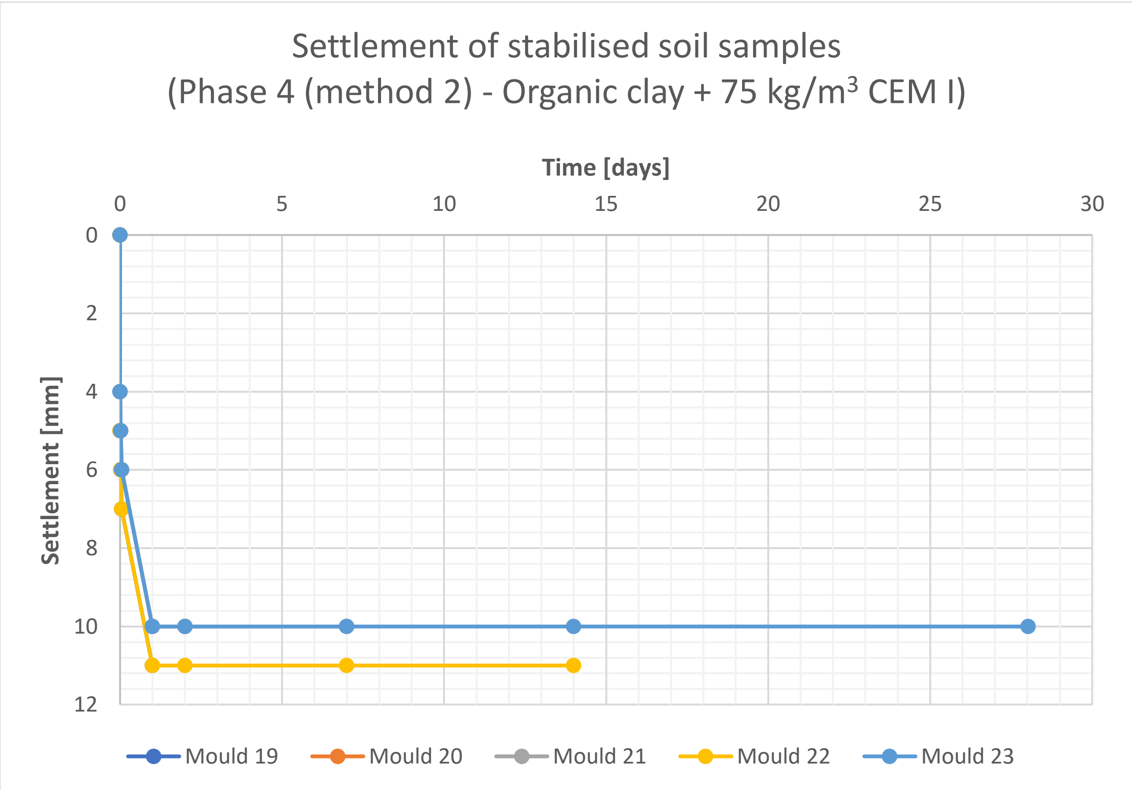


Figure H.5; The measured settlement of the organic clay samples stabilised with 75 kg/m³ of Portland cement (CEM I) in phase 4 (method 2) of the laboratory research. These samples were left to cure for 24 hours, 48 hours, 7 days, 14 days and 28 days respectively. Mould 19 and mould 22 show similar results. Mould 20, 21 and 23 also show similar results.

Based on the measurements of the compression, the compression of all stabilised peat and organic clay samples had ended after 24 hours of curing under load. Physically this means that the unit weight of the samples should be changing up to 24 hours of curing under load after which the unit weight should not change anymore. As such, it is physically not logical to have measured the recorded variations in the unit weight with increasing curing time after at least 24 hours of curing under load. Yet differences in the unit weight of up to 0,15 kN/m^3 were recorded for both the examined stabilised peat and stabilised organic clay samples. These differences of at most 0,15 kN/m^3 are not large and could have been recorded due to any or a combination of the reasons listed below:

- Measuring inaccuracy in the compression and the dimensions of the extruded stabilised soil samples as a result of manual readings on a millimetre scale measuring tape and ruler;
- Differences in the recorded compression between samples from the same batch, possibly due to slight variations in the applied load or minor compositional differences between samples due to insufficient mixing time;
- Possible expansion and subsequent shrinkage of the samples in time as a result of heat generated by hydration reactions between the Portland cement particles and the pore water during curing.

It was also thought that the differences in unit weight could have been explained by the samples taking up water during curing, explaining why for example an increasing unit weight of the stabilised peat samples was recorded. However, this did likely not occur. The measured water content of the extruded stabilised peat and stabilised organic clay samples as presented in table h.6 both showed an almost constant water content with time. If the samples had indeed taken up water, then a significant increase in the water content should have been measured in time.

Table H.6; Water content of the extruded stabilised soil samples during method 2 of phase 4.

Mixture	**Water content of the stabilised soil sample (w_{stab} [%]) after specified curing time**				
	24 hours	**48 hours**	**7 days**	**14 days**	**28 days**
Peat + 50 kg/m^3 CEM I	264	262	262	261	262
Organic clay + 75 kg/m^3 CEM I	91,0	92,8	92,8	92,1	92,6

Based on the compression measurements and the expected constant unit weight after 24 hours of curing under the applied load, only two different unit weights in time were considered:

- The unit weight of the stabilised soil directly after stabilisation;
- The unit weight of the stabilised soil after 24 hours of curing.

Using all unit weights as shown in figure h.2 and figure h.3, the average and the standard deviation of the unit weight of both the stabilised peat and organic clay samples prior to and after 24 hours of curing under load were derived. The average and the standard deviation of the unit weight of the samples prior to loading were determined using only the measured values of the unit weight at 0 days of curing as shown in figure h.2 and figure h.3. The average and the standard deviation of the unit weight of the samples after 24 hours of curing under load and onward were determined using all measured values of the unit weight between 24 hours and 28 days of curing as shown in figure h.2 and figure h.3.

Using equation (H-4) and the mean and standard deviation as presented in table h.7, the 5% characteristic value of the unit weight of both the stabilised peat and the stabilised organic clay samples before and after loading were determined. Here the 5% characteristic value is defined as the unit weight value at which the probability that a lower value of the unit weight will be found is 5%.

$$X_k = X_m - 1{,}64 \cdot \sigma_X \qquad \textbf{(H-4)}$$

where:

X_k	*- characteristic value of soil parameter*	*[any]*
X_m	*- mean (measured) value of soil parameter*	*[any]*
σ_X	*- standard deviation of soil parameter*	*[any]*

Table H.7; The mean and standard deviation of the bulk unit weight of the examined stabilised peat and stabilised organic clay samples.

Mixture	Stabilised bulk unit weight prior to loading (0 curing time)		Stabilised bulk unit weight after loading (24+ hours curing time)	
	Mean value (μ) [kN/m³]	Standard deviation (σ) [kN/m³]	Mean value (μ) [kN/m³]	Standard deviation (σ) [kN/m³]
Peat + 50 kg/m³ Portland cement	11,08	0,087	11,47	0,110
Organic clay + 75 kg/m³ Portland cement	13,69	0,151	14,08	0,108

In the implementation analyses design values of the soil parameters were required. Therefore the 5% characteristic values were required to be converted to design values. In accordance with the guideline for the assessment of the safety of (Dutch) regional flood defences, a partial material factor of 1,0 was to be applied on the unit weight (Stichting Toegepast Onderzoek Waterbeheer, 2015). However, since the stabilised soils are not a naturally occurring soil type, this partial material factor based solely on that guideline cannot be applied. However, the EuroSoilStab design manual and Dutch CUR 2001-10 reported that the design value and characteristic value of the unit weight of stabilised soils are regarded equal, implying that the partial material factor on the unit weight equals 1,0 (see section 2.4.2). As such, there was sufficient ground to apply a partial material of 1,0 on the unit weight of the stabilised soil samples before and after loading. The obtained design values for the unit weight before and after loading for both the stabilised peat and the stabilised organic clay using a partial material factor of 1,0 are presented in table h.8.

Table H.8; The 5% characteristic value and the corresponding design value of the bulk unit weight of the examined stabilised peat and stabilised organic clay samples before and after loading.

Mixture	Stabilised bulk unit weight prior to loading (0 days curing time)		Stabilised bulk unit weight after loading (24+ hours curing time)	
	5% characteristic value [kN/m³]	Design value [kN/m³]	5% characteristic value [kN/m³]	Design value [kN/m³]
Peat + 50 kg/m³ Portland cement	10,94	10,94	11,29	11,29
Organic clay + 75 kg/m³ Portland cement	13,44	13,44	13,90	13,90

H.2 Scenario results Montfoortse Vaart

In this section the complete results from the implementation analyses are presented per examined scenario.

H.2.1 Scenario 1

In scenario 1, the implementation stability analyses were carried out with the assumptions on the change in the unit weight due to stabilisation and the initial strength of the stabilised soil directly after mixing as listed in table h.9. The design values of the parameters as applied during the implementation stability analyses based on the assumptions of table h.9 are presented in table h.10.

Table H.9; The assumptions on the change in the unit weight and the strength of the stabilised soil directly after mixing as applied in scenario 1.

Parameter	Scenario 1
Unit weight	Increase
Initial shear strength after mixing	Zero strength

Table H.10; The design value of the unit weight of the stabilised soil after mixing and the design values of the effective strength parameters of the stabilised soil directly after mixing as applied in scenario 1.

Soil parameter	Mixture		Unit
	Stabilised peat	Stabilised organic clay	
$\gamma_{stab.;bulk;d}$	10,94	13,44	[kN/m^3]
$\gamma_{stab.;sat;d}$	10,94	13,44	[kN/m^3]
c'_d **after mixing**	0,00	0,00	[kPa]
ϕ'_d **after mixing**	0,00	0,00	[°]

The assumptions on the speed of stabilisation and the speed of applying or removing preload as applied during scenario 1 are presented in table h.11. The obtained Factors of Safety at each considered step during the implementation examined in scenario 1 are presented in table h.12. The Factors of Safety at all time steps considered during the implementation examined in scenario 1 are presented graphically in figure h.6. A visual representation of the order of stabilisation at the 25 metres of levee at the Montfoortse Vaart as examined in scenario 1 is presented in figure 5.4. The critical slip surfaces as determined with both the Bishop and Uplift Van calculation model at the end of each major step during the implementation analyses of scenario 1 are presented in figure h.8 through figure h.14.

Table H.11; The assumptions on the speed of stabilisation and the speed of applying or removing preload as applied in scenario 1.

Parameter	Value	Unit
Speed of stabilisation	100	[m^3/h]
Speed of applying or removing 8,0 kPa preload	3	[min/m^2]

Table H.12; The results of the implementation analyses of scenario 1 at each considered step of the implementation.

Implementation analyses results scenario 1						
Time	**Action**	**Stab. soil vol.**	**Load area**	**Stab. length**	**FoS (Bishop)**	**FoS (Uplift Van)**
[days]	**[-]**	**[m^3]**	**[m^2]**	**[m]**	**[-]**	**[-]**
0,00	Normal conditions	-	-	-	0,94	0,93
0,01	Stabilisation block 1	25	-	1	0,92	0,91
0,02	Preloading block 1	-	5	-	0,94	0,93
0,03	Stabilisation broadening block 1	25	-	1	0,92	0,91
0,04	Preloading broadened block 1	-	5	-	0,94	0,93
0,05	Stabilisation broadening block 1 (part 2)	25	-	1	0,93	0,91

0,06	Preloading broadened block 1 (part 2)	-	5	-	0,94	0,93
0,07	Stabilisation broadening block 1 (part 3)	25	-	1	0,93	0,92
0,08	Preloading broadened block 1 (part 3)	-	5	-	0,95	0,94
0,09	Stabilisation broadening block 1 (part 4)	25	-	1	0,94	0,92
0,10	Preloading broadened block 1 (part 4)	-	5	-	0,95	0,94
0,11	Stabilisation broadening block 1 (part 5)	25	-	1	0,94	0,92
0,13	Preloading broadened block 1 (part 5)	-	5	-	0,96	0,94
0,14	Stabilisation broadening block 1 (part 6)	25	-	1	0,94	0,92
0,15	Preloading broadened block 1 (part 6)	-	5	-	0,96	0,94
0,16	Stabilisation broadening block 1 (part 7)	25	-	1	0,94	0,92
0,17	Preloading broadened block 1 (part 7)	-	5	-	0,96	0,94
0,18	Stabilisation broadening block 1 (part 8)	25	-	1	0,94	0,92
0,19	Preloading broadened block 1 (part 8)	-	5	-	0,96	0,94
0,20	Stabilisation broadening block 1 (part 9)	25	-	1	0,94	0,94
0,21	Preloading broadened block 1 (part 9)	-	5	-	0,96	0,96
0,22	Stabilisation broadening block 1 (part 10)	25	-	1	0,94	0,97
0,23	Preloading broadened block 1 (part 10)	-	5	-	0,96	1,00
0,24	Stabilisation broadening block 1 (part 11)	25	-	1	0,94	1,03
0,25	Preloading broadened block 1 (part 11)	-	5	-	0,96	1,04
0,26	Stabilisation broadening block 1 (part 12)	25	-	1	0,94	1,03
0,27	Preloading broadened block 1 (part 12)	-	5	-	0,96	1,02
0,28	Stabilisation broadening block 1 (part 13)	25	-	1	0,94	1,01
0,29	Preloading broadened block 1 (part 13)	-	5	-	0,96	1,02
0,30	Stabilisation broadening block 1 (part 14)	25	-	1	0,93	1,01
0,31	Preloading broadened block 1 (part 14)	-	5	-	0,95	1,01
0,32	Stabilisation broadening block 1 (part 15)	25	-	1	0,93	1,00
0,33	Preloading broadened block 1 (part 15)	-	5	-	0,95	1,00
0,34	Stabilisation broadening block 1 (part 16)	25	-	1	0,93	0,99
0,35	Preloading broadened block 1 (part 16)	-	5	-	0,94	0,99
0,36	Stabilisation broadening block 1 (part 17)	25	-	1	0,92	0,98
0,38	Preloading broadened block 1 (part 17)	-	5	-	0,94	0,98
1,38	Curing 24 hours	-	-	-	1,24	1,14
1,39	Stabilisation block 2	25	-	1	1,22	1,13
1,40	Preloading block 2	-	5	-	1,23	1,13
1,41	Stabilisation broadening block 2	25	-	1	1,22	1,12
1,42	Preloading broadened block 2	-	5	-	1,22	1,12
1,43	Stabilisation broadening block 2 (part 2)	25	-	1	1,21	1,11
1,44	Preloading broadened block 2 (part 2)	-	5	-	1,21	1,11
1,45	Stabilisation broadening block 2 (part 3)	25	-	1	1,20	1,10
1,46	Preloading broadened block 2 (part 3)	-	5	-	1,20	1,10
1,47	Stabilisation broadening block 2 (part 4)	25	-	1	1,19	1,10
1,48	Preloading broadened block 2 (part 4)	-	5	-	1,19	1,10
1,49	Stabilisation broadening block 2 (part 5)	25	-	1	1,18	1,09
1,50	Preloading broadened block 2 (part 5)	-	5	-	1,19	1,09
2,50	Curing 24 hours	-	-	-	1,31	1,19
3,50	Curing 24 hours	-	-	-	1,35	1,22
4,50	Curing 24 hours	-	-	-	1,38	1,24
5,50	Curing 24 hours	-	-	-	1,40	1,25
5,75	Removing all preload	-	120	-	1,12	1,11
5,75	Test high water conditions	-	-	-	1,08	1,07

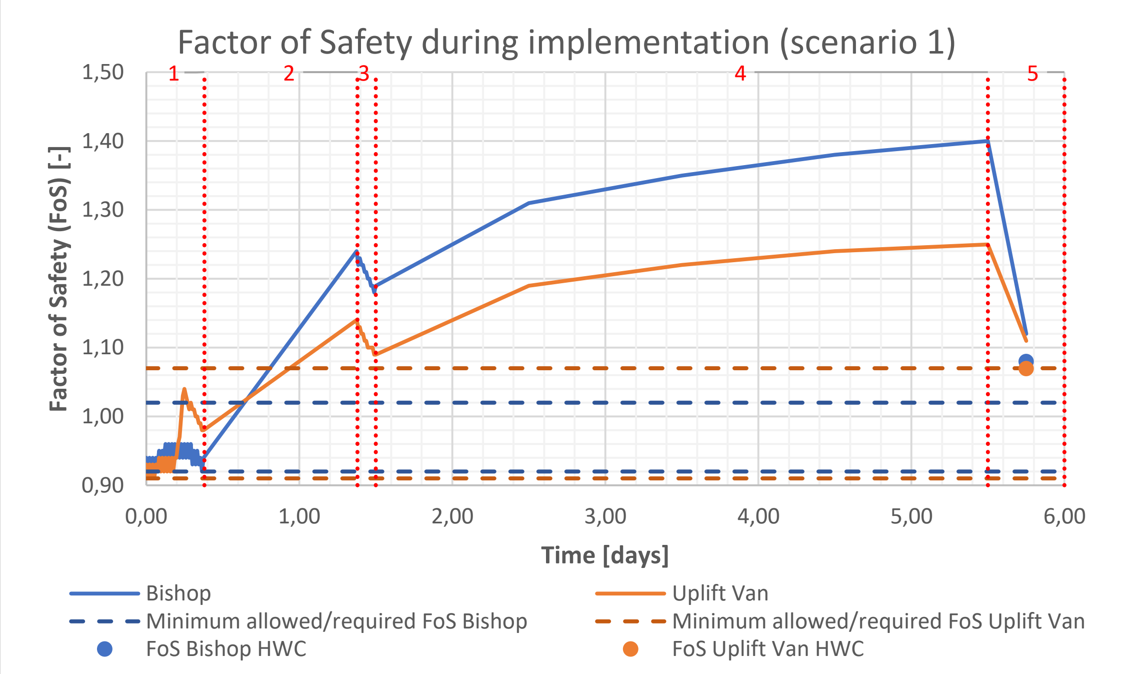


Figure H.6; The development of the Factor of Safety during the implementation of mass stabilisation at the toe of the levee with the assumptions of scenario 1. The red lines represent different actions taken during the implementation, the description of which is presented in table h.13.

Table H.13; Actions taken during the implementation of scenario 1.

Line number figure h.6	Action
1	Stabilisation first 18 metres of soil
2	24 hours of curing of all blocks of stabilised soil
3	Stabilisation last 6 metres of soil
4	96 hours of curing (i.e. 4 days) of all blocks of stabilised soil
5	Removing all preload

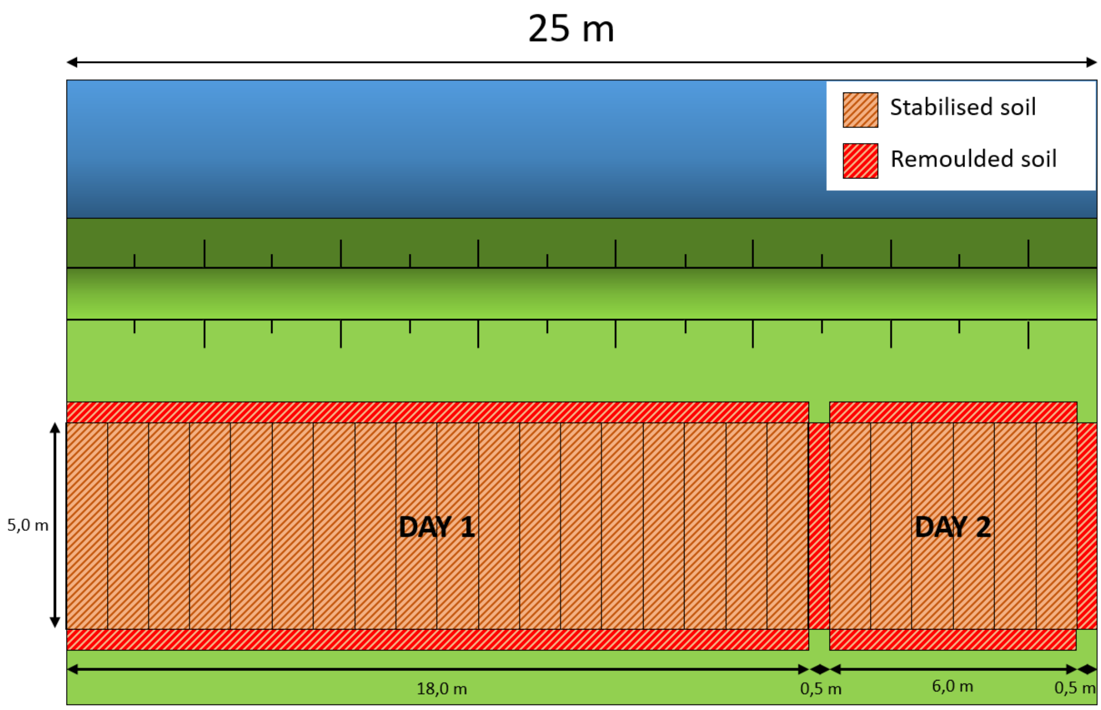


Figure H.7; Top view of the levee showing the obtained implementation for scenario 1.

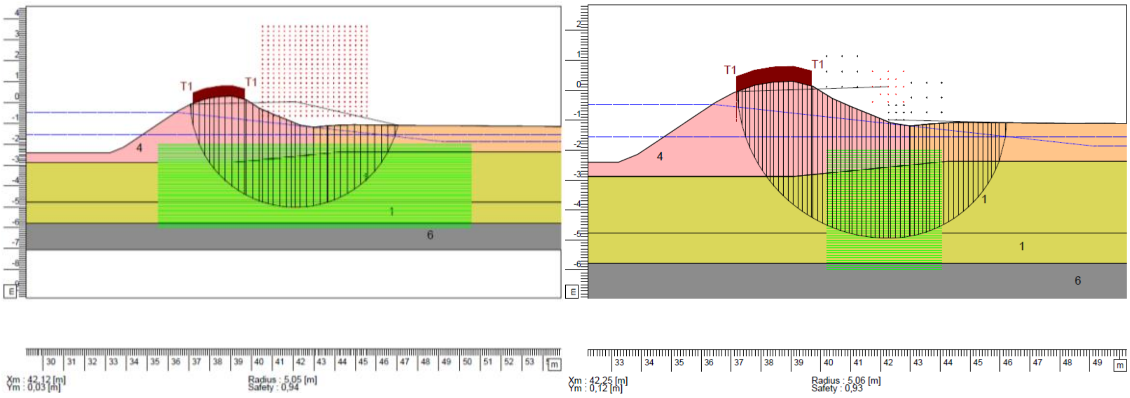


Figure H.8; Critical slip surface in the initial situation at normal water conditions as calculated with the Bishop (left image) and Uplift Van (right image) calculation model.

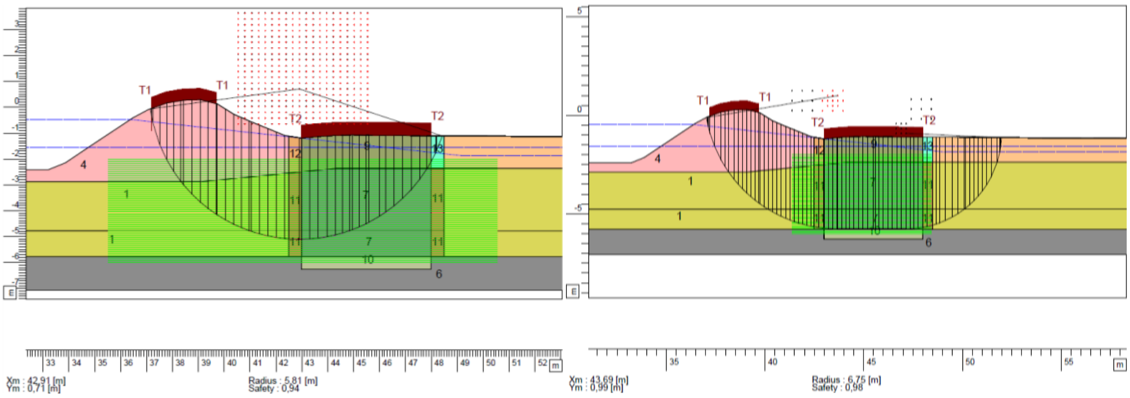


Figure H.9; Critical slip surface at normal water conditions as calculated with the Bishop (left image) and Uplift Van (right image) calculation model after the first section was stabilised.

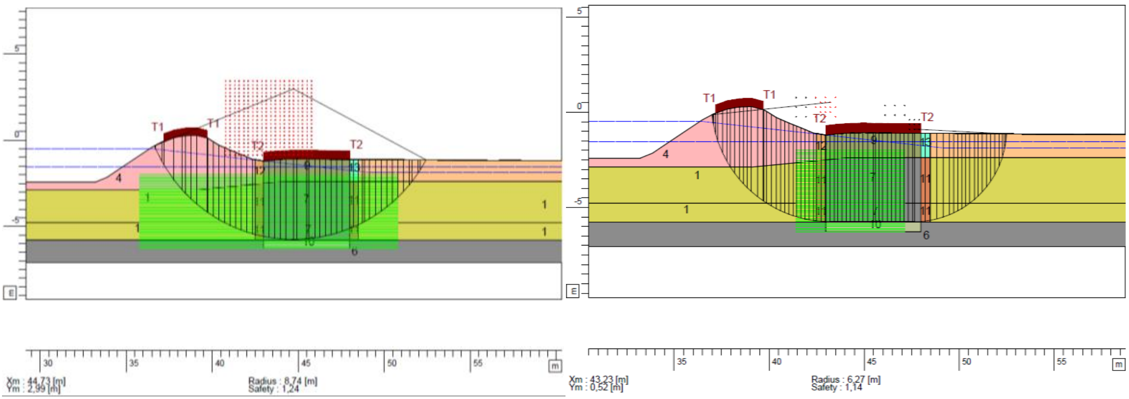


Figure H.10; Critical slip surface at normal water conditions as calculated with the Bishop (left image) and Uplift Van (right image) calculation model after 24 hours of curing after section 1 was stabilised.

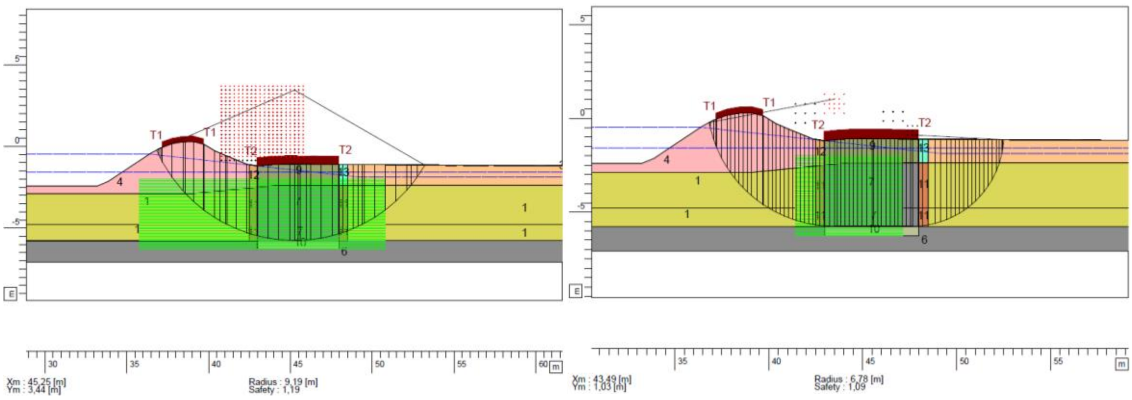


Figure H.11; Critical slip surface at normal water conditions as calculated with the Bishop (left image) and Uplift Van (right image) calculation model after section 2 was stabilised.

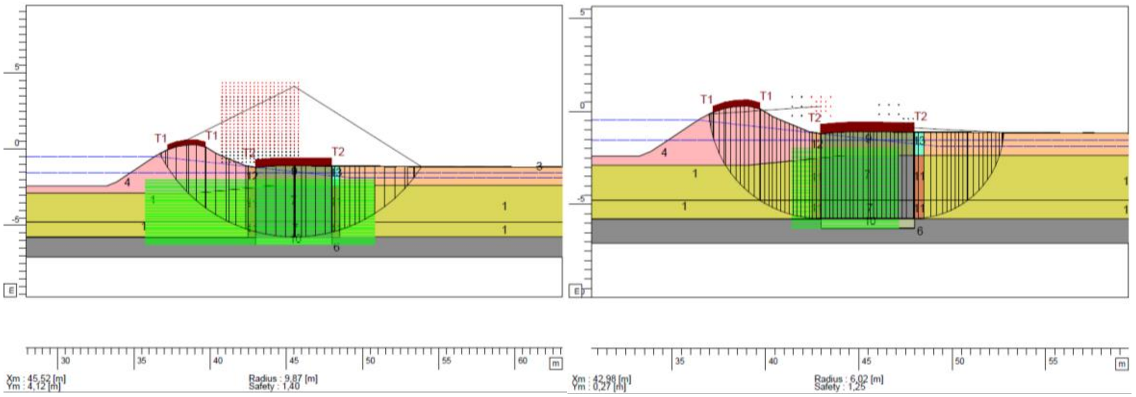


Figure H.12; Critical slip surface at normal water conditions as calculated with the Bishop (left image) and Uplift Van (right image) calculation model after 96 hours of curing after section 2 was stabilised.

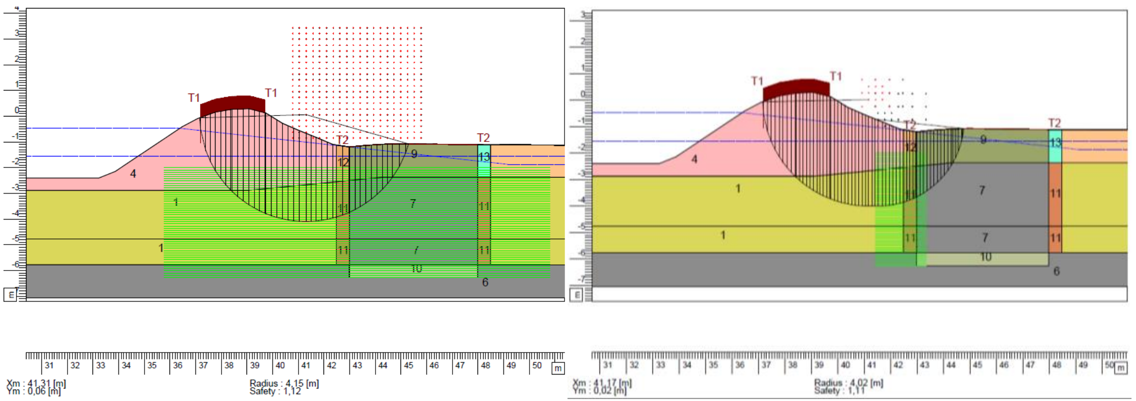


Figure H.13; Critical slip surface at normal water conditions as calculated with the Bishop (left image) and Uplift Van (right image) calculation model after removing all preload.

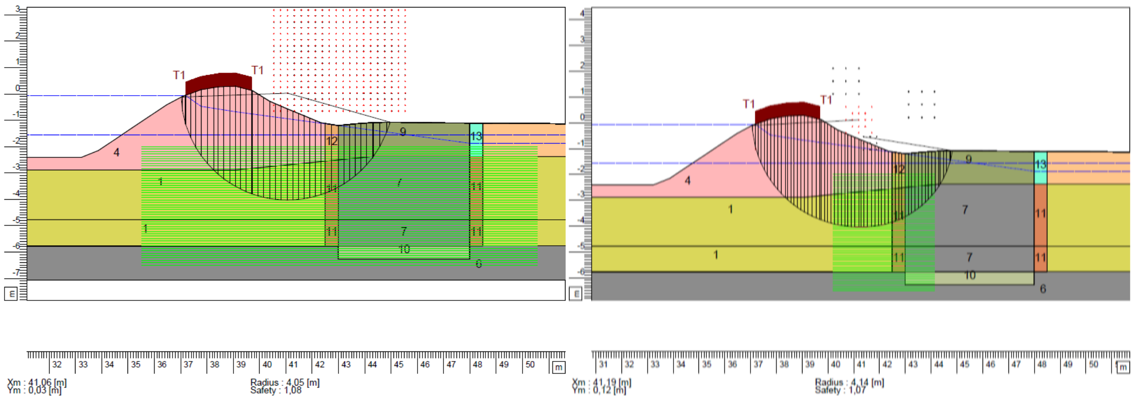


Figure H.14; Critical slip surface in the final situation at high water conditions as calculated with the Bishop (left image) and Uplift Van (right image) calculation model.

H.2.2 Scenario 2

In scenario 2, the implementation stability analyses were carried out with the assumptions on the change in the unit weight due to stabilisation and the initial strength of the stabilised soil directly after mixing as listed in table h.14. The design values of the parameters as applied during the implementation stability analyses based on the assumptions of table h.14 are presented in table h.15.

Table H.14; The assumptions on the change in the unit weight and the strength of the stabilised soil directly after mixing as applied in scenario 2.

Parameter	Scenario 2
Unit weight	Increase
Initial shear strength after mixing	Reduced strength (remoulded)

Table H.15; The design value of the unit weight of the stabilised soil after mixing and the design values of the effective strength parameters of the stabilised soil directly after mixing as applied in scenario 2.

Soil parameter	Mixture		Unit
	Stabilised peat	Stabilised organic clay	
$\gamma_{stab.;bulk;d}$	10,94	13,44	[kN/m^3]
$\gamma_{stab.;sat;d}$	10,94	13,44	[kN/m^3]
c'_d after mixing	0,27	0,33	[kPa]
ϕ'_d after mixing	5,15	13,60	[°]

The assumptions on the speed of stabilisation and the speed of applying or removing preload as applied during scenario 2 are presented in table h.16. The obtained Factors of Safety at each considered step during the implementation examined in scenario 2 are presented in table h.17. The Factors of Safety at all time steps considered during the implementation examined in scenario 2 are presented graphically in figure h.15. A visual representation of the order of stabilisation at the 25 metres of levee at the Montfoortse Vaart as examined in scenario 2 is presented in figure h.16. The critical slip surfaces as determined with both the Bishop and Uplift Van calculation model at the end of each major step during the implementation analyses of scenario 2 are presented in figure h.17 through figure h.21.

Table H.16; The assumptions on the speed of stabilisation and the speed of applying or removing preload as applied in scenario 2.

Parameter	Value	Unit
Speed of stabilisation	100	[m^3/h]
Speed of applying or removing 8,0 kPa preload	3	[min/m^2]

Table H.17; The results of the implementation analyses of scenario 2 at each considered step of the implementation.

Implementation analyses results scenario 2						
Time	**Action**	**Stab. soil vol.**	**Load area**	**Stab. length**	**FoS (Bishop)**	**FoS (Uplift Van)**
[days]	**[-]**	**[m^3]**	**[m^2]**	**[m]**	**[-]**	**[-]**
0,00	Normal conditions	-	-	-	0,94	0,93
0,01	Stabilisation block 1	25	-	1	0,93	0,91
0,02	Preloading block 1	-	5	-	0,94	0,93
0,03	Stabilisation broadening block 1	25	-	1	0,94	0,92
0,04	Preloading broadened broadening block 1	-	5	-	0,95	0,94
0,05	Stabilisation broadening block 1 (part 2)	25	-	1	0,95	0,93
0,06	Preloading broadened block 1 (part 2)	-	5	-	0,97	0,95
0,07	Stabilisation broadening block 1 (part 3)	25	-	1	0,96	0,94
0,08	Preloading broadened block 1 (part 3)	-	5	-	0,98	0,96
0,09	Stabilisation broadening block 1 (part 4)	25	-	1	0,97	0,95
0,10	Preloading broadened block 1 (part 4)	-	5	-	0,99	0,97
0,11	Stabilisation broadening block 1 (part 5)	25	-	1	0,98	0,96
0,13	Preloading broadened block 1 (part 5)	-	5	-	1,00	0,98
0,14	Stabilisation broadening block 1 (part 6)	25	-	1	1,00	0,97
0,15	Preloading broadened block 1 (part 6)	-	5	-	1,02	0,99
0,16	Stabilisation broadening block 1 (part 7)	25	-	1	1,01	0,98
0,17	Preloading broadened block 1 (part 7)	-	5	-	1,03	1,03
0,18	Stabilisation broadening block 1 (part 8)	25	-	1	1,02	1,02
0,19	Preloading broadened block 1 (part 8)	-	5	-	1,03	1,03
0,20	Stabilisation broadening block 1 (part 9)	25	-	1	1,02	1,02
0,21	Preloading broadened block 1 (part 9)	-	5	-	1,04	1,03
0,22	Stabilisation broadening block 1 (part 10)	25	-	1	1,03	1,03
0,23	Preloading broadened block 1 (part 10)	-	5	-	1,05	1,03

0,24	Stabilisation broadening block 1 (part 11)	25	-	1	1,04	1,02
0,25	Preloading broadened block 1 (part 11)	-	5	-	1,05	1,07
0,26	Stabilisation broadening block 1 (part 12)	25	-	1	1,04	1,06
0,27	Preloading broadened block 1 (part 12)	-	5	-	1,06	1,07
0,28	Stabilisation broadening block 1 (part 13)	25	-	1	1,05	1,06
0,29	Preloading broadened block 1 (part 13)	-	5	-	1,07	1,06
0,30	Stabilisation broadening block 1 (part 14)	25	-	1	1,05	1,06
0,31	Preloading broadened block 1 (part 14)	-	5	-	1,07	1,09
0,32	Stabilisation broadening block 1 (part 15)	25	-	1	1,06	1,08
0,33	Preloading broadened block 1 (part 15)	-	5	-	1,08	1,09
0,34	Stabilisation broadening block 1 (part 16)	25	-	1	1,07	1,08
0,35	Preloading broadened block 1 (part 16)	-	5	-	1,08	1,08
0,36	Stabilisation broadening block 1 (part 17)	25	-	1	1,07	1,08
0,38	Preloading broadened block 1 (part 17)	-	5	-	1,09	1,08
0,39	Stabilisation broadening block 1 (part 18)	25	-	1	1,08	1,07
0,40	Preloading broadened block 1 (part 18)	-	5	-	1,10	1,08
0,41	Stabilisation broadening block 1 (part 19)	25	-	1	1,08	1,07
0,42	Preloading broadened block 1 (part 19)	-	5	-	1,10	1,07
0,43	Stabilisation broadening block 1 (part 20)	25	-	1	1,09	1,07
0,44	Preloading broadened block 1 (part 20)	-	5	-	1,10	1,07
0,45	Stabilisation broadening block 1 (part 21)	25	-	1	1,10	1,07
0,46	Preloading broadened block 1 (part 21)	-	5	-	1,10	1,06
0,47	Stabilisation broadening block 1 (part 22)	25	-	1	1,09	1,06
0,48	Preloading broadened block 1 (part 22)	-	5	-	1,10	1,06
0,49	Stabilisation broadening block 1 (part 23)	25	-	1	1,09	1,05
0,50	Preloading broadened block 1 (part 23)	-	5	-	1,09	1,05
0,51	Stabilisation broadening block 1 (part 24)	12,5	-	0,5	1,09	1,05
0,51	Preloading broadened block 1 (part 24)	-	2,5	-	1,09	1,05
1,51	Curing 24 hours	-	-	-	1,26	1,15
2,51	Curing 48 hours	-	-	-	1,34	1,21
3,51	Curing 72 hours	-	-	-	1,36	1,23
4,51	Curing 96 hours	-	-	-	1,39	1,25
4,77	Removing preload	-	122,5	-	1,11	1,11
4,77	Test high water conditions	-	-	-	1,07	1,07

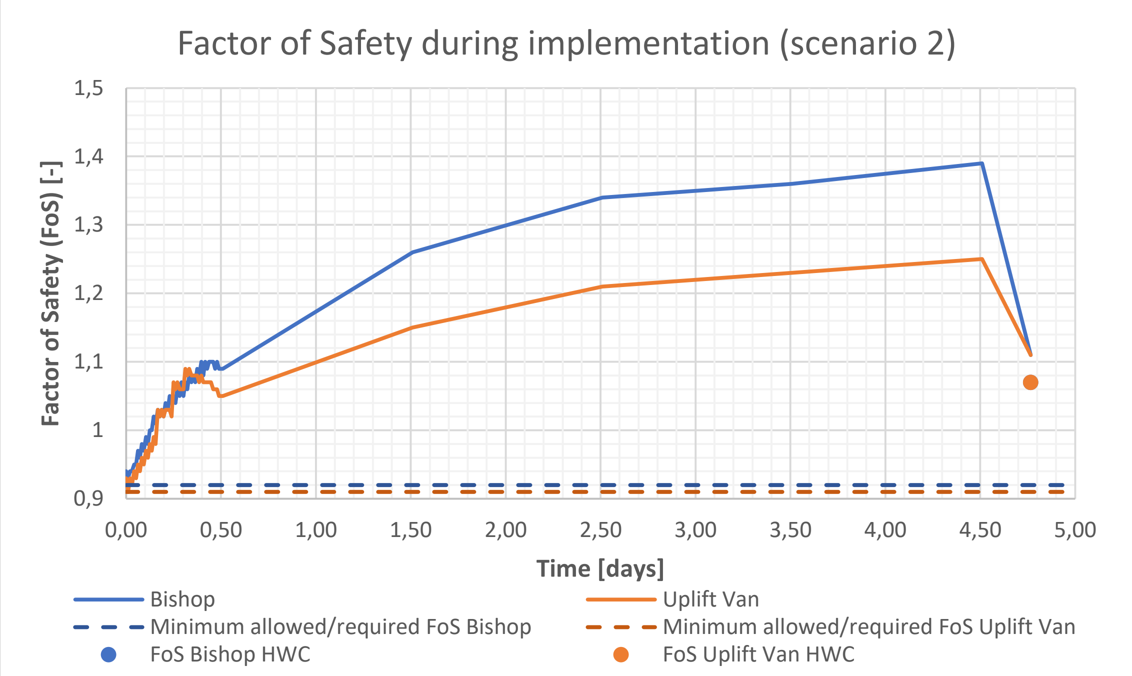


Figure H.15; The development of the Factor of Safety during the implementation of mass stabilisation at the toe of the levee with the assumptions of scenario 2. The red lines represent different actions taken during the implementation, the description of which is presented in table h.18.

Table H.18; Actions taken during the implementation of scenario 2.

Line number figure h.15	Action
1	Stabilisation 24,5 metres of soil
2	96 hours of curing (i.e. 4 days) of all blocks of stabilised soil
3	Removing all preload

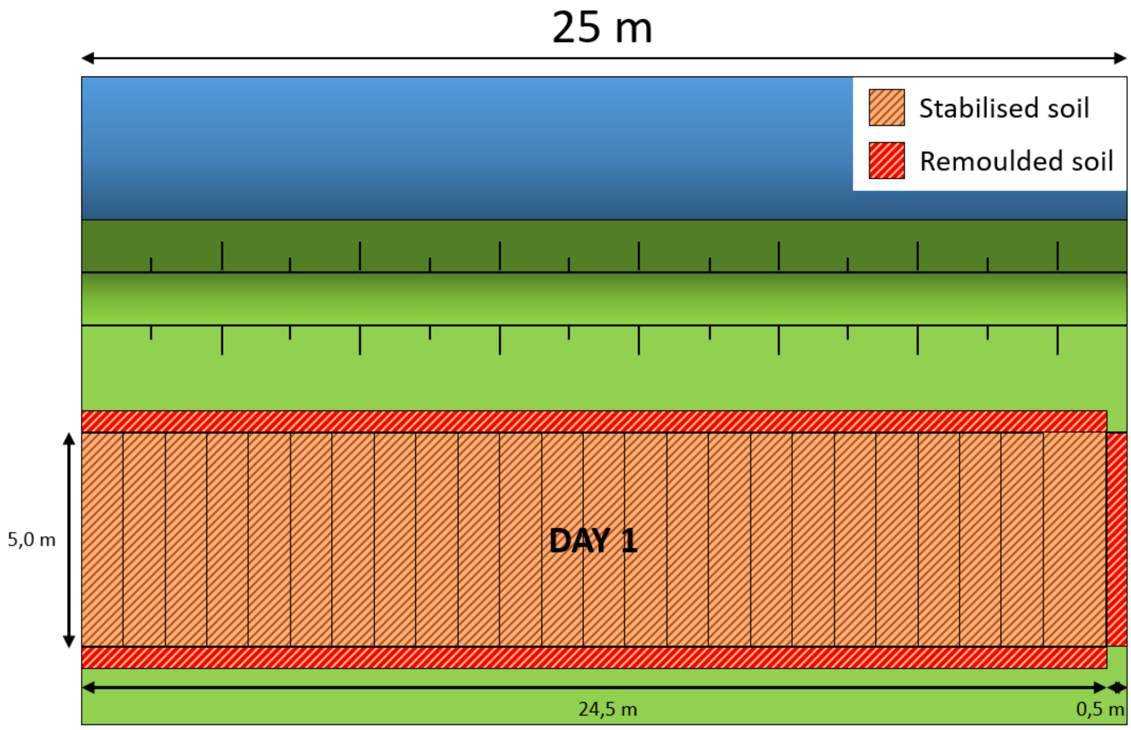


Figure H.16; Top view of the levee showing the obtained implementation for scenario 2.

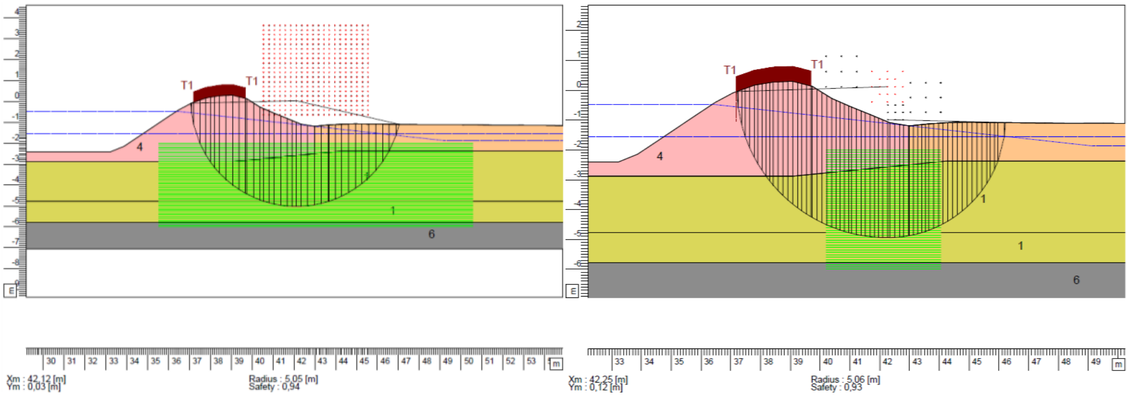


Figure H.17; Critical slip surface in the initial situation at normal water conditions as calculated with the Bishop (left image) and Uplift Van (right image) calculation model.

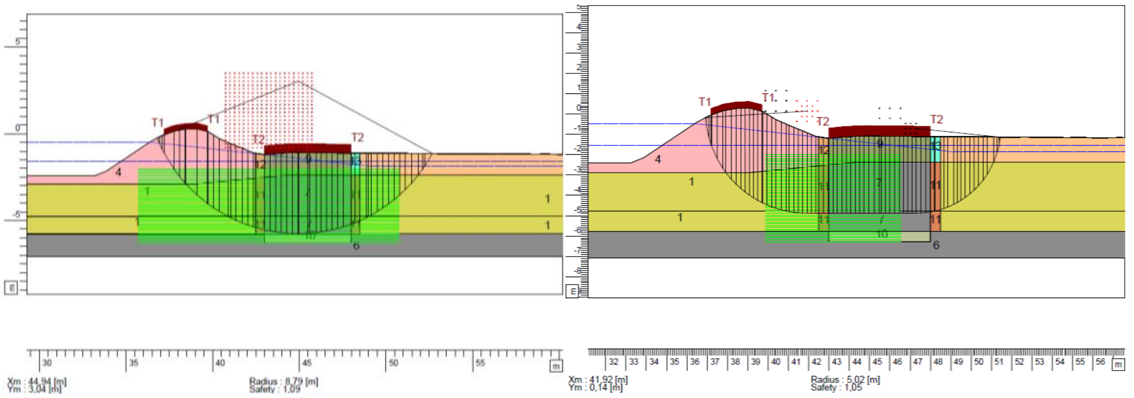


Figure H.18; Critical slip surface at normal water conditions as calculated with the Bishop (left image) and Uplift Van (right image) calculation model after the first section was stabilised.

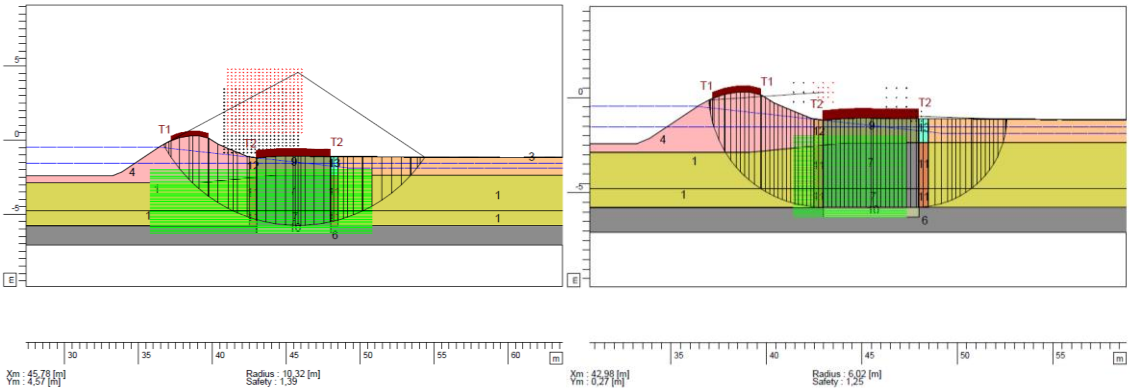


Figure H.19; Critical slip surface at normal water conditions as calculated with the Bishop (left image) and Uplift Van (right image) calculation model after 96 hours of curing after the first section was stabilised.

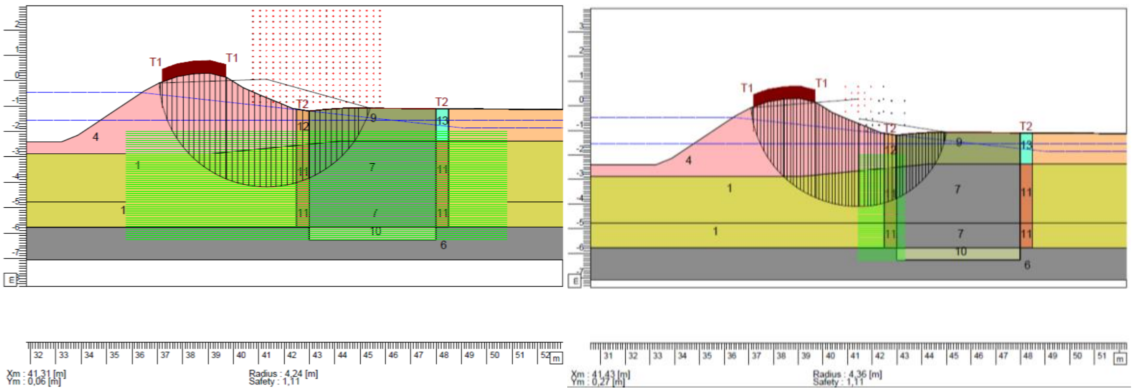


Figure H.20; Critical slip surface at normal water conditions as calculated with the Bishop (left image) and Uplift Van (right image) calculation model after removing all preload.

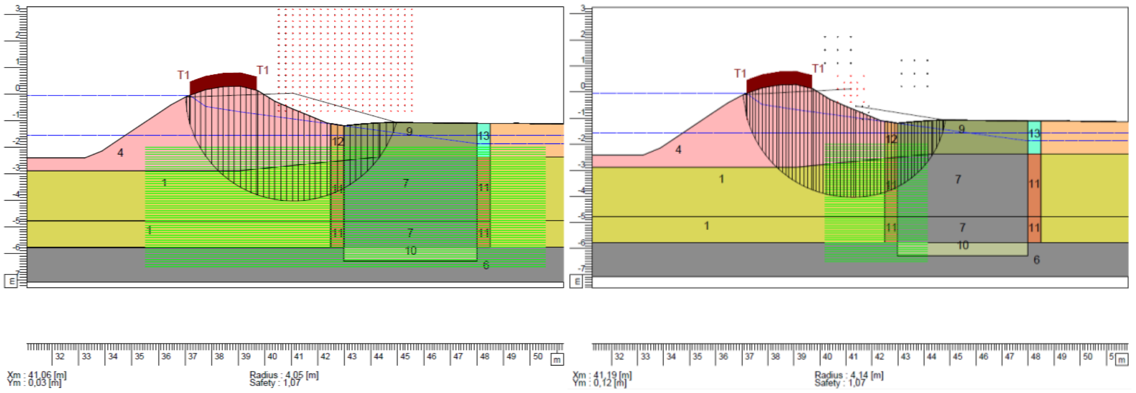


Figure H.21; Critical slip surface in the final situation at high water conditions as calculated with the Bishop (left image) and Uplift Van (right image) calculation model.

H.2.3 Scenario 3

In scenario 3, the implementation stability analyses were carried out with the assumptions on the change in the unit weight due to stabilisation and the initial strength of the stabilised soil directly after mixing as listed in table h.19. The design values of the parameters as applied during the implementation stability analyses based on the assumptions of table h.19 are presented in table h.20.

Table H.19; The assumptions on the change in the unit weight and the strength of the stabilised soil directly after mixing as applied in scenario 3.

Parameter	Scenario 3
Unit weight	No change
Initial shear strength after mixing	Zero strength

Table H.20; The design value of the unit weight of the stabilised soil after mixing and the design values of the effective strength parameters of the stabilised soil directly after mixing as applied in scenario 3.

Soil parameter	Mixture		Unit
	Stabilised peat	Stabilised organic clay	
$\gamma_{stab.;bulk;d}$	10,00	12,80	[kN/m^3]
$\gamma_{stab.;sat;d}$	10,00	12,80	[kN/m^3]
c'_d **after mixing**	0,00	0,00	[kPa]
ϕ'_d **after mixing**	0,00	0,00	[°]

The assumptions on the speed of stabilisation and the speed of applying or removing preload as applied during scenario 3 are presented in table h.21. The obtained Factors of Safety at each considered step during the implementation examined in scenario 3 are presented in

table h.22. The Factors of Safety at all time steps considered during the implementation examined in scenario 3 are presented graphically in figure h.22. A visual representation of the order of stabilisation at the 25 metres of levee at the Montfoortse Vaart as examined in scenario 3 is presented in figure h.23. The critical slip surfaces as determined with both the Bishop and Uplift Van calculation model at the end of each major step during the implementation analyses of scenario 3 are presented in figure h.24 through figure h.32.

Table H.21; The assumptions on the speed of stabilisation and the speed of applying or removing preload as applied in scenario 3.

Parameter	Value	Unit
Speed of stabilisation	100	[m^3/h]
Speed of applying or removing 8,0 kPa preload	3	[min/m^2]

Table H.22; The results of the implementation analyses of scenario 3 at each considered step of the implementation.

Implementation analyses results scenario 3						
Time	Action	Stab. soil vol.	Load area	Stab. length	FoS (Bishop)	FoS (Uplift Van)
[days]	[-]	[m³]	[m²]	[m]	[-]	[-]
0,00	Normal conditions	-	-	-	0,94	0,93
0,01	Stabilisation block 1	25	-	1	0,92	0,91
0,02	Preloading block 1	-	5	-	0,94	0,93
0,03	Stabilisation broadening block 1	25	-	1	0,92	0,91
0,04	Preloading broadened broadening block 1	-	5	-	0,93	0,92
0,05	Stabilisation broadening block 1 (part 2)	25	-	1	0,92	0,91
0,06	Preloading broadened block 1 (part 2)	-	5	-	0,93	0,92
1,06	Curing 24 hours	-	-	-	0,96	0,94
1,07	Stabilisation block 2	25	-	1	0,94	0,92
1,08	Preloading block 2	-	5	-	0,96	0,94
1,09	Stabilisation broadening block 2	25	-	1	0,94	0,92
1,10	Preloading broadened block 2	-	5	-	0,96	0,94
1,11	Stabilisation broadening block 2 (part 2)	25	-	1	0,94	0,92
1,13	Preloading broadened block 2 (part 2)	-	5	-	0,96	0,94
1,14	Stabilisation broadening block 2 (part 3)	25	-	1	0,94	0,92
1,15	Preloading broadened block 2 (part 3)	-	5	-	0,96	0,94
1,16	Stabilisation broadening block 2 (part 4)	25	-	1	0,94	0,92
1,17	Preloading broadened block 2 (part 4)	-	5	-	0,96	0,93
1,18	Stabilisation broadening block 2 (part 5)	25	-	1	0,94	0,91
1,19	Preloading broadened block 2 (part 5)	-	5	-	0,95	0,93
1,20	Stabilisation broadening block 2 (part 6)	25	-	1	0,93	0,91
1,21	Preloading broadened block 2 (part 6)	-	5	-	0,95	0,93
1,22	Stabilisation broadening block 2 (part 7)	25	-	1	0,93	0,91
1,23	Preloading broadened block 2 (part 7)	-	5	-	0,95	0,92
1,24	Stabilisation broadening block 2 (part 8)	25	-	1	0,92	0,91
1,25	Preloading broadened block 2 (part 8)	-	5	-	0,94	0,94
1,26	Stabilisation broadening block 2 (part 9)	25	-	1	0,92	0,93
1,27	Preloading broadened block 2 (part 9)	-	5	-	0,93	0,96
2,27	Curing 24 hours	-	-	-	1,10	1,09
2,28	Stabilisation block 3	25	-	1	1,08	1,08
2,29	Preloading block 3	-	5	-	1,10	1,09
2,30	Stabilisation broadening block 3	25	-	1	1,08	1,07
2,31	Preloading broadened block 3	-	5	-	1,09	1,09
2,32	Stabilisation broadening block 3 (part 2)	25	-	1	1,07	1,08
2,33	Preloading broadened block 3 (part 2)	-	5	-	1,09	1,08
2,34	Stabilisation broadening block 3 (part 3)	25	-	1	1,07	1,08
2,35	Preloading broadened block 3 (part 3)	-	5	-	1,08	1,08
2,36	Stabilisation broadening block 3 (part 4)	25	-	1	1,06	1,07
2,38	Preloading broadened block 3 (part 4)	-	5	-	1,08	1,07
2,39	Stabilisation broadening block 3 (part 5)	25	-	1	1,06	1,06
2,40	Preloading broadened block 3 (part 5)	-	5	-	1,07	1,07
2,41	Stabilisation broadening block 3 (part 6)	25	-	1	1,05	1,06
2,42	Preloading broadened block 3 (part 6)	-	5	-	1,07	1,06
2,43	Stabilisation broadening block 3 (part 7)	25	-	1	1,04	1,05
2,44	Preloading broadened block 3 (part 7)	-	5	-	1,06	1,05
2,45	Stabilisation broadening block 3 (part 8)	25	-	1	1,04	1,04
2,46	Preloading broadened block 3 (part 8)	-	5	-	1,05	1,04
2,47	Stabilisation broadening block 3 (part 9)	25	-	1	1,03	1,04
2,48	Preloading broadened block 3 (part 9)	-	5	-	1,04	1,04
2,49	Stabilisation broadening block 3 (part 10)	37,5	-	1,5	1,01	1,02

2,51	Preloading broadened block 3 (part 10)	-	7,5	-	1,04	1,03
3,51	Curing 24 hours	-	-	-	1,23	1,14
4,51	Curing 48 hours	-	-	-	1,28	1,17
5,51	Curing 72 hours	-	-	-	1,30	1,19
6,51	Curing 96 hours	-	-	-	1,33	1,20
7,51	Curing 120 hours	-	-	-	1,35	1,22
8,51	Curing 144 hours	-	-	-	1,37	1,23
9,51	Curing 168 hours	-	-	-	1,38	1,24
10,51	Curing 192 hours	-	-	-	1,39	1,24
11,51	Curing 216 hours	-	-	-	1,39	1,25
12,51	Curing 240 hours	-	-	-	1,39	1,25
13,51	Curing 264 hours	-	-	-	1,40	1,25
14,51	Curing 288 hours	-	-	-	1,40	1,25
14,76	Removing all preload	-	117,5	-	1,11	1,10
14,76	Test high water conditions	-	-	-	1,07	1,07

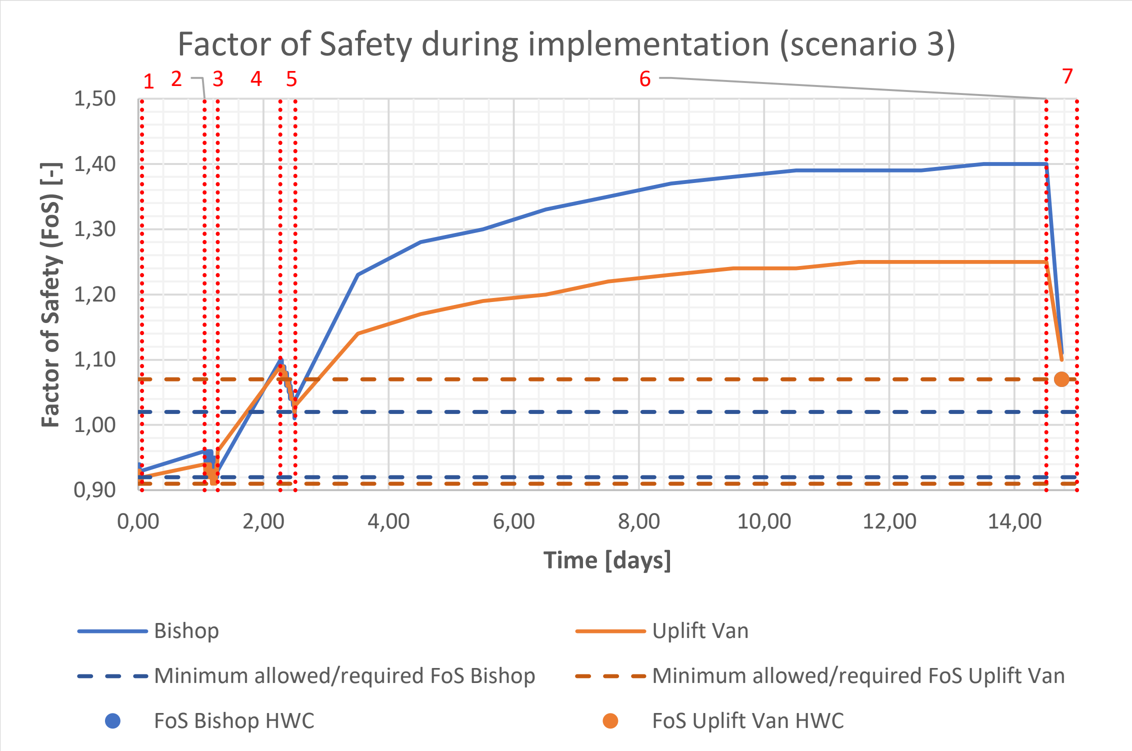


Figure H.22; The development of the Factor of Safety during the implementation of mass stabilisation at the toe of the levee with the assumptions of scenario 3. The red lines represent different actions taken during the implementation, the description of which is presented in table h.23.

Table H.23; Actions taken during the implementation of scenario 3.

Line number figure h.22	Action
1	Stabilisation 2,0 metres of soil
2	24 hours of curing of all blocks of stabilised soil
3	Stabilisation of 10,0 metres of soil
4	24 hours of curing of all blocks of stabilised soil
5	Stabilisation of final 11,5 metres of soil
6	288 hours of curing (i.e. 12 days) of all blocks of stabilised soil
7	Removing all preload

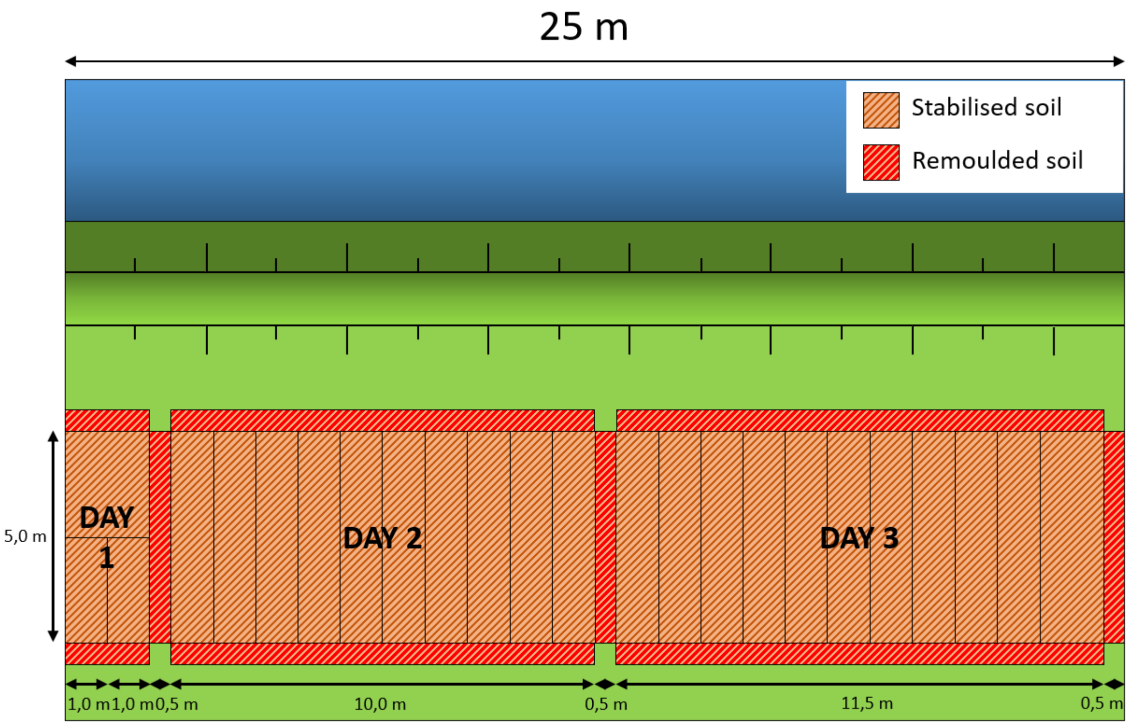


Figure H.23; Top view of the levee showing the obtained implementation for scenario 3.

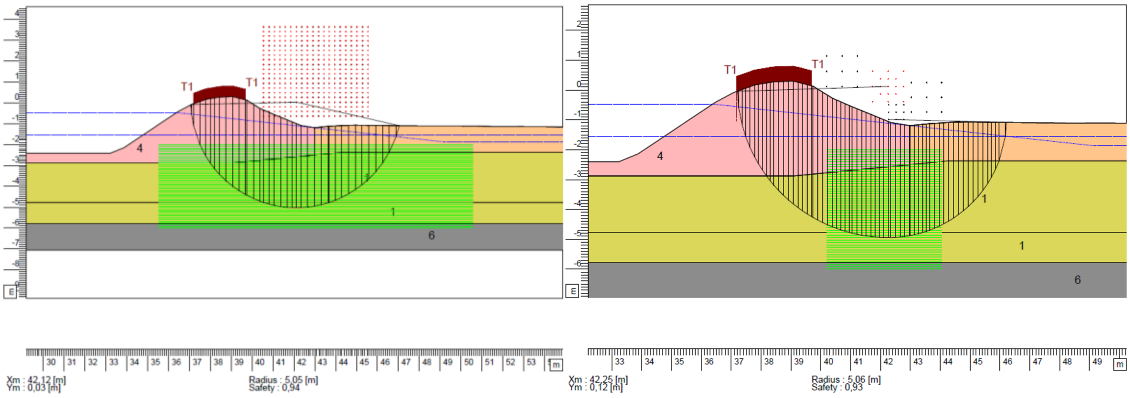


Figure H.24; Critical slip surface in the initial situation at normal water conditions as calculated with the Bishop (left image) and Uplift Van (right image) calculation model.

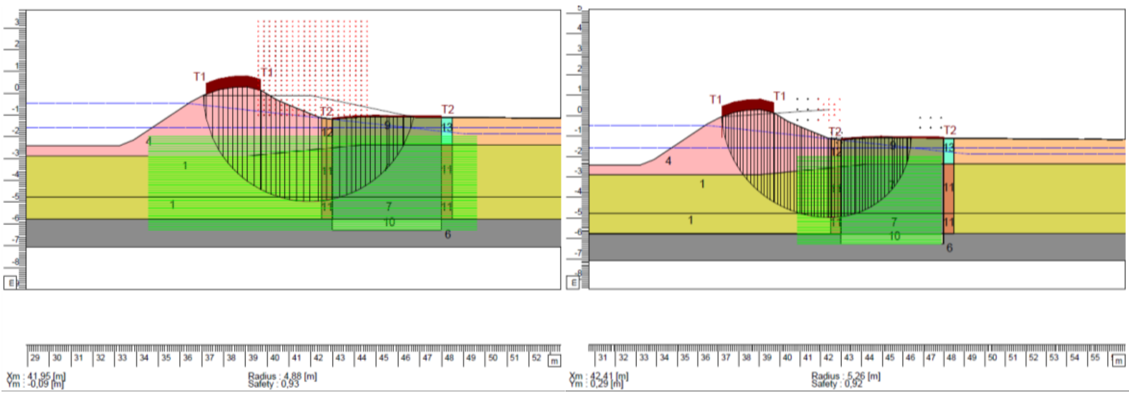


Figure H.25; Critical slip surface at normal water conditions as calculated with the Bishop (left image) and Uplift Van (right image) calculation model after the first section was stabilised.

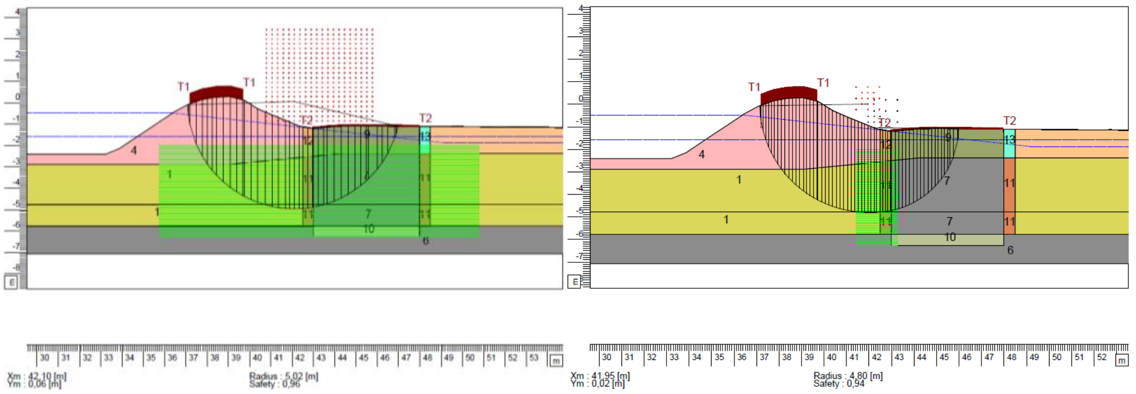


Figure H.26; Critical slip surface at normal water conditions as calculated with the Bishop (left image) and Uplift Van (right image) calculation model after 24 hours of curing after the first section was stabilised.

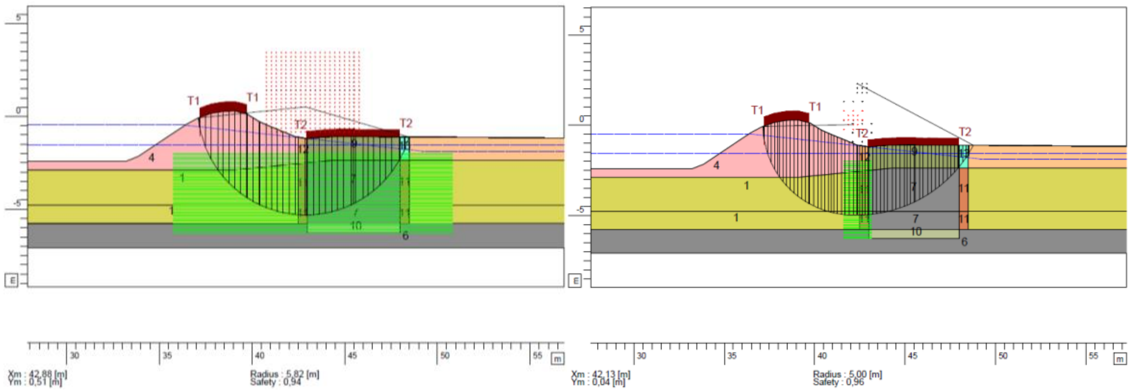


Figure H.27; Critical slip surface at normal water conditions as calculated with the Bishop (left image) and Uplift Van (right image) calculation model after the second section was stabilised.

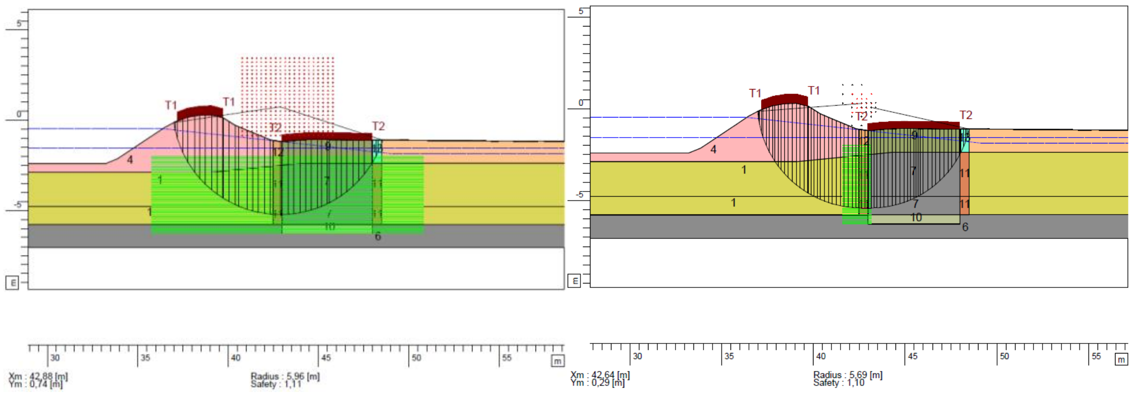


Figure H.28; Critical slip surface at normal water conditions as calculated with the Bishop (left image) and Uplift Van (right image) calculation model after 24 hours of curing after the second section was stabilised.

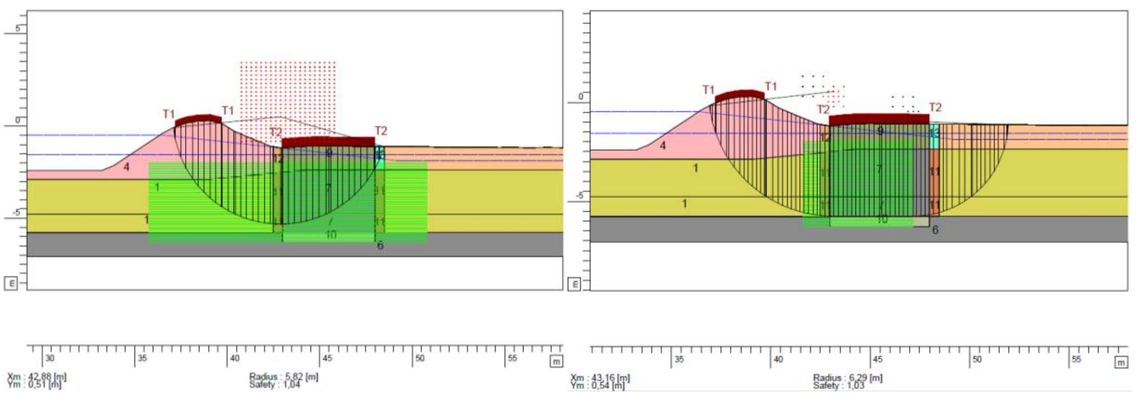


Figure H.29; Critical slip surface at normal water conditions as calculated with the Bishop (left image) and Uplift Van (right image) calculation model after the third section was stabilised.

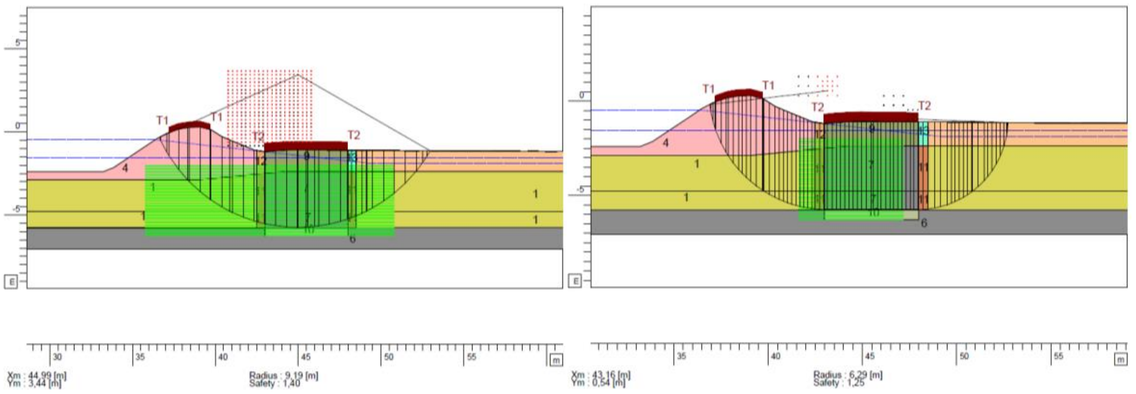


Figure H.30; Critical slip surface at normal water conditions as calculated with the Bishop (left image) and Uplift Van (right image) calculation model after 288 hours of curing after the third section was stabilised.

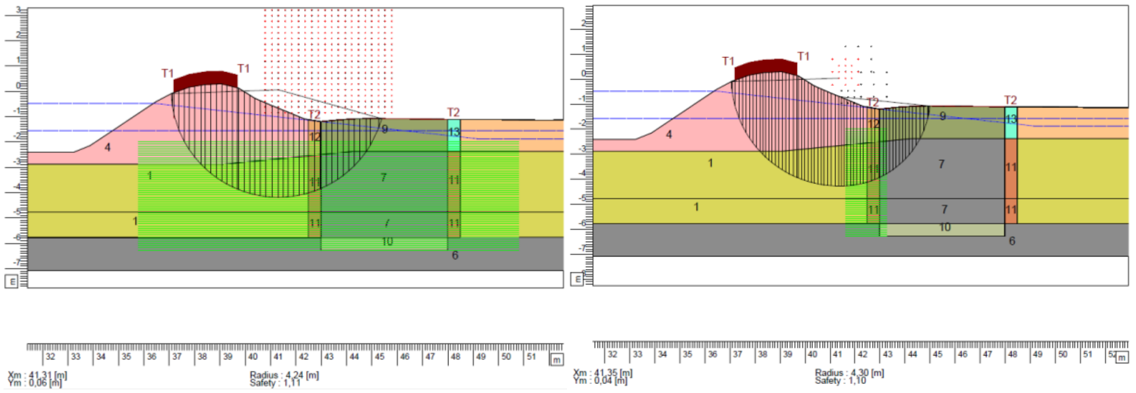


Figure H.31; Critical slip surface at normal water conditions as calculated with the Bishop (left image) and Uplift Van (right image) calculation model after 288 hours of curing after removing all preload.

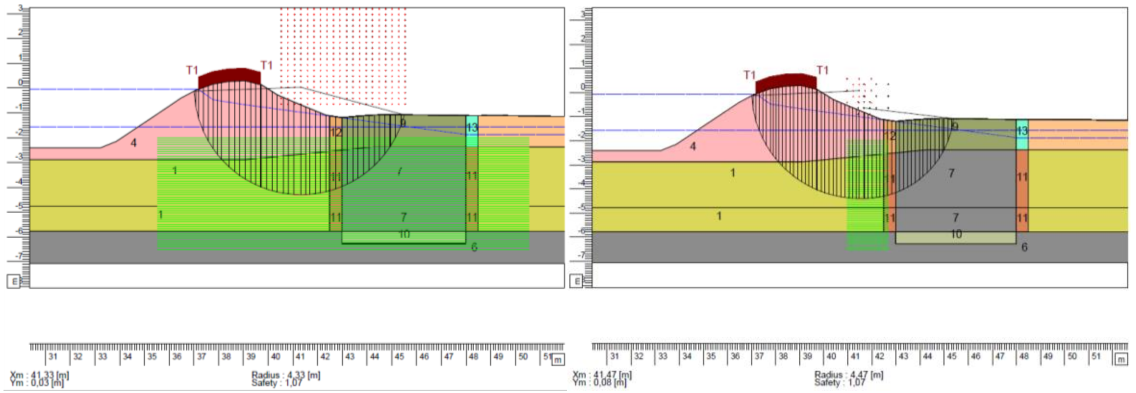


Figure H.32; Critical slip surface in the final situation at high water conditions as calculated with the Bishop (left image) and Uplift Van (right image) calculation model.

H.2.4 Scenario 4

In scenario 4, the implementation stability analyses were carried out with the assumptions on the change in the unit weight due to stabilisation and the initial strength of the stabilised soil directly after mixing as listed in table h.24. The design values of the parameters as applied during the implementation stability analyses based on the assumptions of table h.19 are presented in table h.25.

Table H.24; The assumptions on the change in the unit weight and the strength of the stabilised soil directly after mixing as applied in scenario 4.

Parameter	Scenario 4
Unit weight	No change
Initial shear strength after mixing	Reduced strength (remoulded)

Table H.25; The design value of the unit weight of the stabilised soil after mixing and the design values of the effective strength parameters of the stabilised soil directly after mixing as applied in scenario 4.

Soil parameter	Mixture		Unit
	Stabilised peat	Stabilised organic clay	
$\gamma_{stab.;bulk;d}$	10,00	12,80	[kN/m³]
$\gamma_{stab.;sat;d}$	10,00	12,80	[kN/m³]
c'_d after mixing	0,27	0,33	[kPa]
ϕ'_d after mixing	5,15	13,60	[°]

The assumptions on the speed of stabilisation and the speed of applying or removing preload as applied during scenario 4 are presented in table h.26. The obtained Factors of Safety at each considered step during the implementation examined in scenario 4 are presented in table h.27. The Factors of Safety at all time steps considered during the implementation examined in scenario 4 are presented graphically in figure h.34. A visual representation of the order of stabilisation at the 25 metres of levee at the Montfoortse Vaart as examined in scenario 4 is presented in figure h.35. The critical slip surfaces as determined with both the Bishop and Uplift Van calculation model at the end of each major step during the implementation analyses of scenario 4 are presented in figure h.36 through figure h.39.

Table H.26; The assumptions on the speed of stabilisation and the speed of applying or removing preload as applied in scenario 4.

Parameter	Value	Unit
Speed of stabilisation	100	[m³/h]
Speed of applying or removing 8,0 kPa preload	3	[min/m²]

Table H.27; The results of the implementation analyses of scenario 4 at each considered step of the implementation.

Implementation analyses results scenario 4						
Time	Action	Stab. soil vol.	Load area	Stab. length	FoS (Bishop)	FoS (Uplift Van)
[days]	[-]	[m³]	[m²]	[m]	[-]	[-]
0,00	Normal conditions	-	-	-	0,94	0,93
0,01	Stabilisation block 1	25	-	1	0,92	0,91
0,02	Preloading block 1	-	5	-	0,94	0,93
0,03	Stabilisation broadening block 1	25	-	1	0,93	0,92
0,04	Preloading broadened block 1	-	5	-	0,95	0,93
0,05	Stabilisation broadening block 1 (part 2)	25	-	1	0,94	0,92
0,06	Preloading broadened block 1 (part 2)	-	5	-	0,95	0,94
0,07	Stabilisation broadening block 1 (part 3)	25	-	1	0,94	0,93
0,08	Preloading broadened block 1 (part 3)	-	5	-	0,96	0,95
0,09	Stabilisation broadening block 1 (part 4)	25	-	1	0,95	0,93
0,10	Preloading broadened block 1 (part 4)	-	5	-	0,97	0,95
0,11	Stabilisation broadening block 1 (part 5)	25	-	1	0,96	0,94
0,13	Preloading broadened block 1 (part 5)	-	5	-	0,98	0,98
0,14	Stabilisation broadening block 1 (part 6)	25	-	1	0,96	0,94
0,15	Preloading broadened block 1 (part 6)	-	5	-	0,98	0,96
0,16	Stabilisation broadening block 1 (part 7)	25	-	1	0,97	0,95
0,17	Preloading broadened block 1 (part 7)	-	5	-	0,98	0,97
0,18	Stabilisation broadening block 1 (part 8)	25	-	1	0,97	0,96
0,19	Preloading broadened block 1 (part 8)	-	5	-	0,98	0,97
0,20	Stabilisation broadening block 1 (part 9)	25	-	1	0,97	0,96
0,21	Preloading broadened block 1 (part 9)	-	5	-	0,98	0,98
0,22	Stabilisation broadening block 1 (part 10)	25	-	1	0,97	0,97
0,23	Preloading broadened block 1 (part 10)	-	5	-	0,99	0,98

0,24	Stabilisation broadening block 1 (part 11)	25	-	1	0,97	0,97
0,25	Preloading broadened block 1 (part 11)	-	5	-	0,99	0,99
0,26	Stabilisation broadening block 1 (part 12)	25	-	1	0,97	0,97
0,27	Preloading broadened block 1 (part 12)	-	5	-	0,99	0,99
0,28	Stabilisation broadening block 1 (part 13)	25	-	1	0,97	0,99
0,29	Preloading broadened block 1 (part 13)	-	5	-	0,99	1,01
0,30	Stabilisation broadening block 1 (part 14)	25	-	1	0,97	1,02
0,31	Preloading broadened block 1 (part 14)	-	5	-	0,99	1,01
0,32	Stabilisation broadening block 1 (part 15)	25	-	1	0,97	1,01
0,33	Preloading broadened block 1 (part 15)	-	5	-	0,99	1,03
0,34	Stabilisation broadening block 1 (part 16)	25	-	1	0,97	1,03
0,35	Preloading broadened block 1 (part 16)	-	5	-	0,99	1,03
0,36	Stabilisation broadening block 1 (part 17)	25	-	1	0,97	1,02
0,38	Preloading broadened block 1 (part 17)	-	5	-	0,99	1,03
0,39	Stabilisation broadening block 1 (part 18)	25	-	1	0,97	1,02
0,40	Preloading broadened block 1 (part 18)	-	5	-	0,99	1,02
0,41	Stabilisation broadening block 1 (part 19)	25	-	1	0,97	1,01
0,42	Preloading broadened block 1 (part 19)	-	5	-	0,99	1,02
0,43	Stabilisation broadening block 1 (part 20)	25	-	1	0,97	1,01
0,44	Preloading broadened block 1 (part 20)	-	5	-	0,99	1,01
0,45	Stabilisation broadening block 1 (part 21)	25	-	1	0,97	1,01
0,46	Preloading broadened block 1 (part 21)	-	5	-	0,99	1,01
0,47	Stabilisation broadening block 1 (part 22)	25	-	1	0,97	1,00
0,48	Preloading broadened block 1 (part 22)	-	5	-	0,99	1,00
0,49	Stabilisation broadening block 1 (part 23)	37,5	-	1,5	0,96	1,00
0,51	Preloading broadened block 1 (part 23)	-	7,5	-	0,99	1,00
1,51	Curing 24 hours	-	-	-	1,2	1,11
2,51	Curing 48 hours	-	-	-	1,28	1,17
3,51	Curing 72 hours	-	-	-	1,3	1,19
4,51	Curing 96 hours	-	-	-	1,33	1,2
5,51	Curing 120 hours	-	-	-	1,35	1,22
6,51	Curing 144 hours	-	-	-	1,38	1,24
7,51	Curing 168 hours	-	-	-	1,4	1,25
7,77	Removing preload	-	122,5	-	1,11	1,1
7,77	Test high water conditions	-	-	-	1,07	1,07

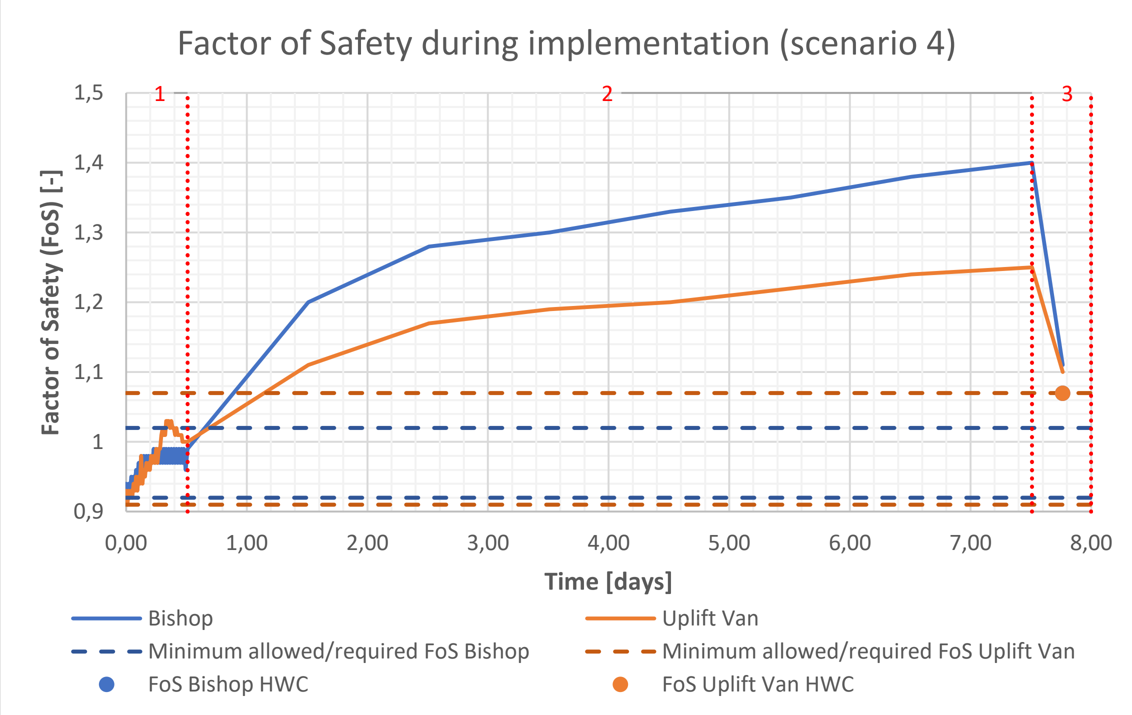


Figure H.33; The development of the Factor of Safety during the implementation of mass stabilisation at the toe of the levee with the assumptions of scenario 4. The red lines represent different actions taken during the implementation, the description of which is presented in table h.28.

Table H.28; Actions taken during the implementation of scenario 4.

Line number figure h.33	Action
1	Stabilisation 24,5 metres of soil
2	168 hours of curing (i.e. 7 days) of all blocks of stabilised soil
3	Removing all preload

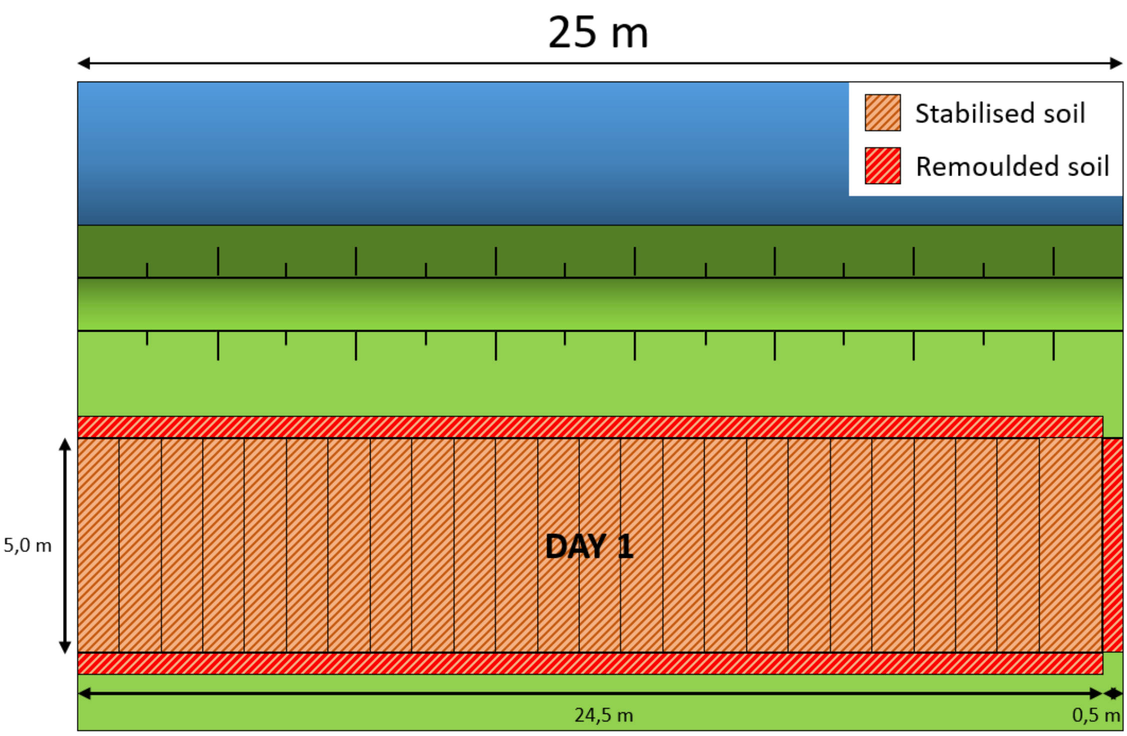


Figure H.34; Top view of the levee showing the obtained implementation for scenario 4.

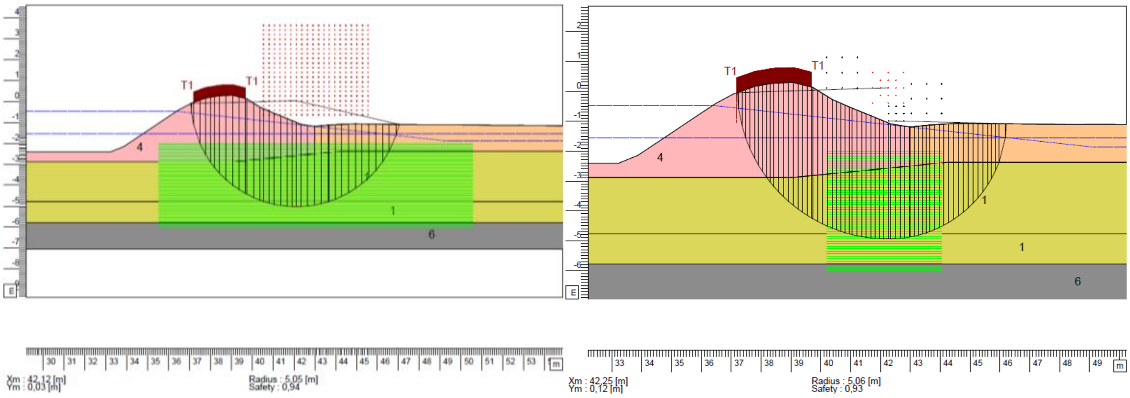


Figure H.35; Critical slip surface in the initial situation at normal water conditions as calculated with the Bishop (upper image) and Uplift Van (lower image) calculation model.

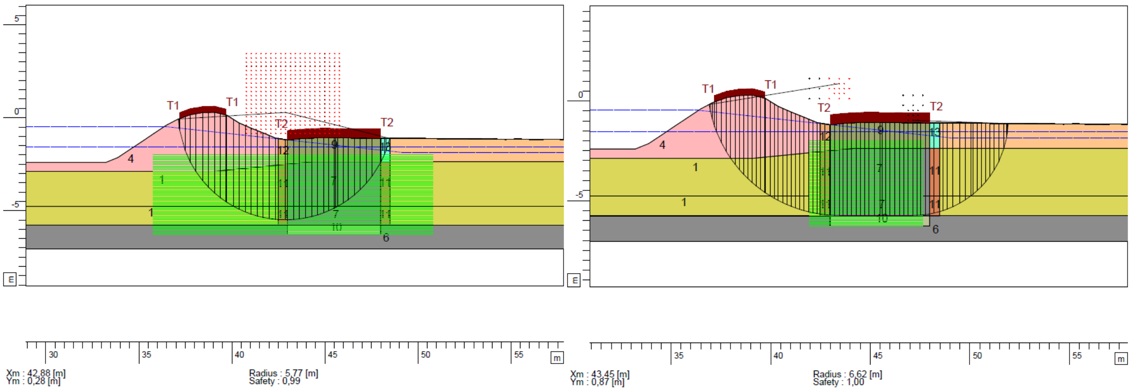


Figure H.36; Critical slip surface at normal water conditions as calculated with the Bishop (upper image) and Uplift Van (lower image) calculation model after the first section was stabilised.

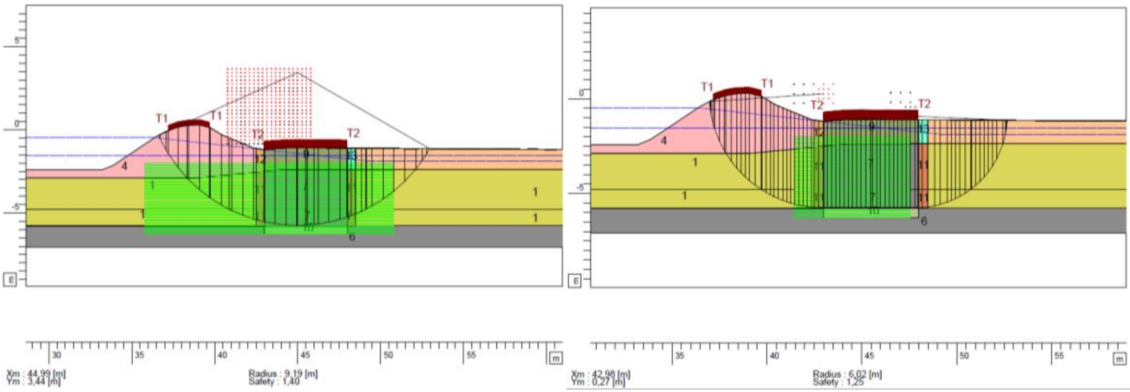


Figure H.37; Critical slip surface at normal water conditions as calculated with the Bishop (upper image) and Uplift Van (lower image) calculation model after 168 hours of curing after the first section was stabilised.

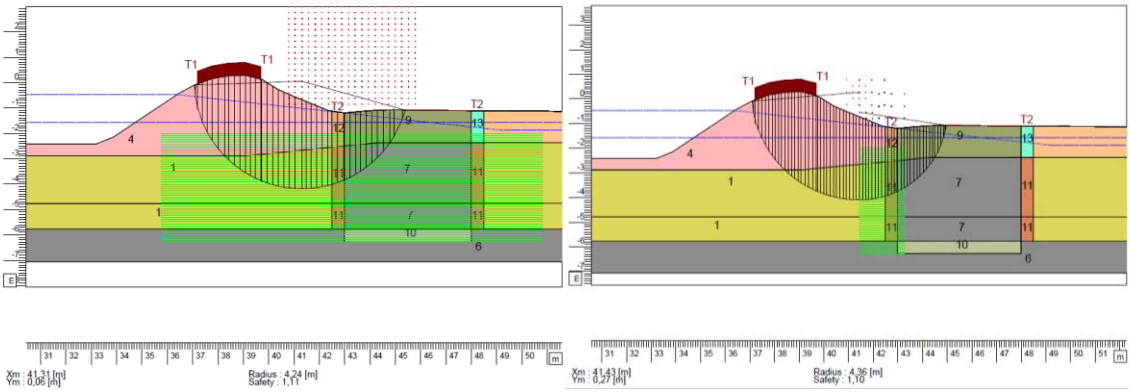


Figure H.38; Critical slip surface at normal water conditions as calculated with the Bishop (upper image) and Uplift Van (lower image) calculation model after removing all preload.

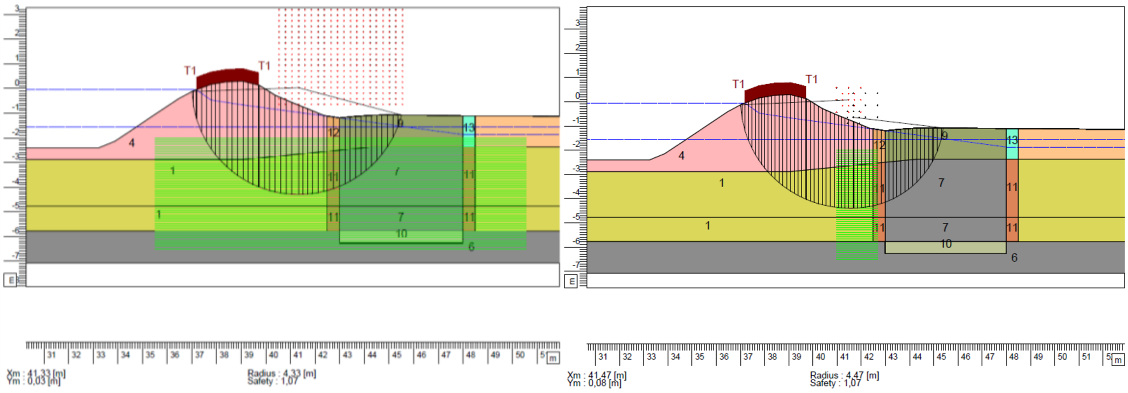


Figure H.39; Critical slip surface in the final situation at high water conditions as calculated with the Bishop (upper image) and Uplift Van (lower image) calculation model.

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