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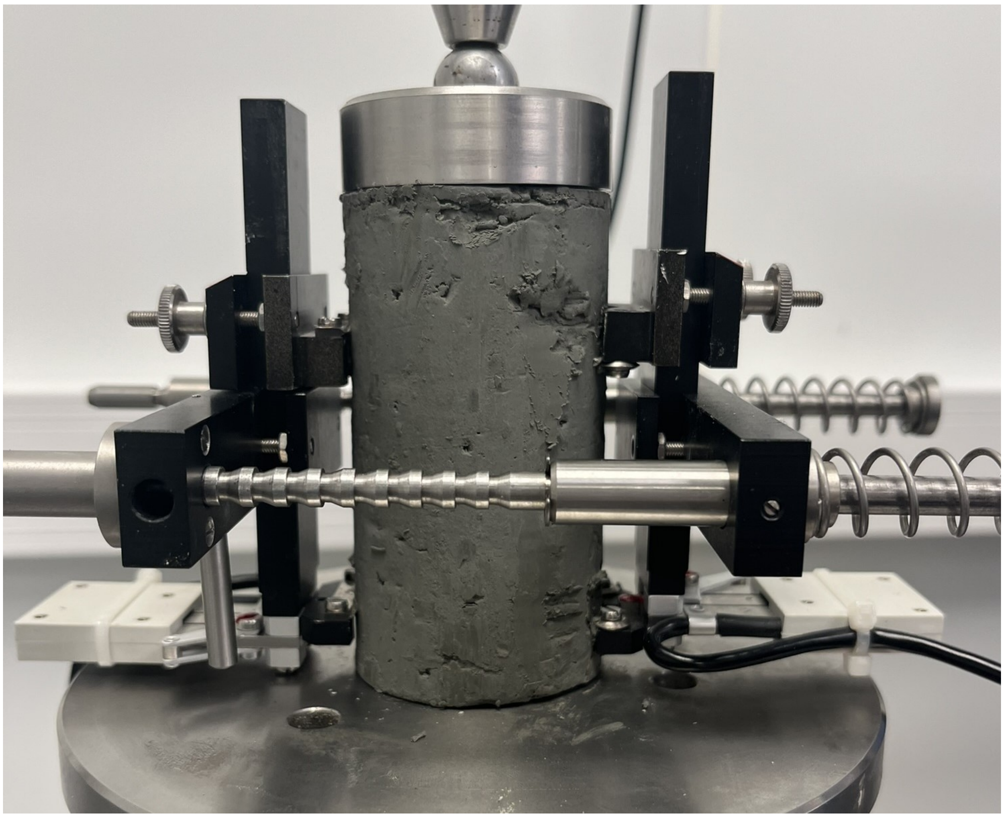

Calcined Clay in Dry Deep Mixing

A preliminary investigation of a binder mixture containing calcined clay to reduce environmental impact.

Master's thesis in Infrastructure and Environmental Engineering

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Cover: Sample mixed with the investigated binder mixture, containing calcined clay, during unconfined compression strength test.

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Abstract

Dry deep mixing is a common form of ground improvement in the Nordic countries. With the increasing strive to reduce carbon emissions, smart solutions are frequently needed. This thesis presents a new type of binder mixture containing calcined clay. The proposed binder mixtures were combined with a wet silty-clay found in the northern parts of Gothenburg. The binders have been subjected to a standardised UCS-test from which the local secant stiffness (E50) and unconfined compressive strength were derived. Data from the laboratory test was compiled and compared against requirements obtained from a finite element analysis. The model was constructed in compliance with Trafikverket's regulations. One of the proposed new binders which contained 80 % lime and 20 % calcined clay surpassed the set criteria after 28 days of curing. The achieved unconfined compressive strength was approximately 200 kPa and 100 MPa for the local secant stiffness. Furthermore, the potential emission reduction with the proposed binder was calculated to 11 % when 80 $\frac{kg_{binder}}{m^3_{clay}}$ was used when compared to a traditional lime-cement mixture. However, the new binder is still in an early testing phase which is associated with large uncertainties, namely the variance in results. In conclusion, while promising, further research will be crucial to fully assess the long-term performance and environmental impact of the proposed binder mixture.

Keywords: Dry deep mixing, Calcined Clay, UCS-test, local secant stiffness, Finite element analysis, Carbon Dioxide Emission.

Acknowledgements

With this section we would like to take the opportunity to thank the people who have helped us and this thesis along the way. Firstly, we would like to thank our examiner Jelke Dijkstra for all constructive criticism and guidance during the project. We would also like to thank the research engineer for the Geology and Geotechnics laboratory Burcu Ayar for keeping our spirits constantly high and with all the help in the laboratory. Last but not least, a huge thanks to our supervisor Vijayshree Sadasivan, without her this thesis would simply not be possible. Her knowledge in the laboratory and feedback during the project played a crucial part for this thesis.

William Högefjord Kristiansson, Gothenburg, June 2024
Tobias Wedholm, Gothenburg, June 2024

List of Acronyms

Below is the list of acronyms that have been used throughout this thesis listed in alphabetical order:

AAB	Alkali-Activated Binders
CAH	Calcium Aluminate Hydroxides
CKD	Cement Kiln Dust
CSH	Calcium Silicate Hydroxides
DDM	Dry Deep Mixing
FE	Finite Element
GGBS	Ground Granulated Blast Slag
LCA	Life-cycle assessment
LL	Liquid limit
ORC	Over Consolidation Ratio
OC	Organic content
PL	Plastic limit
UCS	Unconfined Compressive Strength
WC	Water Content

Nomenclature

Below is the nomenclature of indices and parameters that have been used throughout this thesis.

Indices

M_x	Mixture type
D_x	Curing days
ur	unloading/reloading
Emb	Embankment

Parameters

γ	Unit weight
ν	Poisson's ratio
φ	Friction angle
φ'	Effective friction angle
E'_{ref}	Young's modulus
$c'_{u,ref} = s_{u,ref}$	Undrained Shear Strength
$k_v = k_h$	Hydraulic conductivity, Vertical & Horizontal direction
$K_{0,x} = K_{0,z}$	Lateral stress
K_0^{NC}	Lateral stress in normal consolidation
e_0	initial void ratio
λ^*	Modified compression index
κ^*	Modified swelling index
c'_{ref}	Effective cohesion
C_k	Void ratio dependency for permeability
S_{Emb}	Settlement for embankment

M_{sf}	Factor of safety
qu	Unconfined compressive strength
E_{50}	Secant stiffness modulus at 50% ultimate stress
E_0	Initial tangent modulus (Initial stiffness)

Contents

List of Figures

List of Tables

1 Background

The building sector stands before a tremendous change in the strive for a more climate neutral future. Sweden has set a goal of carbon neutrality before the year 2045 (Regeringskansliet, 2023). The building sector is currently responsible for 22 % of the total carbon dioxide emissions in Sweden (Fossilfritt Sverige, 2022). In addition, the emissions from the building sector has reached an all time high from a global perspective (United Nations, 2022). Especially high is the cement production which contributes roughly seven percent towards the rising global emissions (O'Rourke et al., 2009). Although a larger CO_2-production by weight of product, lime contributes with approximately one percent of the global industry emissions (CARMEUSE, 2023). Furthermore, both lime and cement are affiliated with high energy consumption and extraction of natural resources.

The substantial amounts of soft soil present, especially in Nordic countries, has caused ground improvements to be frequently needed. Dry deep mixing (DDM) has been widely used during the last 50 years in the Nordic countries for soft soil stabilisation (Hov & Larsson, 2023). DDM is a universal term for ground improvement when a dry mixture often is mixed with the soil (Larsson, 2021). The DDM method utilises both chemical and mechanical processes to improve the structural integrity of the soil (Chen et al., 2020). However, concerns regarding contamination of the surrounding soil due to leaching from the columns is associated with using lime and cement for ground improvement (Löfroth, 2005). The effects of leaching from the columns are difficult to estimate and are limited in there research. Leaching from the columns can however have a quite significant impact due to an increase in pH and calcium-oxides (CaO) in the surrounding soil.

This study has been investigating the possibility of including thermally activated clay in DDM to reduce the environmental impact. This was done by replacing parts of the usually used lime-cement mixture with activated clay. The calculations for the environmental impact reduction of DDM with a new binder mixture was based on a scenario where a generic railway embankment is modelled.

The stability and settlement of the embankment were evaluated to estimate when the structural integrity was deemed to be sufficient. Following this process laboratory work was conducted. This was done to find a new binder mixture which fulfilled these criteria. Due to the topic not being previously researched a large focus was on investigating the viability of the method. This study should therefore act as a baseline to if the method works or not for future investigations and research.

1.1 Research questions

- Is it possible to replace parts of lime and cement with activated clay to contribute to structural integrity in a DDM solution?
- Does the potential new binder, including calcined clay, generate sufficient structural integrity to support a generic embankment?
- How would a potential new binder mixture compare to a standardised lime-cement mixture regarding environmental impact?

1.2 Limitations and assumptions

Achieving totally homogeneous columns are practically impossible for both laboratory work as well as in-situ. This will be disregarded when comparing results from the numerical analysis and measured values. Therefore, the homogeneity is not assumed to be further affected by the process of deep mixing at a larger scale than in a laboratory setting. This meant that the results from heterogeneous samples were assumed to represent a full scale column. In addition, no economic aspect will be taken into account when analysing the method.

Different common practises which are utilised in the industry during dimensioning and construction stages will not be considered when reviewing the results achieved in this project. Furthermore, the time in which the deformations of the embankment are analysed will be done after full consolidation. This implies that effects such as creep will not be taken into account for either settlement or stability.

2 Literature review

A mixture of lime and cement is the most common binder in ground improvement and DDM is today referred to as lime-cement columns (Larsson, 2021). Although DDM is a proven and functioning method from a structural point of view, problems are exposed when the environmental effects are examined. Cement, which as previously mentioned is one of the most commonly used products in DDM, contributes large amount of CO_2 emissions. Additionally, the production of cement generates significant amount of by-products which are in themselves problematic (Huntzinger & Eatmon, 2009).

The hydration process of lime and cement occurs in three distinct stages. The calcium-oxides available for binding react with the ground water and create calcium-hydroxides $[Ca(OH)_2]$ (Hov & Larsson, 2023). The $Ca(OH)_2$ in turn reacts with the aluminosilicate minerals through pozzolanic reactions. The resulting particles are calcium silicate hydrates (CSH) and calcium aluminate hydrates (CAH). The produced CAH and CSH will in turn react with the soil particles and cause the soil to flocculate (Chen et al., 2020). The reactions take several years to finalise and are further slowed down with low temperature and pH (Broms, 1991; Carlsten, 1996).

Since DDM has become an established method, several binders including fly ash and blast furnace slag have been evaluated and compared (Broms, 1991). Life-cycle assessment (LCA) has been the standard way to assess climate impact. However, the climate impact from binders such as the the one presented in this report, is a complex issue. More established alternatives to the traditional lime-cement binder mixture have been evaluated using LCA. One of the more researched alternative has been cement kiln dust (CKD), which is a waste product of cement production. Replacing a cement binder mixture with CKD can reduce the carbon dioxide emissions by 50 to 55 % CO_2-eq per ton of binder (Hov & Larsson, 2023). However, CKD has had negative effects regarding respiratory health risks in addition to the ones regarding environmental impact. CKD has been seen as hazardous waste but was more generally classified as special waste by EPA (1993). If not handled according to the regulations set in place, CKD is seen as hazardous waste and is therefore problematic when regarding storage and handling (EPA, 1999).

Research indicate that activated clay result in 94 g CO_2/kg in comparison to 824 g CO_2/kg to produce cement (Plusquellec et al., 2020). Furthermore, activated clay does not require as high temperatures as traditional cement to be activated and thereby reduces the overall energy consumption (Plusquellec et al., 2020). When comparing the produced emissions caused by activated clay and lime the difference is even greater. The production of lime produces more than 987 g CO_2/kg (Schorcht et al., 2013).

A sub-category of binders used for ground improvement is alkali-activated binders (AAB), also known as geopolymer (Bernal et al., 2013). AAB can consist of multiple different products which chemically react with an alkaline activator such as lime. The alkali-activated material reacts with aluminosilicates present in the soil and creates a cementitious effect (Bernal et al., 2013). Research indicates that alkali-activated fly ash produces a higher strength than traditionally used cement (Cristelo et al., 2011). Cristelo validated these results with a field test which alluded to a similar strength build up. Fly ash combined with slag was evaluated and has shown to obtain higher compressive strengths than specimens prepared with cement and/or lime when stabilising marine clay (Arulrajah et al., 2018). Ground granulated blast slag (GGBS) can also be alkali-activated to create geopolymers which allows for more pozzolanic reactions to occur (Sargent et al., 2013). In a followup study also performed by Sargent et al. (2016), alkali-activated GGBS yielded higher strength than those observed in specimens prepared with CEM I. AAB in the form of sodium hydroxide (N_aOH) was added to GGBS which produced significant strength and durability improvements. GGBS have also been proven to increase strength and reduce void ratios if combined with fly ash (Abbey & Olubanwo, 2018). A study which compared nine different binders for strength, stiffness and climate impact found that a mixture of Portland cement and blast furnace slag performed best in most aspects (Ramírez et al., 2024). The usage of AAB, in the form of products such as fly ash, and GGBS does however rely on waste products. Although good results were achieved, this causes a production of such binder to not be a sustainable option in a long term perspective. Current research is universally in agreement that there has been potential for a more environmentally friendly binder. Clays which contained kaolinite can be stabilized with lime to both improve workability and strength build up (Hilt & Davidson, 1960).

Due to the assumption of laboratory prepared and tested samples representing the in-situ parameters of columns, large discrepancies can occur. The measured in-situ unconfined compressive strength (UCS) and stiffness can be between 20-40 % lower than the ones achieved from laboratory samples (Sekhar et al., 2010). Due to the inherent variability of results, in both laboratory and in-situ mixing, estimations of needed and achieved structural parameters is a complicated process. Reports regarding spatial variability of in-situ mixed lime-cement columns shows that the difference can be quite significant (Wong et al., 2024). The report show that the lowest measured Young's modulus in a sample from a given site was around half of that of the stiffest sample. This finding was presented as a relation between E_{50} and the UCS of each sample, to minimise the impact of the differences in sampling depths. The stiffness-strength relations indicated on similar trends where the minimum derived stiffness-strength relation was around half of the maximum one.

Ground improvement in general is combined with an inherent variability in site conditions due to the many aspects which directly or indirectly affect the results. Due to this, an optimal universal binder is not feasible (Ahnberg et al., 2003). Many aspects could affect the choice of type and amount of binder. In addition, the robustness of the binder is difficult to measure without numerous laboratory tests. In the study presented by Ahnberg et al. (2003), several binder types were tested with different types of soils. The study showed a large, and in advance unpredictable, variability of the results. Some binders showed fairly good results in many soils while some binders showed results ranging from very good to very poor depending on type of soil.

Specific types of natural Swedish clays contain minerals which are aluminosilicates, where kaolin is one of the most common ones (Maj & Matus, 2023). For the aluminosilicates in the clay to chemically react they do however need to be activated. This can be done either by a mechanical or thermal process. The thermal process is done by heating up the minerals to a temperature of approximately 600-800 degrees depending on the specific soil (Plusquellec et al., 2020). The thermally activated clay is generally referred to as calcined clay. No studies have been conducted on the strength and stiffness when calcine clay is included, highlighting the importance of this study.

3 Method

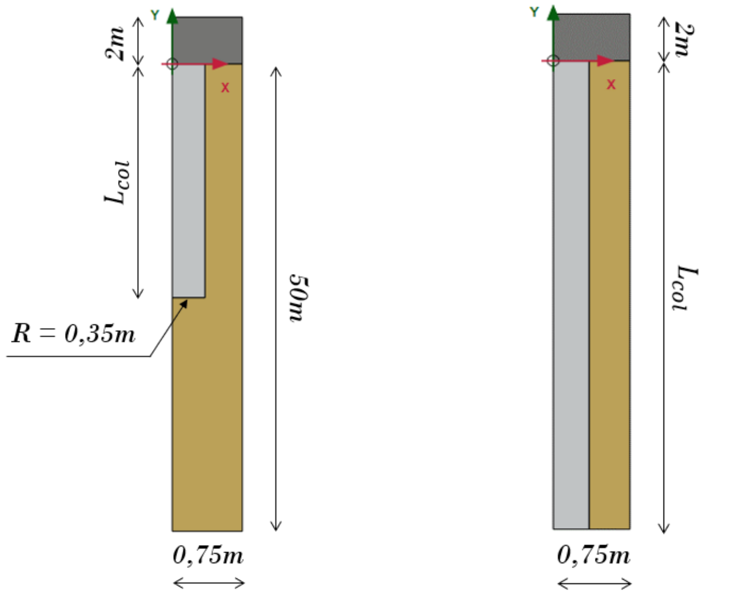
Firstly, a finite element (FE)-analysis was conducted. This was done to investigate what strength and stiffness parameters were needed in DDM-columns at a generic embankment project. The achieved parameters from the FE-analysis were used as a foundation for the laboratory work. The laboratory work intended to investigate the viability of using a more environmentally friendly binder containing calcined clay. This was done while trying to achieve strength and stiffness which were equal to or higher than the ones found in the FE-analysis.

3.1 Numerical model

Two main aspects were investigated with the numerical model, the total settlement of the embankment and the stability. This project has been investigating both end-bearing as well as floating columns by simulating scenarios in the software "PLAXIS 2D CONNECT" edition V22. In both cases the columns were assumed to have a diameter of 70 centimetres and a centre to centre distance of 1.5 meters. In the case of investigating total settlements of the embankment, axis-symmetry was utilised. The model was based on a single column where full embankment height of 2 meters was present. A visualisation of this model can be seen in Figure 3.1. For the stability analysis the entire embankment was modelled where a width of 12 meters with full embankment height was assumed. Thus, accommodating Trafikverket's requirements for railway constructions (Trafikverket, 2019), see Figure 3.2. Column lengths of 10, 15, 20, and 25 meters were investigated for both floating and end-bearing columns. The depth of the clay layer for the floating columns was fixed at 50 meters.

Due to the project being focused on the material properties some additional assumptions were made. More specifically, installation effects from the DDM were neglected. The columns were assumed to be homogeneous in terms of spatial variability and mixing. This assumption implies that the columns had no weaker areas resulting from factors such as air pockets or grain size variability, which could potentially affect their structural integrity. In addition no variability of parameters with the depth was assumed for the columns. The soil was however modelled with the soft-soil model where depth dependency in the material parameters was considered. However, no spatial variability for the soil was assumed. The groundwater table was assumed to be located 1 meter under the ground surface and drainage was only permitted through the top.

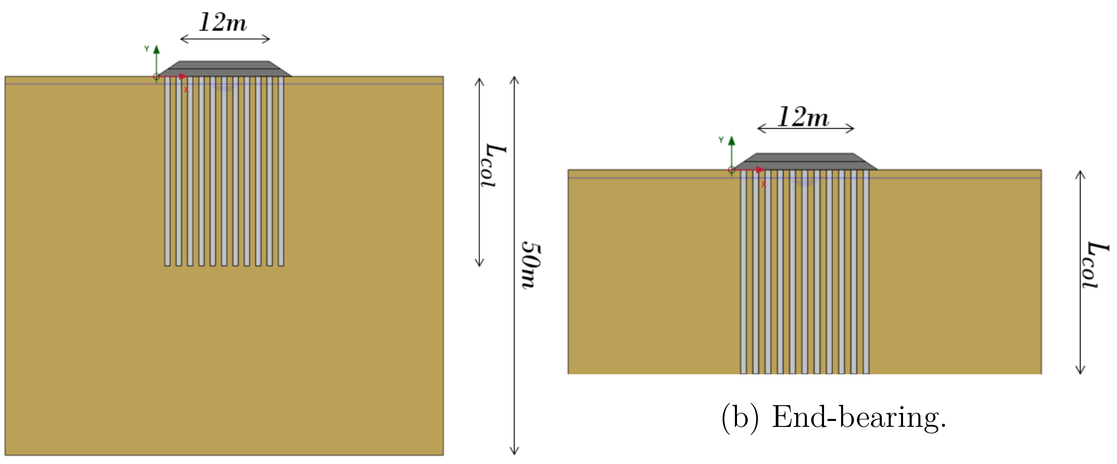
Figure 3.1
Conceptual models for the settlement analysis.



(a) Floating. (b) End-bearing.

Note: The figures are not made to scale in order to enhance the clarity.

Figure 3.2
Numerical model for stability analysis.



(a) Floating. (b) End-bearing.

Table 3.1
Allowed total settlements for railway embankments.

Max velocity [km/h]	Allowed total settlement [cm]
100	30
160	20
200	20
250	20
350	10

Source: (Karlsson & Moritz, 2014).

Table 3.2
Minimum allowed safety factor per safety classification.

Safety Classification	Safety Factor
1	1.35
2	1.5
3	1.65

Source: (Karlsson & Moritz, 2014).

When analysing Trafikverket's regulations for railway embankment there are mainly three separate criteria regarding deformations. These are total settlements, differential settlements and stability (Karlsson & Moritz, 2014). The criterion regarding differential settlements was disregarded for this project since the investigated theoretical site was assumed to be homogeneous. The criterion of total settlements for railway is dependant on the velocity which the railway is intended for, as can be seen in Table 3.1. Since the assumed maximum velocity was 250 km/h, a limit of 20 cm was used as the maximum allowed total settlement. The stability of the embankment was measured in terms of a safety factor. Depending on the safety classification needed for a project different safety factors are required, as can be seen in Table 3.2. This project, which investigated the increase in loading capacity using columns, required safety classification 2. This implied a safety factor of $F_{sf} \geq 1.5$ as per Trafikverket's regulations (Karlsson & Moritz, 2014).

The aim of the finite element analysis was to find the minimum amount of shear strength ($c'_{u,ref}$) and stiffness (E'_{ref}) required to pass the criteria described in TK GEO 13 (Karlsson & Moritz, 2014). To validate that the settlement requirement was fulfilled the deformations on the top of the embankment was measured. Two additional points were investigated, at the top of the column as well as at the top of the clay layer between two columns. The deformation at these points were not held to the total settlement criteria but rather used to investigate how the stresses were transferred to the columns. All scenarios were subjected to a plain strain safety analysis, using the phi-C reduction method. To maintain the robustness of the numerical model and enhance the comparability of the results, a minimal number of parameter changes were made between simulations. The model remained identical except for variations in strength, stiffness and length of the columns. All simulations were divided into two parts where a normal consolidation simulation was conducted and was followed by a large strain analysis.

To simulate the column, Mohr-Coulomb soil model was selected. The parameters used in Mohr-Coulomb can all easily be derived with basic laboratory tests which was the primary factor for choosing this model (Bentley, 2022). The selected soil model for the clay was the soft soil model. The selection was primarily based on the models simplicity and reliability of predicting large deformations in clayey soils (Karstunen M & Amardeep A, 2017). The embankment was also modelled with the Mohr-Coulomb model.

The parameters for the embankment was taken from Abusharar et al. (2009) and the column parameter were inspired by Tornborg et al. (2021), which can be seen in Table 3.3. The soil parameters used in the analysis were primarily based on the Västlänken project, more specifically the section which is located by the central station in Gothenburg. The parameters which were achieved during testing of that clay was presented in Bozkurt et al. (2023). Parameters which were not presented in Bozkurt's report were instead inspired by soil tests conducted on similar clay from adjacent areas (Tornborg et al., 2021).

Once the soil parameters, which can be seen in Table 3.4, were inserted into the model an iterative process followed. This process intended to investigate the lowest required shear strength and stiffness of the columns while satisfying the previously mentioned criteria. This was done separately for each of the column types and lengths described earlier. The results achieved in the iterative process were then validated by performing a mesh sensitivity analysis where the results were seen to converge.

Table 3.3
Parameters used for the Embankment and Columns.

Parameter	Unit	Embankment	Column
Model	-	Mohr-Coulomb	Mohr-Coulomb
Drainage type	-	Drained	Undrained B
γ	kN/m^3	20	17
E'_{ref}	MN/m^2	8000	a
ν	-	0.3	0.3
$c'_{u,ref} = s_{u,ref}$	kN/m^2	1	a
φ	deg	30	-
$k_v = k_h$	m/day	$9 \cdot 10^{-3}$	$0.08 \cdot 10^{-3}$

Note: a - These values were iterated.

Table 3.4
Soil parameters used in FE-analysis.

Parameter	Unit	Clay
Model	-	Soft soil
Drainage type	-	Undrained A
γ	kN/m^3	16
e_0	-	1.93
λ^*	-	0.083
κ^*	-	$6.8 \cdot 10^{-3}$
ν_{ur}	-	0.2
C'_{ref}	kN/m^2	2
φ'	*deg*	26.7
K_0^{NC}	-	0.55
$k_v = k_h$	m/day	$8.12 \cdot 10^{-5}$
C_k	-	0.308
$K_{0,x} = K_{0,z}$	-	0.552
OCR	-	1.2

Note: The table shows the parameters as well as the corresponding units and values used for the clay layer in the FE-analysis.

3.2 Laboratory work

The parameters which were achieved in the FE-analysis acted as a lower boundary which the experimental parameters were held against. To be able to compare the two, some modification was done. The derived shear strength is half of that of the compressive strength of a soil which was the characteristic investigated with the performed UCS test. Therefore the achieved $C_{u,ref}$ was doubled in order to accurately represent the required compressive strength (q_u). The composition of the binder mixtures examined were decided in consultation with supervisors, based on the findings in the literature review. When the chemical reactions, which the cementitious processes rely on, were analysed some key aspects were found. A binder should have partly consisted of a product rich in calcium-oxides, such as lime, and a material containing aluminosilicate minerals. In order to achieve a binder as environmentally friendly as possible, a calcined clay containing aluminosilicates which required as little refinement to produce as possible was considered.

Four different binder-mixtures were tested and are visible in Table 3.5. When designing the mixture ratios both mixture 1 (M1) and mixture 2 (M2) were based on preliminary studies, that alluded towards a high potential for similar binder ratios. Since cement itself contains the required components for a cementitious process, the impact of adding calcined clay would have been difficult to determine. On the contrary, lime is not a composite material, and thus, it would not have a significant impact on structural integrity by itself. Therefore, a significant change would suggest that calcined clay had played a crucial part in the reactions that have occurred. Mixture 4 (M4) was used as a reference and to allow for more comparisons with a binder similar to the industry standard. The clay used in the mixtures was gathered from Kärra, located in the northern parts of Gothenburg. The clay was collected at 4 to 5 meters depth and the in-situ parameters are shown in Table 3.6. Most clay from the Gothenburg area have similar properties and can generally be classified as silty-clay.

Table 3.5
Ratios and binder contents of the used mixtures.

Mixture lable	BC [kg/m^3]	Clay [g]	L [g]	CC [g]	C [g]
M_1D_x	102.5	2000	104	26	0
M_2D_x	82.0	2100	87.5	21.5	0
M_3D_x	82.2	2000	10.5	93.5	0
M_4D_x	82.2	2000	52	0	52

Note: The table shows the used binder content (BC), Lime (L), Calcined clay (CC) and Cement (C). The different mixtures are labelled from one to four with a number after the the letter D to indicate the number of curing days prior to testing for each sample.

Table 3.6
In-situ parameters for Kärra-clay.

WC [%]	OC [%]	γ [kN/m^3]	Cu [kPa]	S [-]	LL [%]	PL[%]
88	6.9	15.3	10.4	11.5	29	86

Note: WC denotes the soil water content, OC represents the organic content, S stands for the sensitivity, LL and PL are the Atterberg limits.

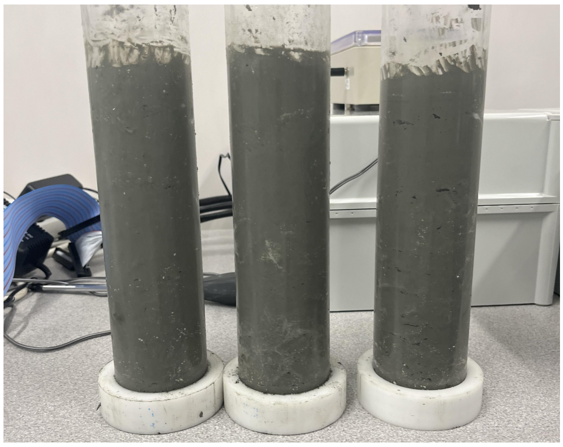
The preparation of the composite clay was carried out by mixing binders with clay until no major inconsistencies were present. The binder was added in portions of a third at a time where the total mixing time was around four minutes. Figure 3.3 shows the result from the mixing process.

The sample tubes were prepared as visualised in Figure 3.4. The composite clay was then carefully compacted with a brass rod until minimum amount of air pockets were present. Some cracks and air pockets could however not be removed which can be seen in Figure 3.3. To simulate the behaviour of the composite clay further down in the column, a static load of 20 kPa was applied during the entire curing process. Two of the samples, $M_4D_{14}^a$ and $M_4D_{28}^a$, were cured without static load applied. This was done to simulate the difference in structural integrity between different depths in the columns. M_2D_x, M_3D_x and M_4D_x were prepared in a way that two separate samples could be extracted from the same sample tube. In addition to the samples used for UCS-test, samples for indentation test were prepared. The indentation test samples were 20 millimetres thick discs with a diameter of 55 millimetres.

During curing, the sample tubes were stored in a container filled with water at room temperature. This ensured that the composite clay was close to fully saturated during the period when the majority of the cementitious processes occurred. Once a sample tube had completed its specific hardening period it was removed from the container and used for two different testing methods.

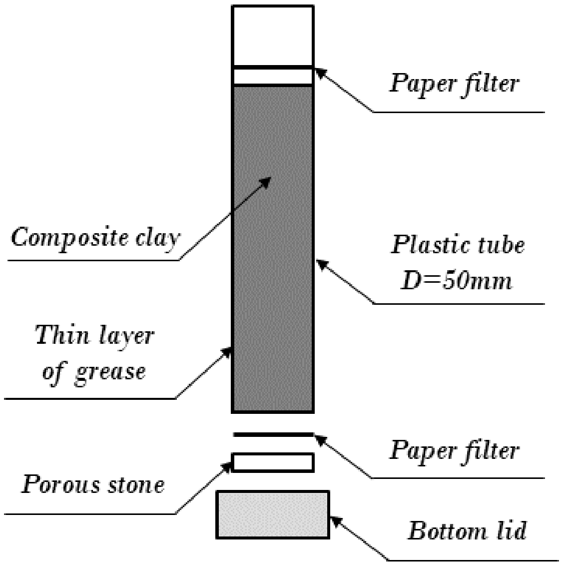
Figure 3.3
Gothenburg clay mixed with the M1 binder.

(a) Composite clay.



(b) Prepared sample tubes.

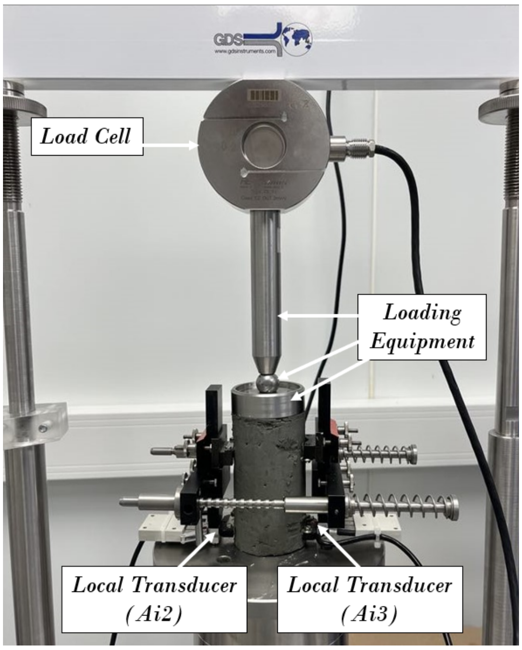
Figure 3.4
Composition of the sample tubes.



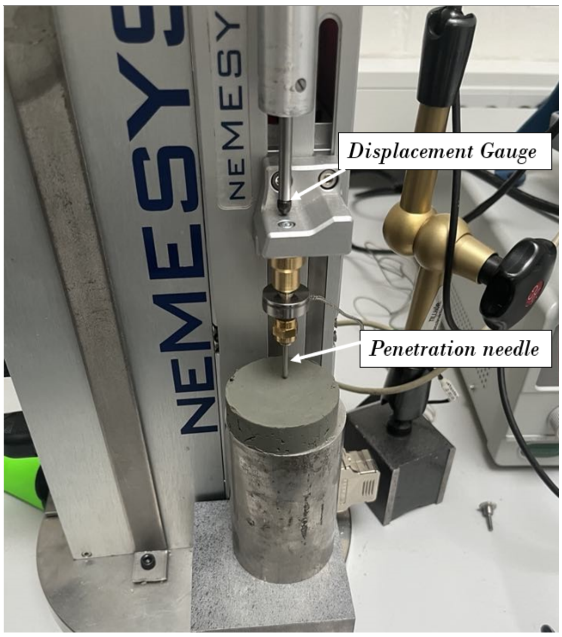
The UCS-test programme is described in Table 3.7. When the composite material was extracted, samples were cut and smoothed out on each end. This was done to ensure that the force was applied axially and to minimise seating load errors. In addition, water content measurements were taken from each sample tube which are presented in Table 3.7. The UCS-tests were performed with equipment from GDS-instruments listed in Table 3.8. During all UCS-tests local strain transducers were attached to the samples, which can be seen in Figure 3.5. This was done to measure the local strains and derive the local secant stiffness. The UCS-tests were the main source of data with the possibility of deriving the unconfined compressive strength. Measurements of global stiffness as well as local secant stiffness stiffness were derived from the UCS-test as well.

The performed indentation tests intended to validate the UCS-test results. The indentation test gave an indication on how the resistance to deformation was related between samples. This was done to objectively determine if the UCS were reasonable in relation to previously performed analysis. In addition to the indentation test listed in Table 3.7, one additional indentation test was performed M_1D_{42}. M_1D_{42} was mainly used to investigate the time frame of curing and if the sample still showed significant improvements. The setup for the indentation test can be seen in Figure 3.5.

Figure 3.5
Laboratory test setup.



(a) UCS test setup.



(b) Indentation test setup.

Table 3.7
Laboratory test programme.

Sample	H [mm]	D [mm]	m [g]	WC [%]	LR [mm/min]	IT [Y/N]
M_1D_7	92.5	50	273	41.14	0.0925	Y
M_1D_{14}	104	50	305	42.6	0.104	Y
M_1D_{28}	100	50	294	42.8	0.1	Y
M_2D_7	110	50	321	43	0.109	Y
$M_2D_7^a$	112	50	328	43	0.112	N
M_2D_{14}	100	49.5	298	42.6	0.1	Y
$M_2D_{14}^a$	100	50	298	42.5	0.1	N
M_2D_{28}	100	49.2	292.7	43	0.1	Y
$M_2D_{28}^a$	100	49.2	292.4	43	0.1	N
M_3D_7	100	50	287	42.7	0.1	Y
$M_3D_7^a$	106	49	312	42.7	0.106	N
M_3D_{14}	100	49	293.9	43.5	0.1	Y
$M_3D_{14}^a$	100	49.5	296.9	43.5	0.1	N
M_3D_{28}	100	49.5	293.2	40	0.1	Y
$M_3D_{28}^a$	100	49.5	297	40	0.1	N
M_4D_{14}	100	50	295.5	41.8	0.1	Y
$M_4D_{14}^a$	100	49.5	296.3	41.8	0.1	N
M_4D_{14}	100	49.5	296.3	41.8	0.1	N
$M_4D_{14}(2)$	100	49.5	293.5	44.2	0.1	N
M_4D_{28}	100	49.5	293.8	42	0.1	Y
$M_4D_{28}^a$	100	49.5	293.7	43	0.1	N
$M_4D_{28}(2)$	100	49.6	297	43	0.1	N

Note: LR-Loading rate.

IT-indentation test.

(2)-*Samples not stored under static loading.*

a- Second sample from the same tube.

Table 3.8
Laboratory equipment.

Name	Purpose	Specification	Resolution	Tolerance
GDS Load Frame	Load frame for UCS/testing	Max capacity: 10 kN	0.1mm	Axial load: 0.1% Displacement: 0.2%
VETEK-TS CTS821TC25	Cell attached to Load frame which measures the load applied to the sample	Max capacity: 10 kN	$1.1\frac{mV}{V}$	$\leq \pm 0.023\%$
Local strain transducers	Attached to the sample and measures the local strain.	Range: $\pm 2.5mm$	$1\frac{mV}{V}$	0.5%

3.3 Carbon emissions

The final part of the project was to perform a CO_2-eq calculation. This was done to evaluate how the different mixtures would affect the environmental impact in the hypothetical scenarios described earlier. As previously mentioned, the dimensioning of the columns were not optimised to each scenario. This implies that different spacings, diameters and thereby number of columns were not evaluated. The CO_2-eq calculations were therefore performed with this as a limitation to the accuracy. The mixtures which were found to surpass the requirements of strength and stiffness will therefore be based on the same volume of treated clay.

In the total CO_2-eq calculations, the emission data for each material presented in Chapter 2 was used. These included all the processes needed in producing the materials but excluded emissions from transportation. No further investigation on the emissions was done regarding aspects such as which source of energy was used during production.

4 Results and Discussion

The results achieved in the FE-analysis and during the laboratory testing will be presented and discussed. The raw data which the presented results are based on is included in the appendices. The global stress-strain graphs can be seen in appendix C and the local stress-strain graphs are presented in appendix D.

4.1 FE-analysis

The results achieved during the FE-analysis are presented in Table 4.1. The full table, including the results from the large strain analysis, can be seen in appendix A. When analysing floating columns with a length of 10 meters, no realistic solution was found. Thus, this specific scenario was excluded since it would not significantly contribute to the research to include unrealistic scenarios. The most critical scenario, in regards to required strength and stiffness, was found to be the floating columns with a length of 15 meters.

A result which meets the criteria can be achieved by several combinations of strength and stiffness. However, to find all satisfactory combinations of values was outside the scope of this paper. In addition, it was not prioritised to achieve settlements equal to the criteria by optimising the parameters but rather achieve results in the vicinity of the set boundaries. This was mainly due to the inherent uncertainties associated with the used models. During the FE-analysis it was however found that the strength parameter had a significantly greater impact on the total settlement than the stiffness for both floating and end-bearing columns.

The presented safety factor regarding the stability analysis was achieved immediately following embankment construction and thus during the most critical phase. All investigated scenarios were found to pass the requirement affiliated with safety classification 2.

As can be seen in Table 4.1 the difference in required parameters between the different column lengths is marginal. In order to investigate this further the neutral plane for the different column lengths were evaluated and can be seen in appendix B. In all scenarios the columns were fully mobilised which caused the neutral plane to be shallowly located. The neutral plane in all scenarios was located less than 1 meter below the ground surface. The fully mobilised columns were partly due to the phenomena of soil arching which caused the columns to be the main receiver of stress, see Figure 4.1. As a consequence of shallowly located neutral plane the different column lengths does not significantly affect the total settlements apart from minor effects. With the increasing column length, the friction forces increased and the amount of untreated clay until firm layer decreased. These are the main factors which the small differences in required strength and stiffness between scenarios can be attributed to.

Table 4.1
Required column parameter derived from PLAXIS.

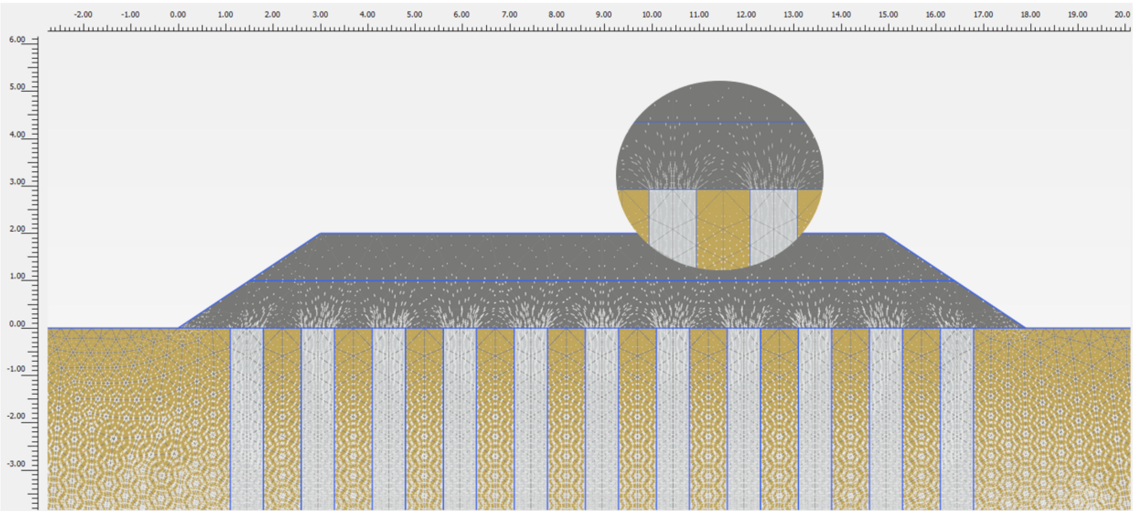
	Length [m]	$C_{u,ref}$ [kPa]	$q_u[kPa]$	E'_{ref} [MPa]	S_{Emb} [cm]	M_{sf} [-]
Floating	15	85	170	50	20	1.9
	20	80	160	50	19.4	1.6
	25	80	160	25	18.9	1.65
End-bearing	10	65	130	15	17.3	1.8
	15	70	140	25	19.1	1.7
	20	75	150	15	19	1.85
	25	75	150	25	17.7	1.78

Note: The results presented in the table were achieved during a Normal analysis.

S_{Emb} *- Settlement of the embankment.*

M_{sf} *- Safety factor.*

Figure 4.1
Visualisation of total principal stress in embankment.



4.2 UCS and indentation-test

In total 21 different UCS tests were carried out and the highest and lowest strength (q_u) are presented in Figure 4.2 and Table 4.2. M1 and M2 showed UCS values comparable to M4 after 28 days of curing. Despite having the same calcined clay to lime ratio, M1 possessed greater strength than M2. This difference can likely be attributed to the higher binder content in M1, which provides more available reactions per amount of clay.

The main outlier in Figure 4.2 is mixture 3 (M3). M3 failed to reach any significant strength. Given the small amount of lime present in M3 the mixture could not initiate enough cementitious reactions to have an impact. When deciding the mixture ratios M3 was hypothesised to react in this way. However, it was still decided to retain the ratio to understand where the extremities lie. Visible in Figure 4.2 and Table 4.2 is the variance between samples extracted from the same tube. The difference in strength is not significant but the majority of the samples taken from the top half performed slightly better. One reason could potentially be due to the bottom half failing to reach appropriate saturation levels. The laboratory work was however planned in way where the samples would be in contact with water as much as possible to minimise these effects. If the samples are not fully saturated there may not be enough water available to react with all the lime to create $Ca(OH)_2$. However, when visually comparing the two halves of the sample there is a noticeable difference, see Figure 4.3. The bottom half of the sample clearly had more air-pockets and cracks when compared to the top half. Suggesting that the top halves were more compacted and thus had fewer weak areas. This would most likely explain the difference in measured strength rather than saturation issues. Worth mentioning is the lack of air pockets and cracks does not necessarily mean that the sample is more representative towards the in-situ columns.

Another distinct trend can be seen in Figure 4.4, where the strength increase is highlighted. Both M1 and M2 exhibit a significant increase in strength with an increasing number of curing days which suggests still ongoing pozzolanic reactions. The strength development seems to progress linearly during the entirety of the curing period for M1 and M2. This trend is further validated by Figure 4.5 where the measured resistance is depicted in relation to the UCS. The relation between the two measurements is quite similar, validating the accuracy of the result.

Figure 4.2
Unconfined compression strength test results.

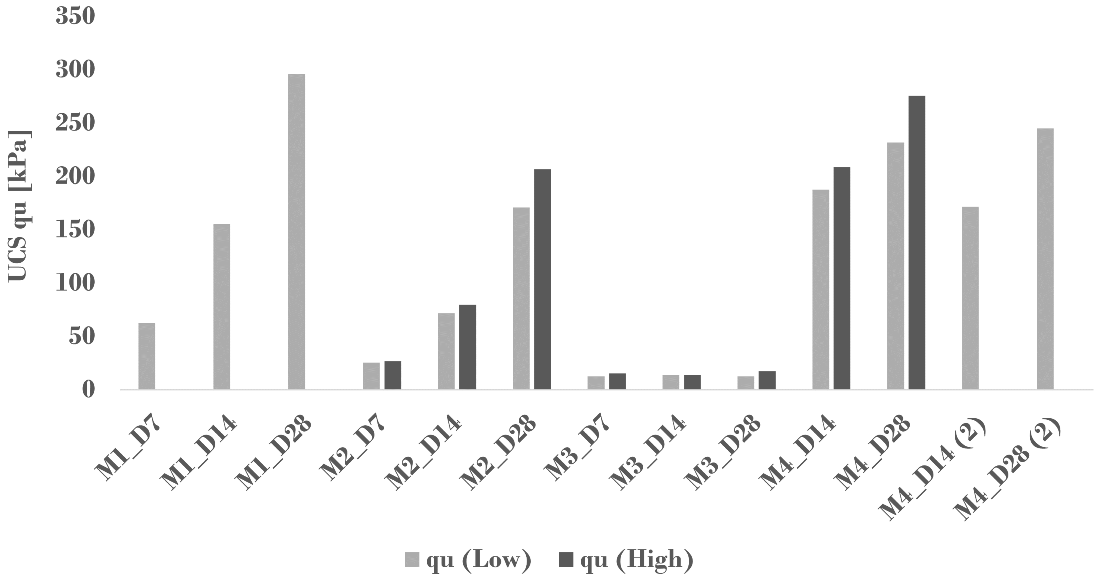


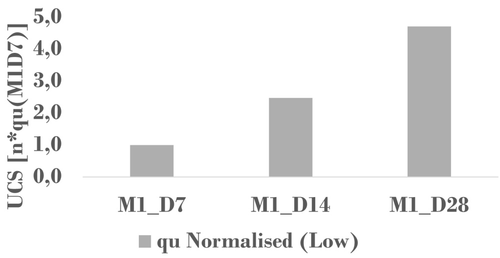
Table 4.2
Tabulated unconfined compression strength test results.

Sample	qu (Low) [kPa]	qu (High) [kPa]	Variance [%]
M_1D_7	63	-	-
M_1D_{14}	156	-	-
M_1D_{28}	296	-	-
M_2D_7	26	27	6
M_2D_{14}	72	80	10
M_2D_{28}	171	207	17
M_3D_7	13	16	18
M_3D_{14}	14	14	0
M_3D_{28}	13	18	28
M_4D_{14}	188	209	10
M_4D_{28}	232	276	16
$M_4D_{14}(2)$	172	-	-
$M_4D_{28}(2)$	245	-	-

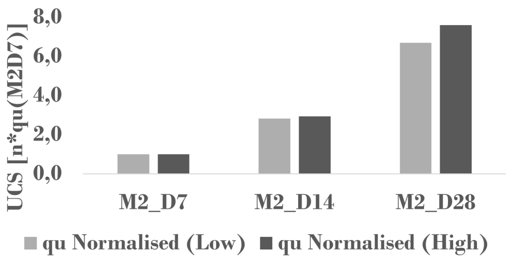
Figure 4.3
Comparison between the top and bottom halves of M_2D_7.

Note:The figure highlights the difference in compaction between the bottom half (left) and top half (right) of the same sample.

Figure 4.4
Normalised UCS results.

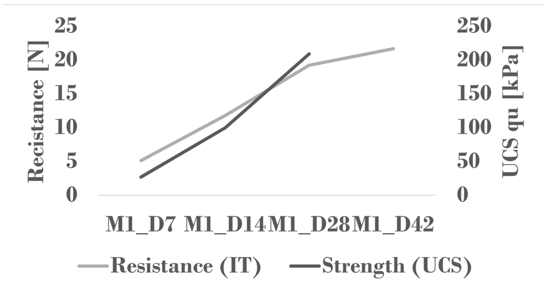


(a) Normalised UCS results for M1.

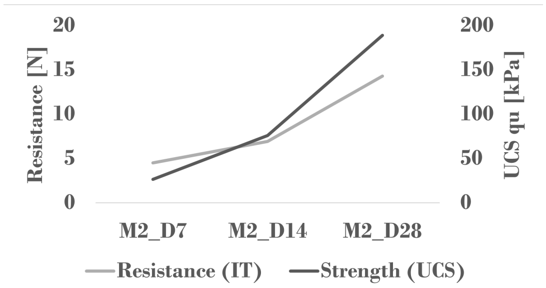


(b) Normalised UCS results for M2.

Figure 4.5
Relation between UCS and Resistance.



(a) Relation between UCS and Resistance for M1.



(b) Relation between UCS and Resistance for M2.

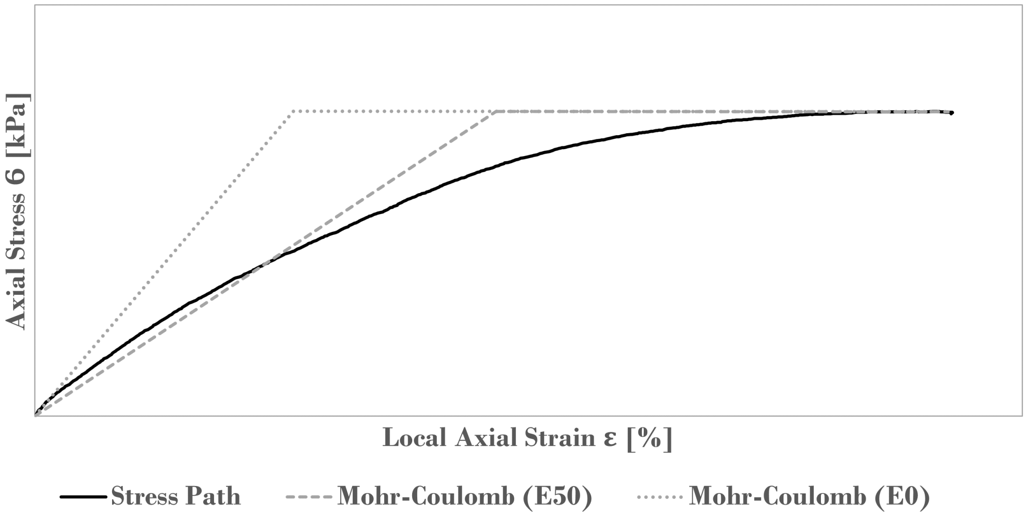
The stiffness of the samples were assumed to be represented by the derived E_{50} from the local strain transducers. This was done with the limitations of the chosen soil model in mind. As earlier described the soil model used for the columns was Mohr-Coulomb which assumes linear-elastic perfectly-plastic behaviour of the soil. The behaviour of the tested samples was neither perfectly linear in the elastic region nor perfectly plastic. In order to best represent the samples with the Mohr-Coulomb model it is recommended to use the derived E_{50} as the stiffness (Józsa, 2011). The difference of using either E_{50} or E_0 as the stiffness in the Mohr-Coulomb soil model is visualised in Figure 4.6.

The achieved local secant stiffness for different samples are presented in Figure 4.7 and Table 4.3. Similarly to the UCS-results, local E_{50} results showed that M3 failed to see any significant increase with time. M1 and M2 show significant increases in local secant stiffness over time. Both M1 and M2 are, at 28 days of curing, comparable to M4. Illustrated in Figure 4.8, the increase in local E_{50} does not show as clear indication to a linear increase as the UCS-results.

The variance between the highest and lowest measured local secant stiffness were quite significant when analysing M2. This is especially true for M_2D_{28} where a roughly 50% variance was measured, see Table 4.3 and Figure 4.7.

In both Figure 4.7 and 4.8 the result for M_1D_7 is not included. The measured local secant stiffness regarding this specific sample was decided to be excluded due to misreadings. The readings indicated on a local E_{50} 10 times greater than the second largest measurement. Several factors could be the issue leading to misreading but it was mainly attributed to faulty equipment which was later rectified.

Figure 4.6
Difference between E_{50} and E_0 when representing E'_{ref}.



Note: The figure shows the difference between using E_{50} *and* E_0 *as the assigned stiffness while working with the Mohr-Coulomb soil model.*

Table 4.3
Tabulated local secant stiffness results.

Sample	E50 (Low) [MPa]	E50 (High) [MPa]	Variance [%]
M_1D_7	-	-	-
M_1D_{14}	66	-	-
M_1D_{28}	133	-	-
M_2D_7	9	14	36
M_2D_{14}	21	33	36
M_2D_{28}	81	150	46
M_3D_7	3	15	81
M_3D_{14}	2	6	63
M_3D_{28}	7	21	67
M_4D_{14}	101	115	12
M_4D_{28}	134	142	6
$M_4D_{14}(2)$	123	-	-
$M_4D_{28}(2)$	238	-	-

Figure 4.7
Local secant stiffness results.

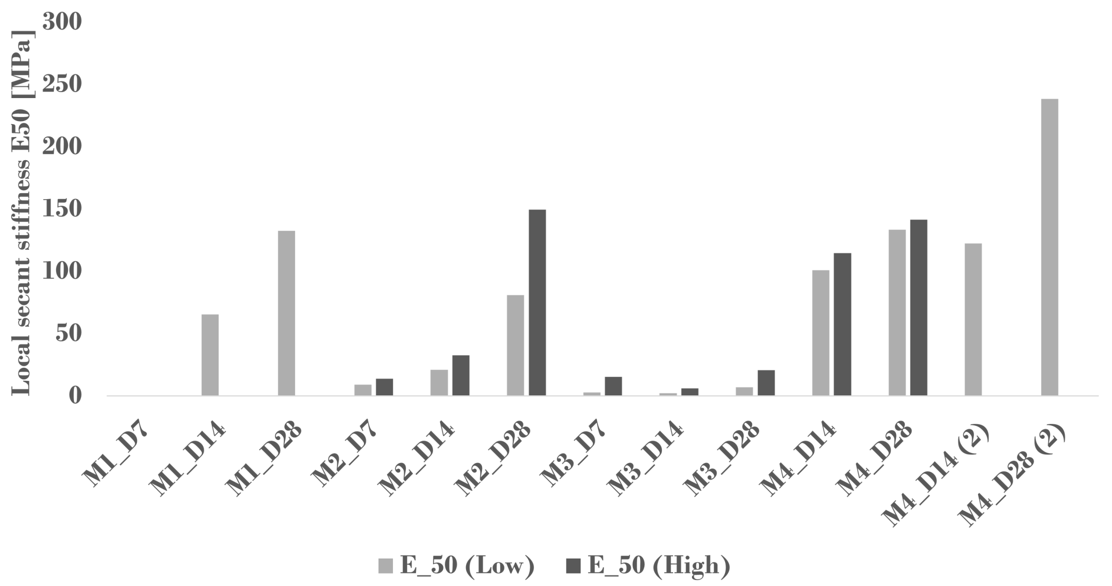
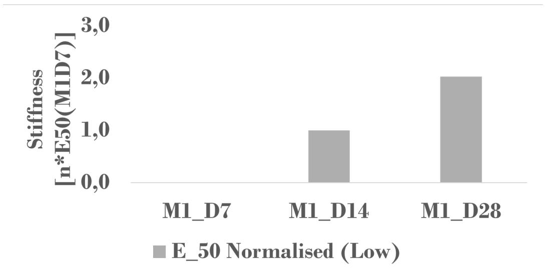
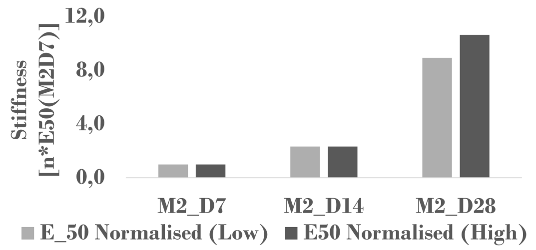


Figure 4.8
Normalised local secant stiffness.



(a) UCS test setup.



(b) Indentation test setup.

Note: The figures display the relation of local secant stiffness results between different curing times. M1 is normalised to the result of a curing time of 14 days while M2 is normalised to the result for 7 days curing time.

Mixture M4(2) performed surprisingly well in relation to M4 which had been cured under static load. Both $M_4D_{14}(2)$ and $M_4D_{28}(2)$ outperformed M_4D_{14} and M_4D_{28} with regards to local secant stiffness. Similar results were seen for UCS although not as exaggerated. There was not enough evidence to attribute the increase in strength and stiffness solely to being cured without static load. $M_4D_x(2)$, M_4D_{14} and M_4D_{28} were all prepared in separate sample tubes which can subconsciously introduce impacting differences. Amount of compaction and heterogeneity in the mixture were two of the differences which could greatly impact the result. In addition, the difference could potentially be a result of how the load is applied rather than a direct consequence of the load itself. The load was applied during curing in a way which necessitated a cap to be used on the top of the sample tube. This cap allowed water flow through small holes. However, it is not unreasonable to assume that the sample's access to water was somewhat compromised due to the cap. This could have affected the amount of pozzolanic reactions which occurred and thereby the structural integrity.

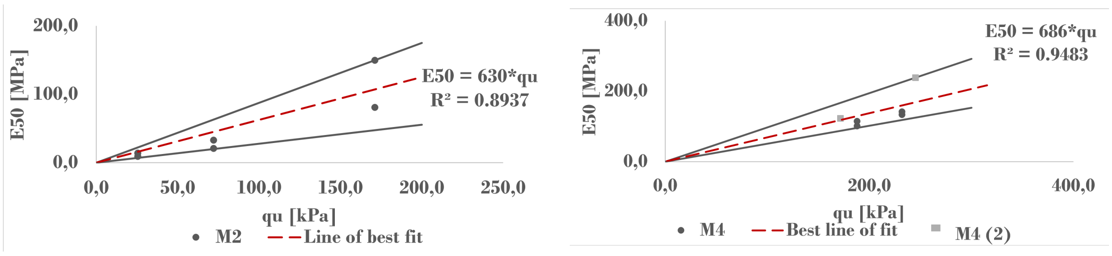
There were some apparent issues regarding the equipment used during laboratory testing. The contact between the transducers and the samples caused indentations on the softer samples. For stiffer samples the same issue was not encountered. This is something which can be seen in the results where the stress-strain graphs indicated problems with accurate readings. Some of the local stress-strain tests did not produce a graph which behaves in an expected way, which can be seen in appendix D. In addition, the two transducers measuring during the same test should have showed similar results, which was not always the case. The issues with ambiguous readings relates to difficulties in getting an accurate measurement of the local stiffness of the sample.

The proposed calcined clay binder, as previously mentioned, achieved a UCS and local secant stiffness which were comparable to the values measured for M4. However, in Figure 4.9 the variance in the results is visible. Since M1 only had one sample per curing time, the variance was not possible to derive. As could be expected, M4 had a higher R-squared value, indicating a smaller variance. On the contrary, the best line of fit is nearly identical for both mixtures. This suggests that roughly the same amount of strength increase results in approximately the same local E_{50} increase.

Figure 4.10 illustrates the lowest measured UCS and local secant stiffness values for all tested samples. Furthermore, the most critical requirements found during the FE-analysis, 15 meters floating columns, are displayed as well. M3 failed to reach the requirements. Additionally, all mixtures required at least 14 days of curing to pass the criteria. Interestingly the best performing samples were the M_1D_{28} and $M_4D_{28}(2)$ samples.

Figure 4.9

Relation between local secant stiffness and UCS.



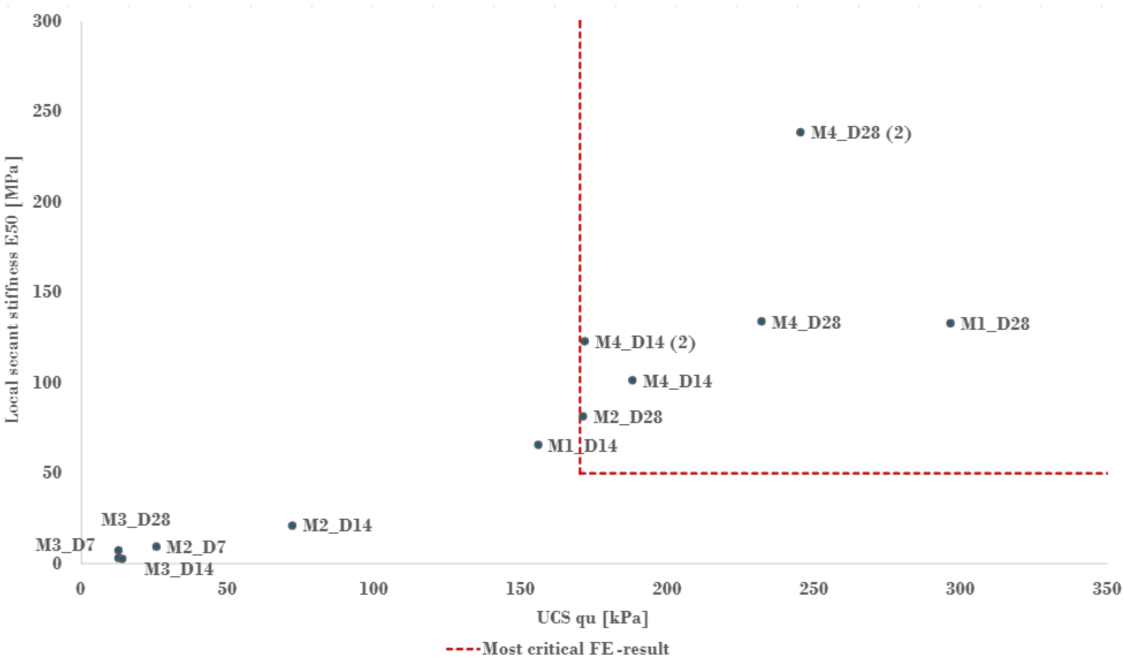
(a) Relation for M2.

(b) Relation for M4.

Note: The best line of fit, largest and smallest relation as well as the coefficient of determination are displayed for each mixture.

Figure 4.10

Comparison of laboratory and FE-analysis results.



4.3 Carbon emission results

The weight of each material needed in the binders for the different column lengths are presented in Table 4.4. The weights are presented as a theoretical $tons/meter_{embankment}$. As can be seen in Table 4.5, M2 had a reduction of total CO_2 emissions of around 11% in relation to M4. As previously mentioned, these numbers should be seen as an indication of the possible effect rather than accurately representative. This is due to the small amount of optimising which was done for each scenario. Since M3 did not surpass the criteria found in the FE-analysis, this mixture was not included in the CO_2 calculations. M1 resulted in an increase in emission which can be attributed to the difference in total amount of binder used in this mixture. There are several aspects of the total CO_2 emission, such as transportation, which are difficult to estimate in a hypothetical scenario. This would impact the specific reduction/increase of emission since it was not included in this calculation. Additionally, the data which was used to represent the amount of emission attributed to producing the material affect the result significantly.

Table 4.4
Amount of needed material per meter embankment measured in tons.

Column length	Mixture	Lime [ton]	Cement [ton]	Calcined clay [ton]
10 m	M1	2.26	0	0.56
	M2	1.81	0	0.45
	M3	0.23	0	2.03
	M4	1.13	1.13	0
15 m	M1	3.39	0	0.85
	M2	2.71	0	0.68
	M3	0.34	0	3.05
	M4	1.69	1.69	0
20 m	M1	4.52	0	1.13
	M2	3.61	0	0.9
	M3	0.45	0	4.06
	M4	2.26	2.26	0
25 m	M1	5.64	0	1.41
	M2	4.52	0	1.13
	M3	0.56	0	5.08
	M4	2.82	2.82	0

Table 4.5
CO_2 calculation results.

Mixture	CO_{2Lime} [kg]	$CO_{2Cement}$ [kg]	CO_{2CC} [kg]	CO_{2Total} [kg]	Difference [%]
M1	3340	0	80	3420	+12%
M2	2670	0	60	2740	−11%
M4	1670	1400	0	3070	±0%

5 Conclusion and recommendations

In conclusion, the implementation of calcined clay in DDM seems promising. The results from this study indicated to a theoretically working method for such implementation. The lower boundary requirements achieved during the FE-analysis were surpassed by multiple different samples. M1 and M2 surpassed the requirements for the most critical scenario investigated, at the end of the 28 days curing time. Mixture M1 consisted of 80 % lime and 20 % calcined clay with a binder content of 100 $\frac{kg_{binder}}{m^3_{clay}}$, where M2 utilised the same ratio with a binder content of 80 $\frac{kg_{binder}}{m^3_{clay}}$. M1 and M2 reached a UCS, after 28 days of curing, of 297 kPa and 171-207 kPa respectively. As for the local secant stiffness M1 and M2 achieved 133 MPa and 81-150 MPa respectively. This was promising as the critical strength found in the FE-analysis was 170 kPa with a stiffness of 50 MPa. In addition to a seemingly working concept, the CO_2 calculation indicated that this would result in a reduction in emission for mixture M2 of approximately 11

M3 which consisted of 90 % calcined clay and 10 % lime with a binder content of 80 $\frac{kg_{binder}}{m^3_{clay}}$ did not experienced significant improvements. The results of UCS and stiffness were similar throughout the entire curing time. This indicated on towards a mixture where not enough pozzolanic reactions had taken place to contribute to the structural integrity.

Although the UCS and stiffness of M1 and M2 were comparable with a lime-cement mixture, an indication of larger variance was seen. The variance between the highest and lowest measurements of UCS and stiffness for M2 was 17 % and 46 % respectively. In comparison, the lime-cement mixture showed a variance in UCS and stiffness of 16 % and 6 % respectively. With the small margin to the required strength, this causes unwanted uncertainties. It should however be mentioned that the small data pool of the study could have compromised the accuracy of the variance.

Results achieved in this study indicated on a slower strength development for binders which contained calcined clay, rather than lime-cement. The comparison between the two, with a curing time of 14 days, showed that M4 had obtained significantly higher strength and stiffness than M1 and M2. However, the mixtures were similar after 28 days of curing. Additionally, the indentation test on M_1D_{42} indicated a still ongoing strength development.

The promising results achieved lends itself to conducting further research in the topic. There are several aspects which should be researched further in combination with evaluating the extremities. The data pool regarding the inclusion of calcined clay is still too small to draw any reliable conclusions on the viability. Numerous tests should be conducted to minimise the uncertainties. M1 and M2 seemed promising and should be further researched in terms of binder content as well as replacing the lime with cement. This could, if similar results are achieved, further lower the carbon emission. Furthermore, changes in ratios of which the binder consists of should be investigated with regards to optimising the achieved strength and stiffness. This is to investigate at what ratio of materials all the added lime/cement reacts with all added calcined clay. This study concluded that replacing 90 % of the binder with calcined clay did not react in an effective manner. The maximum amount of binder which can be replaced with calcined clay while still producing sufficient amount of pozzolanic reactions, should be evaluated. Finally, we recommend to more closely evaluate the time frame of significant strength development. Where a comparison between a traditionally used binder and binders similar to M1 and M2, should be made. This is to investigate the magnitude of strength and stiffness the binders converge towards over time.

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A

Tabulated results from FE-analysis.

	Length [m]	Type	$C_{u,ref}$ [kPa]	E'_{ref} [MPa]	S_{Emb} [cm]	M_{sf} [-]
Floating	10	Normal	-	-	-	-
	10	Large strain	-	-	-	-
	15	Normal	85	50	20	1.9
	15	Large strain	85	50	17.7	1.9
	20	Normal	80	50	19.4	1.6
	20	Large strain	80	50	16.2	1.6
	25	Normal	80	25	18.9	1.65
	25	Large strain	80	25	17.2	1.65
End-bearing	10	Normal	65	15	17.3	1.8
	10	Large strain	65	15	18.3	1.8
	15	Normal	70	25	19.1	1.7
	15	Large strain	70	25	16.3	1.7
	20	Normal	75	15	19	1.85
	20	Large strain	75	15	17.3	1.85
	25	Normal	75	25	17.7	1.78
	25	Large strain	75	25	15.9	1.78

Note: The table shows the strength and stiffness of the columns following the iteration process as well as the correlating settlements of the embankment. In addition, the minimum safety factor achieved during the simulation is presented. The parameters are presented for each of the column lengths with a normal and large strain analysis for both floating and end-bearing columns.

B

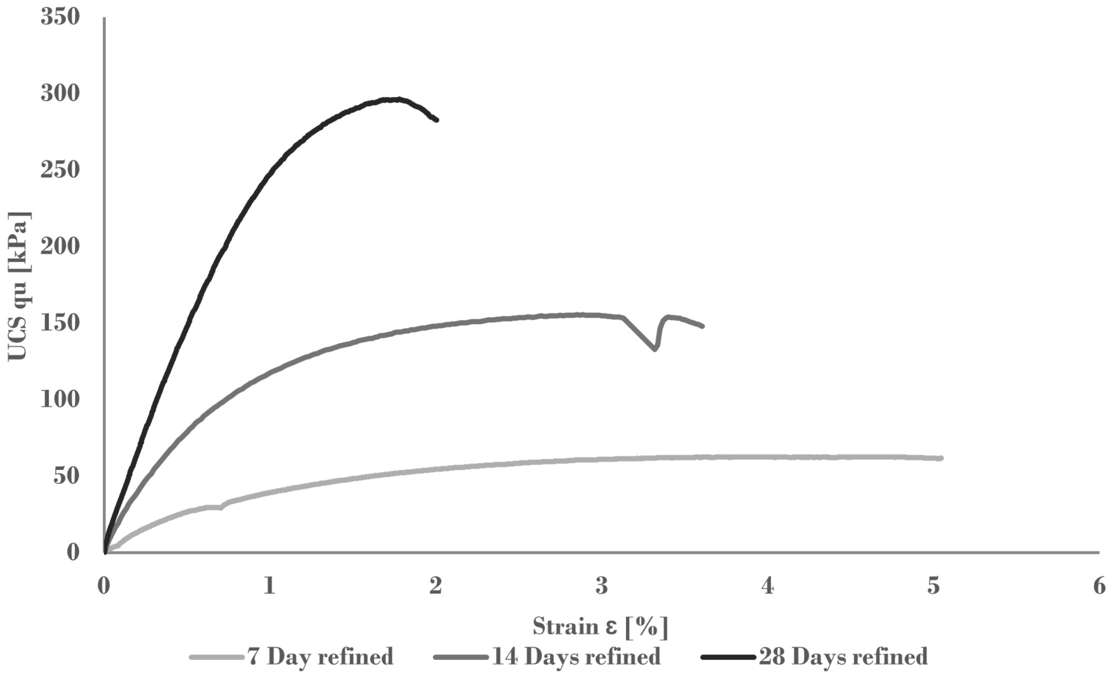
Shear stress over column length.

Note: The figure is taken from the scenario with 25 meter long floating columns. Shear stress over the column length for deriving depth of neutral plane.

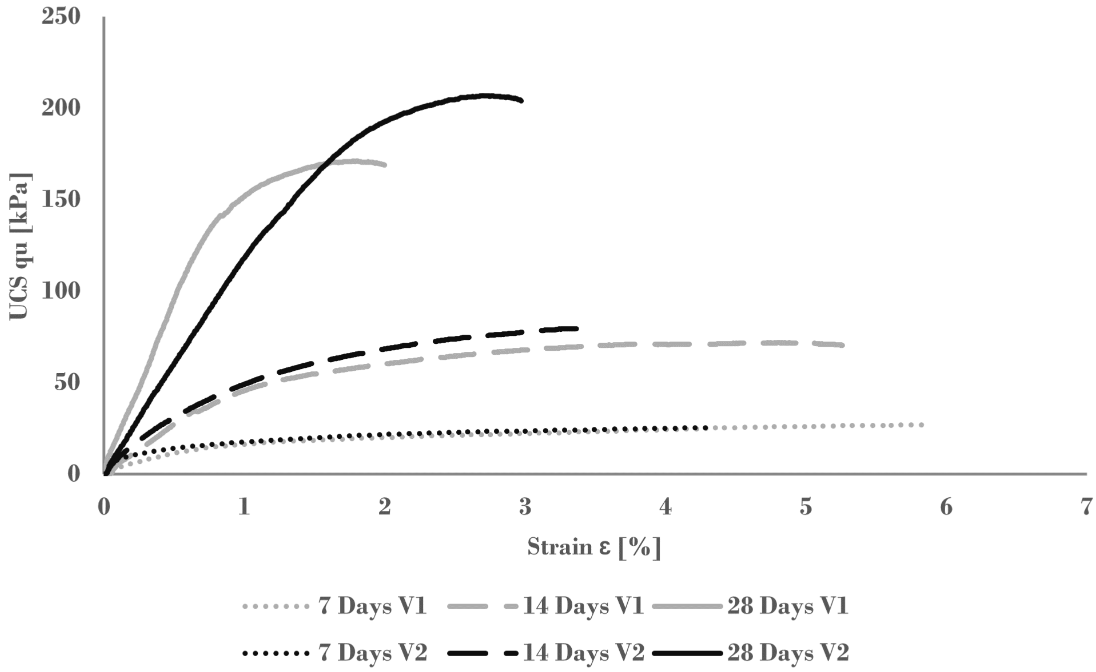
C

Global Stress-Strain graphs

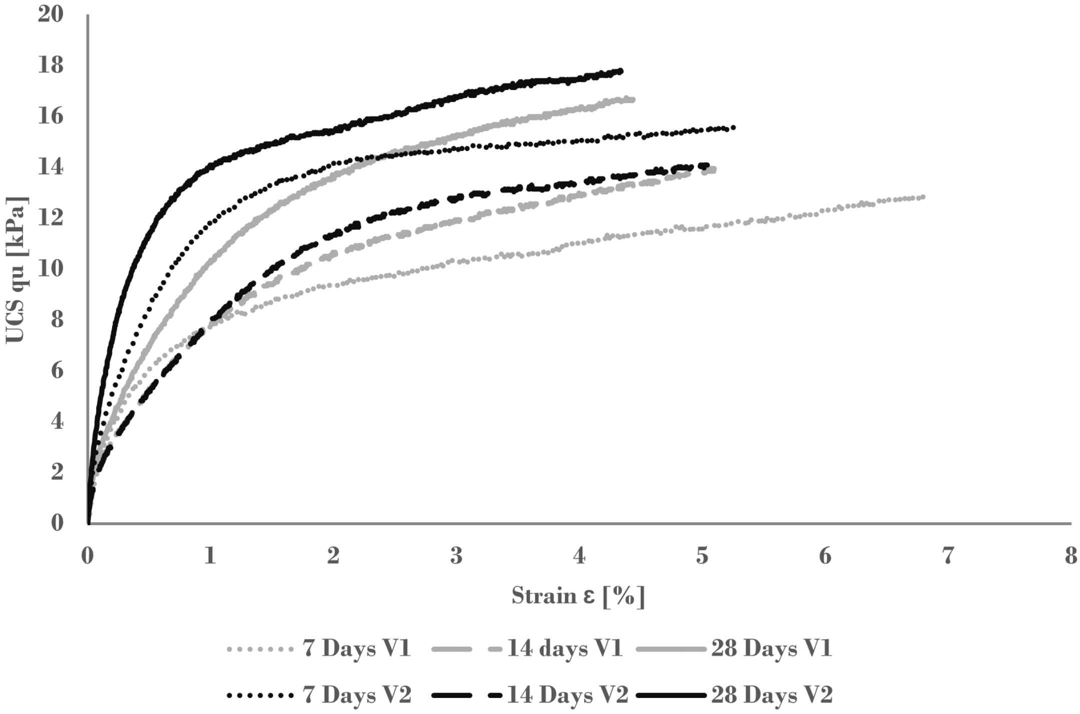
C.1 Mixture M1



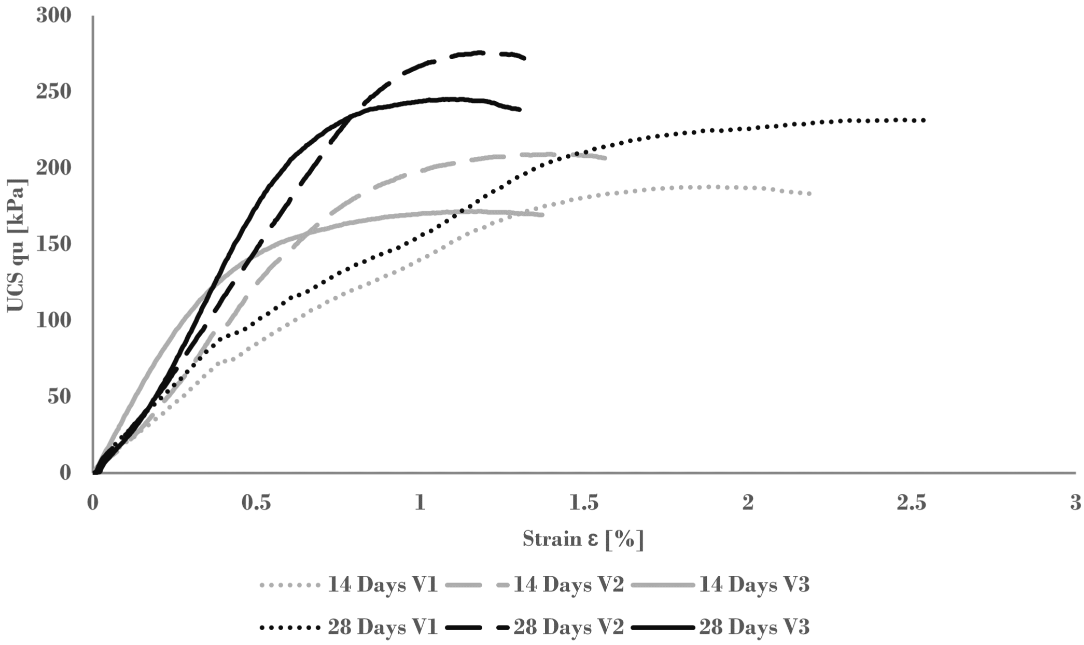
C.2 Mixture M2



C.3 Mixture M3



C.4 Mixture M4

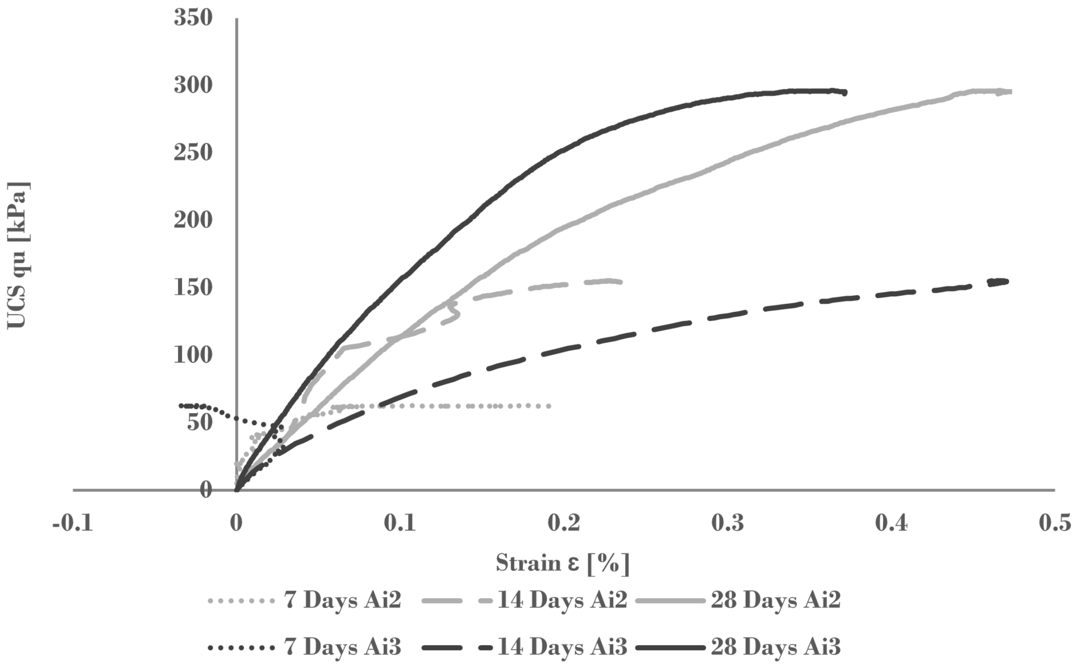


D

Local Stress-Strain graphs

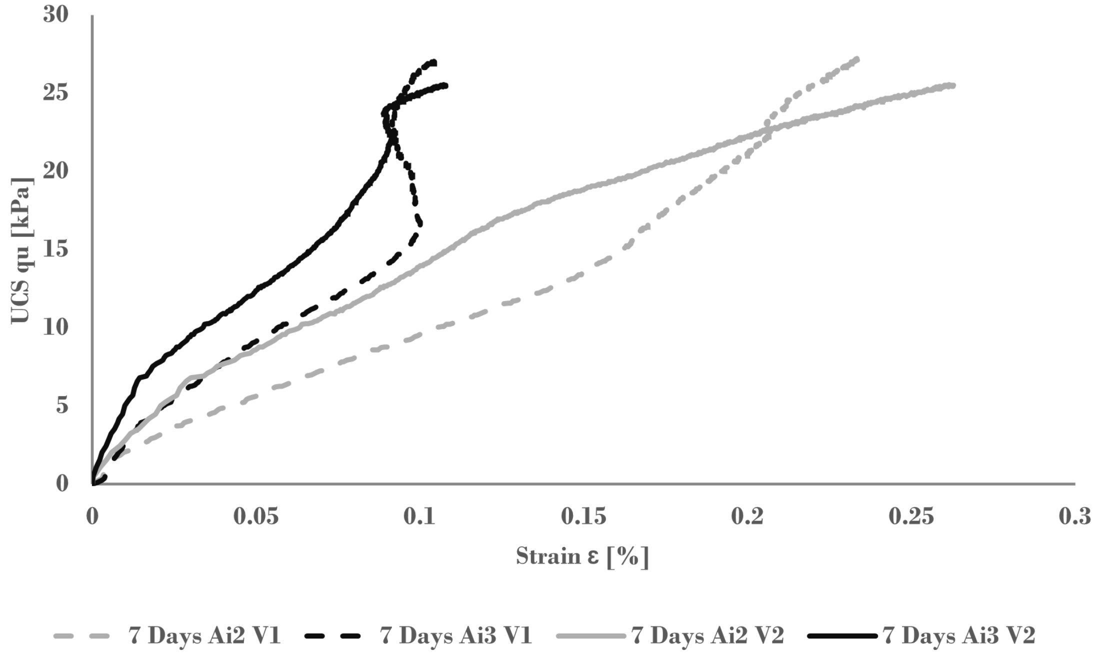
V1 references the bottom half of the sample tube and V2 the top half.

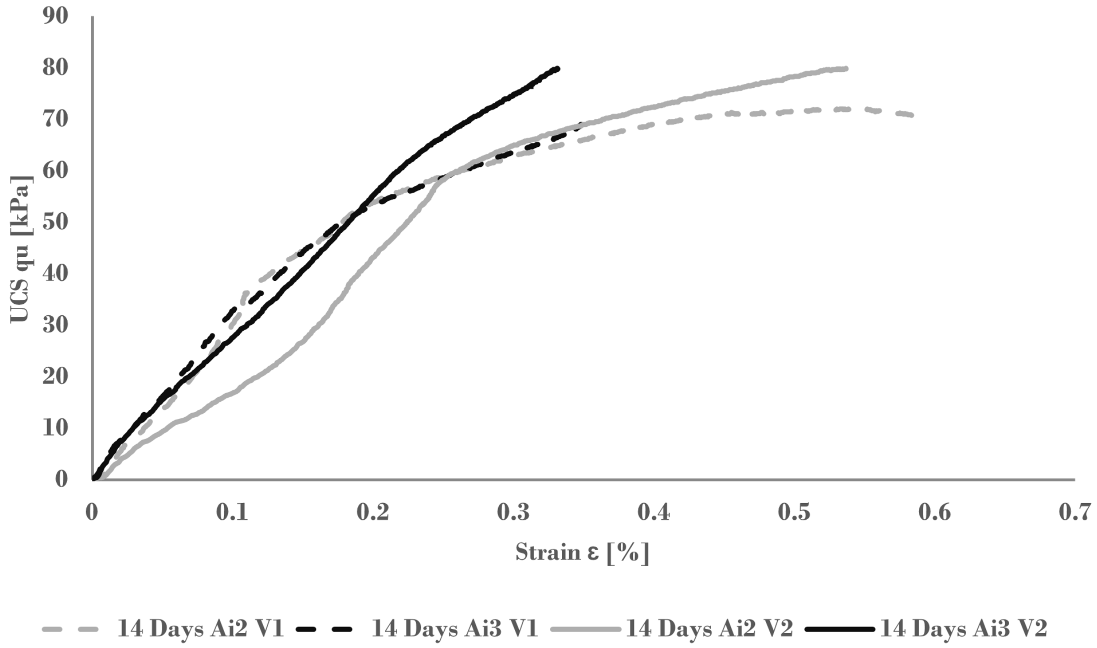
D.1 Mixture M1

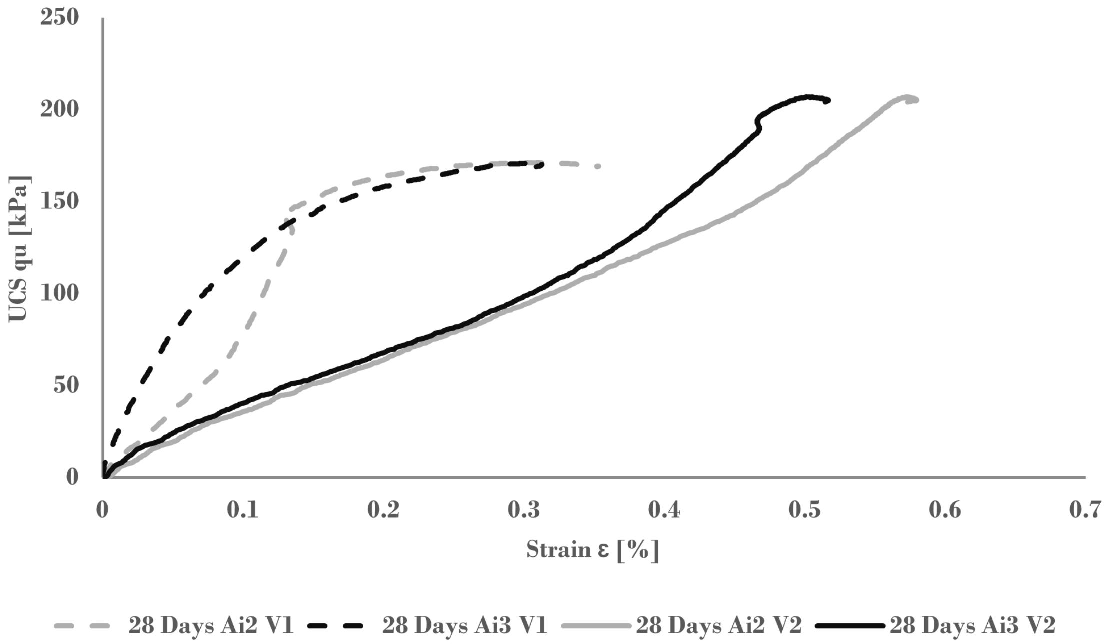


D.2 Mixture M2

V1 references the bottom half of the sample tube and V2 the top half.

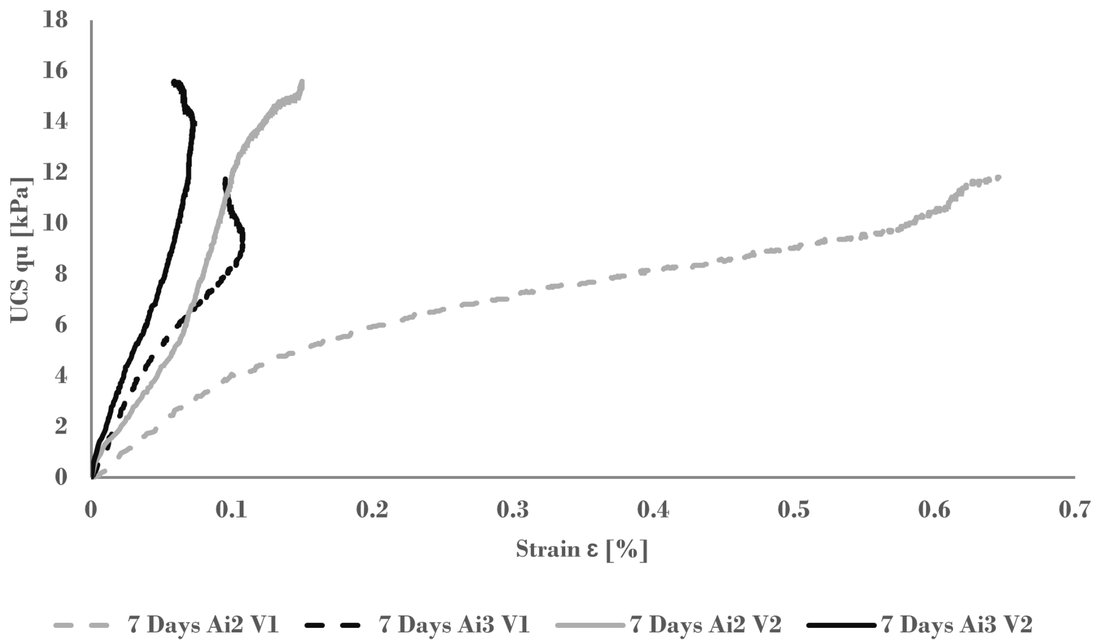


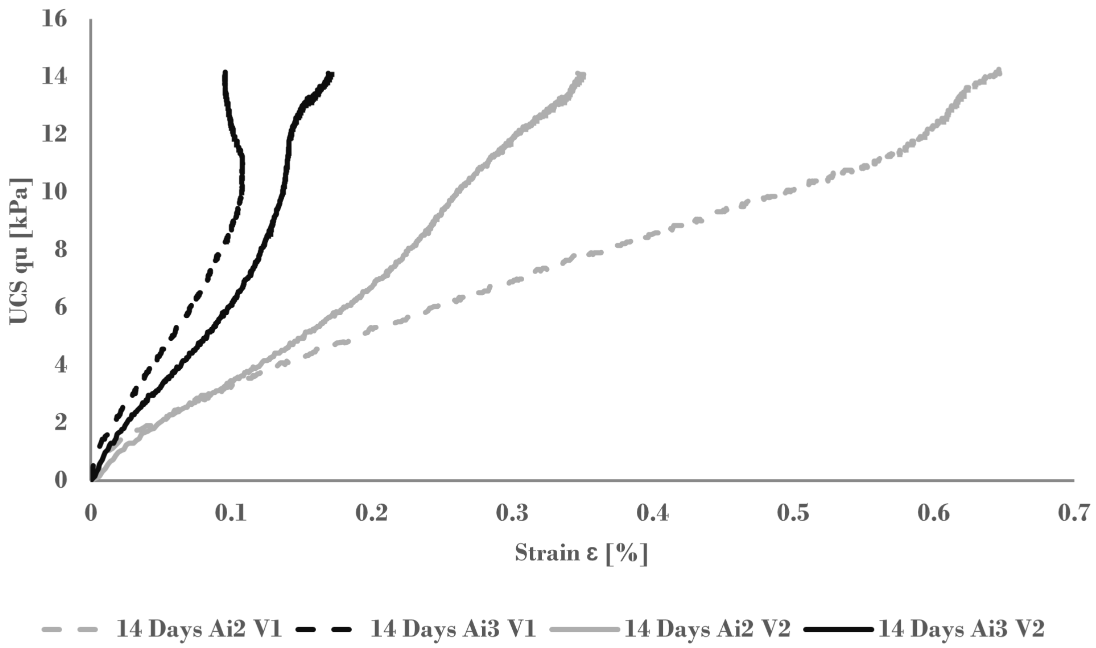


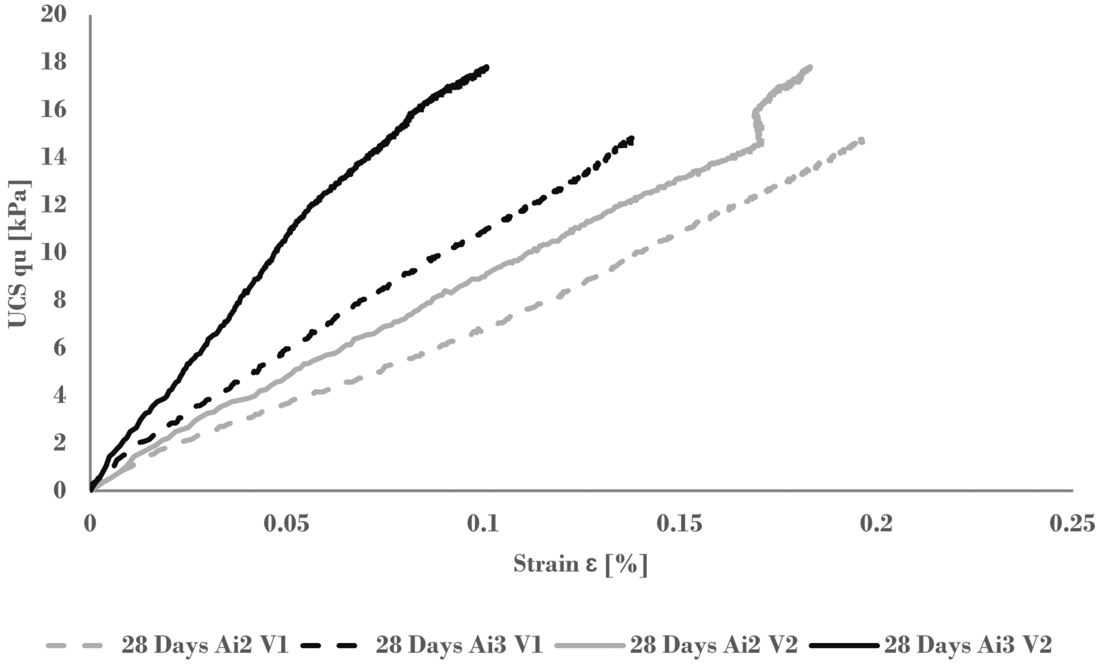

250
200
150
100
50
0
UCS qu [kPa]
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
Strain ε [%]
28 Days Ai2 V1
28 Days Ai3 V1
28 Days Ai2 V2
28 Days Ai3 V2

D.3 Mixture M3

V1 references the bottom half of the sample tube and V2 the top half.

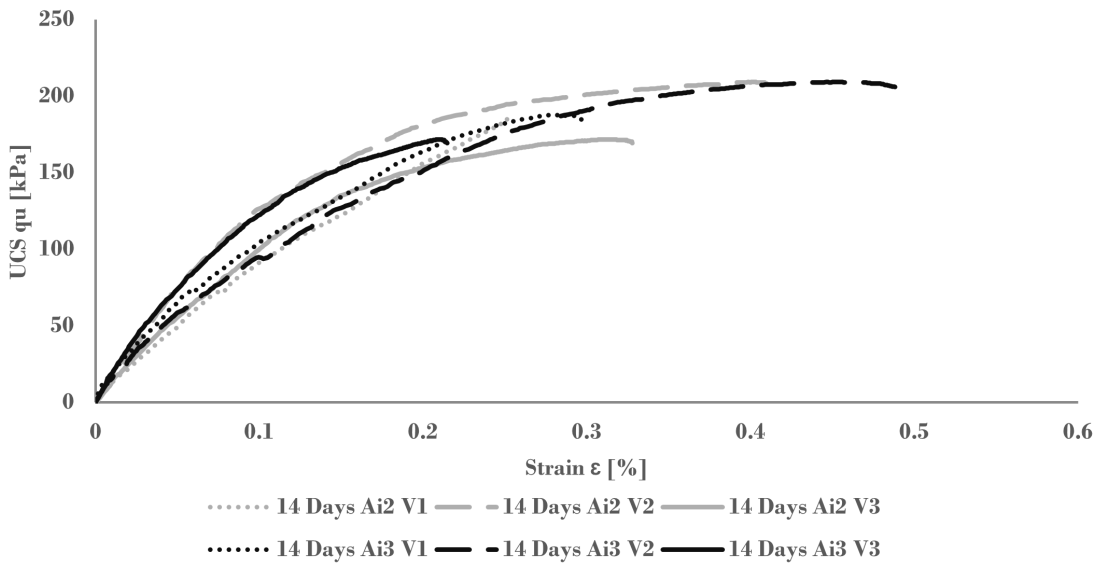


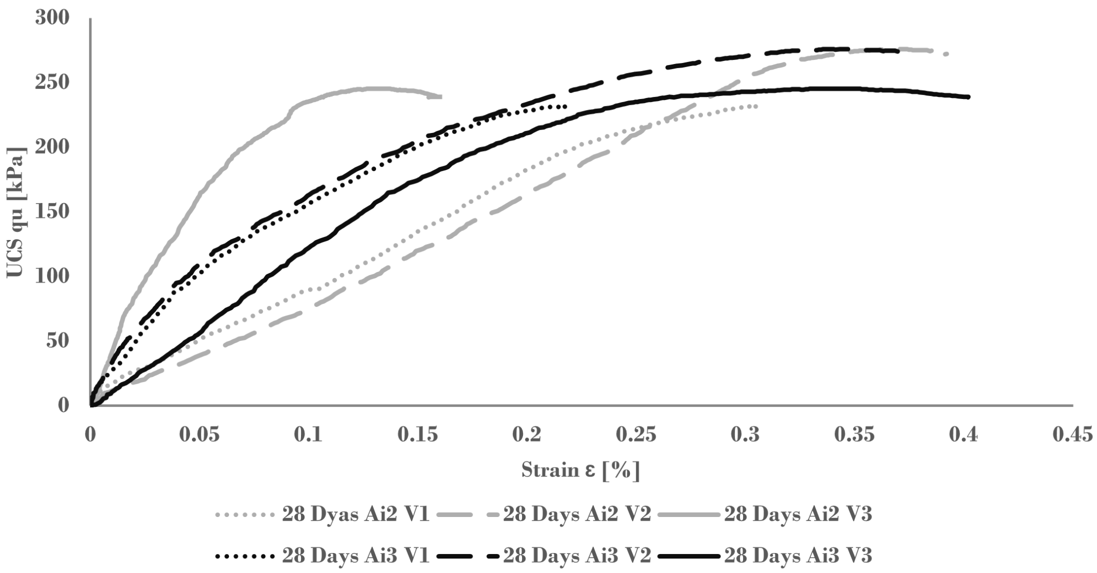



20
18
16
14
12
10
8
6
4
2
0
UCS qu [kPa]
0
0.05
0.1
0.15
0.2
0.25
Strain ε [%]
28 Days Ai2 V1
28 Days Ai3 V1
28 Days Ai2 V2
28 Days Ai3 V2

D.4 Mixture M4

V1 references the bottom half of the sample tube and V2 the top half.





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