Master's thesis

NTNU
Norwegian University of Science and Technology
Faculty of Engineering
Department of Civil and Environmental Engineering

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Comparing laboratory and field stabilised clay

Methods for laboratory sample preparation to represent lime and field stabilised clay

Master's thesis in Civil and Environmental Engineering
Supervisor: Priscilla Paniagua, NTNU/NGI
Co-supervisor: Sølve Hov, NGI

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Kunnskap for en bedre verden

Preface

This master's thesis on soil deep stabilisation, by the use of quicklime and cement, aims to uncover why the laboratory test mixes seems to fail to replicate the properties of field stabilised material with the methods used in Norway today, and how they could be improved. The background for the thesis is based on an idea by Arnfinn Emdal, and a continuation of the work done on lime-cement deep mix soil stabilisation by the Geotechnical Engineering section at the Norwegian University of Science and Technology. The thesis is also written in cooperation with the Norwegian Geotechnical Institute and their extensive work on the same subject.

I wish to thank my supervisors, Priscilla Paniagua and Sølve Hov, for providing great assistance, insight and enthusiasm regarding my work. I also want to acknowledge and thank the NTNU Department of Physics with Dag Werner Breiby with his PhD candidates Kim Robert Bjørk Tekseth and Fazel Mirzaei for help with all the CT-analysis performed for this thesis. Finally, thanks to Espen Andersen and Karl Ivar Kvisvik at the geotechnical laboratory for all assistance provided.

Trondheim, 11.06.2021

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Abstract

In this thesis on the deep mix stabilisation method, the methods used for laboratory sample preparation are examined and compared to actual field stabilised material. Laboratory prepared samples were made to reproduce field stabilised material. The reference material from field was clay stabilised with cement and quicklime in different quantities. The objective was to discover why the Norwegian standardised laboratory sample preparation methods, the SVV-method and the NGF-method, often fails to represent the field material properties, especially strength.

Samples from field stabilised clay columns, with binder quantities of 30, 50, and 80 kg/m^3, were tested for density, water content, unconfined compressive strength, and analysed by CT-imaging. The material properties varied a lot. This could be due to extensive binder accumulations, which were discovered by both visual inspection and CT-imaging. The accumulations were more prominent in lower binder quantity samples, and seems to have a strong relation to the mixing process efficiency.

Laboratory samples with the two standardised sample preparation methods in Norway were produced to match the field material compositions and curing conditions. The results showed that the NGF-method produced samples with a higher shear strength, density and stiffness than the SVV-method, but still failed to reach the values for the field material. It was also discovered with CT-imaging how the laboratory methods produced highly porous samples with pore structures being very dependent on the binder quantity used. The field samples had very low porosity for all binder quantities and also much higher densities. For laboratory samples the strength seemed linearly increasing with the density. Field samples did not follow the same trend even with comparable material composition and curing conditions.

In addition to the standardised preparation methods, other methods were proposed and tested to see if they could create samples which better describes the field properties. Using the SVV and NGF-method with increased compaction energy tended to yield samples with more accurate shear strengths and density compared to the field material. Even though no definite recommendation could be made on improving the laboratory sample preparation methods, the results are still important for stating the potential of improving the methods, and which method is worth investigating with further research.

Sammendrag

I denne masteroppgaven om kalksementstabilisering ble prepareringsmetodene for prøveinnblandinger av kalksementstabilisert leire testet og sammenlignet opp mot feltstabilisert materiale. Referansematerialet var leire stabilisert med sement og brent kalk i ulike bindemiddelsmengder. Målet var å finne ut hvorfor de standardiserte metodene for laboratoriepreparering av kalksementprøver, SVV- og NGF-metoden, ofte ikke produserer prøver som representerer feltstabilisert leire, særlig med hensyn på styrke.

Leire fra kalksementpeler med bindemiddelmengder 30-, 50- og 80 kg/m^3 ble testet for densitet og enaksial skjærstyrke samt analysert ved CT bildebehandling. Materialvariasjonen i prøvene var høye. Dette kan skyldes forekomsten av bindemiddelopphopinger som var svært synlig både rent visuelt og i CT-bildene. Bindemiddelopphopningene var tydeligst i prøver med lav bindemiddelmengde og har sannsynligvis stor korrelasjon med effektivitet av innblandingsarbeidet i felt.

Laboratorieprøver, preparert med de standardiserte metodene for prøvepreparering brukt i Norge, ble produsert slik at de skal samsvare med feltmateriale i form av materialsammensetning og herdeforhold. Resultatene viste at prøver preparert med NGF-metoden gav en høyere skjærstyrke, densitet og stivhet enn for SVV-metoden. Allikevel var verdiene betydeligere lavere sammenlignet med feltprøvene. CT-bildene tydeliggjorde også hvordan prøveinnblandingen i laboratoriet produserer prøver med høy porøsitet med varierende struktur ut ifra bindemiddelmengden brukt. Feltprøvene hadde veldig lav porøsitet for alle bindemiddelmengder og derav også høyere densitet. For laboratorieprøvene virket skjærstyrken å øke lineært med densiteten. Feltprøven fulgte ikke samme trend selv ved tilnærmet lik materialsammensetning og herdeforhold.

I tillegg til testing av de standardiserte metodene, ble nye metoder for preparering av laboratorieprøver testet for å se om de bedre kan beskrive egenskapene til feltstabilisert materiale. Bruk av SVV- og NGF-metoden med forhøyet komprimeringsenergi virker som at gav en mer presis beskrivelse av styrke og densitet til feltmaterialer. Selv om ingen anbefaling på forbedring av prepareringsmetoder for laboratorieprøver er resultatene nyttige for å fremheve potensiale i metodeendring, samt og gi en pekepinn på hvilke metoder er de beste alternativer for videre undersøkelser.

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Acronyms and symbols

B60	Quicklime (Terraplus B60, CaO>60%)
CEM	Cement
CKD	Cement Kiln Dust
COV	Coefficient Of Variation
CPTU	Cone Penetration Test
CT	Micro Computed Tomography (also µCT)
NGF	Norsk Geoteknisk Forening [English: Norwegian Geotechnical Assosiation]
NGI	Norwegian Geotechnical Institute
RCP	Reverse Column Penetration
SVV	Statens Vegvesen [English: Norwegian Public Road Adminstration]

c_u	Undrained shear strength
$c_{u,28}$	28 days undrained shear strength
c_{ur}	Remoulded shear strength
E_{50}	Stiffness
ε_v	Vertical strain
I_L	Liquid index
I_P	Plasticity index
K_0'	Earth pressure coefficient
q_u	Undrained compressive strength
$q_{u,28}$	28 days undrained compressive strength
ρ	Density
S_t	Sensitivity
t	Time
w	Water content
w_L	Liquid limit
w_P	Plasticity limit

1 | Introduction

1.1 Background

Deep stabilisation of soil with lime and cement, also denoted as deep mixing, is a frequent used method for ground improvement in Norway today. For every deep mixing project it is recommended to perform a laboratory test mix where clay collected from the project site is stabilised with the proposed binder agents to experience which properties could be achieved in real field stabilisation.

To perform the laboratory test mix, standard size samples with stabilised clay are prepared and cured before testing. For the sample preparation procedure there exist two standardised methods. The first method is from the Norwegian Public Road Administration, which in this thesis mainly is referred to as the SVV-method, and the other method is from the Norwegian Geotechnical Association, which mainly is referred to as the NGF-method. Both methods are quite similar except for the moulding technique used to create standardised samples sizes. By experiences, the two methods tend to create samples with very different properties. This is a problem as both methods are supposed to describe the properties of the same field stabilised material. To add to the problem, none of the methods actually manage to reach the strength of what is usually achieved in field. As a consequence, researchers have been trying to improve the laboratory sample preparation methods. For instance, during the work on this thesis, the Norwegian Public Road Administration (project KlimaGrunn) is trying to develop a new standardised laboratory sample preparation method, which intentionally could be a better asset in deciding design values for ground improvements (Seehusen, 31.05.2021).

1.2 Objectives

The overall objective of this thesis is to examine the properties of both field and laboratory stabilised material for comparing and uncovering how and why laboratory procedures for sample preparation fail to predict the measured field strength. For this, the following sub-objectives are also specified:

a) To compare the two Norwegian standardised methods for laboratory sample preparation in order to define which one is best suited for reproducing field samples properties.

b) To propose and test new methods for laboratory sample preparations to examine possible approaches for improving the current standardised procedures.

1.3 Approach

The approach for this thesis is first retrieving and testing field stabilised clay from a Norwegian road project where the Norwegian Geotechnical Institute (NGI) has installed lime-cement piles stabilised with three different binder quantities. Laboratory samples with the two Norwegian standardised preparation method is later produced and tested with the same methods as for the field samples. This creates a good basis for evaluating the difference in field and laboratory stabilised material. For the last part, seven new methods for laboratory sample preparation are suggested, tested and compared to the obtained field material properties.

The testing procedure is mainly based on the standardised test of unconfined compressive strength. For this thesis extensive micro computed tomography (µCT) imaging was also performed on the field and the laboratory samples produced by the two standardised preparation methods. This gives an unique insight of how the mixing methods in field and the preparation methods in laboratory affects the structures inside each samples. The CT-analysis were not performed on samples from the new trial laboratory preparation methods.

1.4 Limitations

The findings in this thesis is limited to one single deep mix stabilisation project. The methods for field stabilisation are quite standard for projects in Norway but the soil and the variable settings as mixing equipment, binder type and curing conditions do not change. This may limit the findings relative to other projects.

Another limitation is the data amount analysed. Even though a lot of material is tested, huge variations in the results especially in field material is observed. Therefore, there will exist an uncertainty especially when appointing an uniform property to the soil. Larger data sets would expectantly reduce the deviations giving more confidence in the obtained results.

1.5 Outline of the thesis

The thesis is presented with five remaining chapters. Chapter 2 is a theoretical review on the most important factors that affects the properties of the lime-cement stabilised soils, which is an important literature basis for this thesis. Chapter 3 is the methods chapter in which all test methods, as well as the stabilisation and sample preparation methods for the field and laboratory stabilised material is presented with the respective test plan. Chapter 4 presents all the results from the tests performed. Here, field samples test results are presented first, followed up by the standardised laboratory samples results and finally the new trial methods samples results. Chapter 5 and 6 are the discussion and conclusion respectively.

In the appendix, a great selection of the results are re-presented in larger scale. This is due to the importance of the image results, especially for the CT-scans. In the appendix, a more detailed laboratory plan is also presented.

2 | Theory

2.1 Field stabilisation

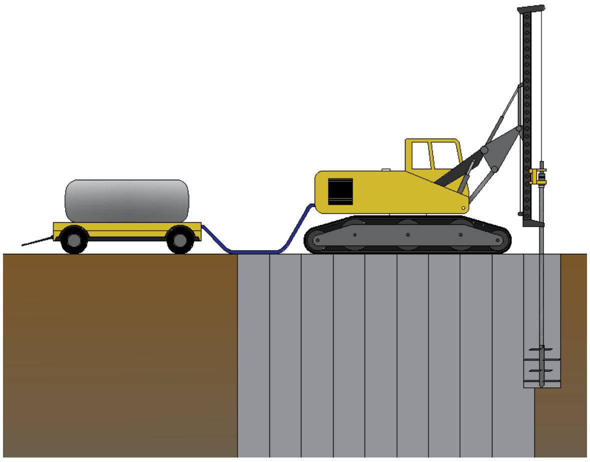


Figure 2.1: Deep stabilisation principle

The most common procedure for deep stabilisation with lime and cement is described in the Norwegian guideline for lime-cement stabilisation (Norsk Geoteknisk Forening, 2012). The stabilisation is performed by drilling down to the desired depth (up to 25 meters) with a special design mixing tool of between 0.5 and 1 meter in diameter (but typically 0.6 or 0.8 m). The binder is fed to the soil through a nozzle on the rotating mixing tool while it is retrieved towards the surface at a preset rate. There is a few factors in the installation process that will define the properties of the stabilised soil. Of all the settings that are varied in the installation process the following are often identified as the most prominent:

- *Retrieval rate (mm/rev)*: The rate of which the mixing tool is retrieved towards the surface. The binder feeding rate must be calculated based on this to reach the desired binder amount per soil volume.
- *Rotation speed (revolution/sec)*: Number of revolutions per seconds on the mixing tool.
- *Installation pressure (bars)*: The pressure in which the binder is injected through the feeder nozzle. Need to be adjusted for the soil depth.
- *Mixing tool*: The design and geometry of the mixing tool. For instance, the number of blades.

All these factors are dependent on the drilling rig system and its capacity (Larsson, Dahlström, & Nilsson, 2005b). Controlling these factors is important to understand the product of the stabilised output, especially as binder accumulations being a common observation in lime-cement columns (Larsson, 2001). For instance, a Swedish study on lime cement columns done by Larsson et al. (2005b) and Larsson, Dahlström, and Nilsson (2005a) identified that retrieval rate and the number of blades on the mixing tool were the main factors for qualitative mixing. Lower retrieval rate and higher number of blades on the mixing tool highly reduced the binder accumulations in the soil and increases the shear strength with a lower deviation. The article also introduced a blade rotation number T, a measurement of mixing effort (see equation 2.1).

$$T = \Sigma M \times \frac{1}{s} \tag{2.1}$$

Here ΣM is the number of blades on the mixing tool and s is the retrieval rate in m/revolution. The study implied that T>400/m gave quite homogeneous lime-cement piles with no visual binder accumulations. However, the variation in the piles was very high even with a similar installation method. It is interesting evaluating the quality of the clay using the T-rate in this thesis as the Swedish methods on deep mix stabilisation are similar to those used in Norway.

2.2 Laboratory sample preparation

Test mixing of lime-cement with soils in the laboratory is an essential part of the quality assurance within the design phase of any deep stabilisation project. The Norwegian Standard (European Standard) NS-EN 14679 states this necessity for the purpose of controlling which material properties can be obtained, and which types and amount of binder have to be used, to achieve the desired soil strength (Norsk Geoteknisk Forening, 2012). Laboratory produced samples can also be used to define in-situ design values. For example, the national guideline Handbook V221 from the Norwegian Public Road Administration states that the design strength for the deep stabilisation piles should always be the minimum value between a predefined set of empirical characteristic maximum strength values and the measured laboratory strength of lime-cement samples (Statens Vegvesen, 2014). This guideline is based on that laboratory prepared samples almost with no exceptions tends to yield a lower shear strength than the actual in-situ soil. This experience is not uniformly shared across the international geotechnical deep mix community. For instance, in Japanese and American practice, the unconfined strength of field sample for onshore stabilised clay is usually significantly lower than the laboratory samples, often less than half the strength (Kitazume & Terashi, 2013) (Bruce et al., 2013). However, neither the methods for laboratory sample preparation or the field mixing methods are uniform across the international geotechnical societies.

Even though the concept of laboratory mix test is widely customary, certain parts of the actual methods of laboratory sample preparation are quite different across the world. The most significant part is often the moulding technique. The most common are, as presented by Kitazume et al. (2015):

- *Tapping* (TP): Layer compaction by tapping the mould against a hard surface
- *Rodding* (RD): Using a rod to softly tamp layers of soil
- *Dynamic Compaction* (DC): Layer compaction by dropping a weight with constant fall height and number of drops
- *Static Compaction* (SC): Statically compression of each layer for a given time
- *No Compaction* (NC): Filling of mould with no compaction

Which of the method used, in addition to standardised method in the respective geotechnical societies, is highly dependent on the applicability of the methods on the relevant soil. For instance, Kitazume et al. (2015) stated that tapping and rodding are highly suitable for soils with low shear strengths and high liquidity index, with rodding also suitable for semi-low shear strengths (10-20 kPa). Rodding can also successfully be used, along with dynamic compaction, on soils with higher shear strength and low liquidity index.

However, the moulding technique used is more dependent on the actual deep mixing methods used in the stabilisation process. For example, the most common method for deep stabilisation in Japan and the USA is the wet/slurry mixing method where water is added to the mix in addition to the binder, creating a more liquid consistency of the stabilised material. The tapping method is as a consequence of this the preferred moulding method, especially in Japan, but also in America along with rodding for more stiff materials (Kitazume & Terashi, 2013) (Bruce et al., 2013)). In Norway and Sweden on the other hand, the most common stabilisation method is dry mixing where no water is added. This produces a stiffer material in which makes tapping an inapplicable moulding method. In Sweden the standardised method is static compaction (Åhnberg & Andersson, 2015). In Norway, both static compaction method and dynamic compaction method are used, respectively referred to as the SVV-method from the Norwegian Public Road Administration and the NGF-method from the Norwegian Geotechnical Association. The SVV-method and NGF-method, as earlier mentioned, usually produces samples with significantly different properties, which is the basis for this thesis. The methods are described in detail for chapter 3.

2.3 Binder type and ratio

The concept of deep stabilisation with lime-cement binders is the using lime and cements to increase the strength of the soil. However, it is not standardised neither in which ratio the binder or the exact types of binder to be used, but recommendations on the binder composition, ratio and quantity are to a certain degree given in the lime-cement stabilisation guideline (Norsk Geoteknisk Forening, 2012). In earlier deep stabilisation projects, it has been normal to stabilise with only one component. For example, in Sweden, mixing of lime and cement as binder became normal in the late 1980s, while use of just lime was most common before (Åhnberg, 2006). In recent years, combining cement with lime or products of cement kiln dust is most common in the Nordic countries.

The idea of combining lime and cement is based on the different reactions the binders have on the soil. In a lime-cement mix, the lime acts as a rapid stabiliser in which burnt lime, also called quicklime, instantly binds water, and dries the soil (see equation 2.2). This will increase the stability in the soil, mainly in a short term. In a long term perspective the lime will also increase the shear strength with a pozzolanic reaction (Janz & Johansson, 2002).

$$CaO + H_2O \longrightarrow Ca(OH)_2 \tag{2.2}$$

When the hydrated lime ($Ca(OH)_2$) later dissolves in the pore water, a cation exchange occurs at the surface of the soil particles and the plasticity limit (w_p) of the soil increases, decreasing the plasticity index (I_p). If the pH in the soil remains high, a pozzolanic reaction may occur if the calcium ions together with silica and/or aluminium from the soil creates a cementitious gel

(CSH) (see equation 2.3). This may contribute to increasing the long-term shear strength of the soil (Kitazume & Terashi, 2013).

$$Ca(OH)_2 + pozzolana\,(e.g.\ SiO_2) + H_2O \longrightarrow CSH \qquad (simplified) \tag{2.3}$$

The cement, which mainly refers to Portland cements, is the main long term stabilising agent of the binder mix. The cement is slowly hydrated relative to the lime, but on the other hand contributes a lot to increasing the long-term shear strength of the soil. This is due to the creation of cementitious agents (CSH) by the reaction between cement clinker (mainly alite and belite) and water contained in the soil (see equation 2.4) (Jacobsen et al., 2016).

$$2C_3S + 6H \longrightarrow C_3S_2H_3 + 3CH \qquad (alite\ hydration\ simplified) \tag{2.4}$$

In Norway, standard Portland cements are used without or with secondary cementitious agents (respectively CEM I and CEM II cements by the Norwegian standard NS-EN 197-1). For cements with pre-mixed secondary agents, for example as the pozzolanic agent fly ash, the properties of the cements are made to satisfy the same quality as "clean" cement (CEM I). This is normally done to cut cost and reduce the environmental impact, especially for the concrete industry (Jacobsen et al., 2016).

It is a well-known fact that both productions of Portland cement and quicklime produces a vast amount of CO_2-emissions. Therefore, for the lime-cement soil stabilisation industry, efforts have been made to further reduce the environmental impact, and to some extent the cost. This can be done with mainly three strategies. The first one is reducing the overall amount of binder to a minimum amount necessary for sufficient stabilisation. Another strategy is replacing all the lime and cement with other kinds of binder. The third is partly replacing the lime or/and cement with other constituent with binding effects. For instance, the Japanese Geotechnical society has a broad use of blast furnace cements (comparable to CEM III cements from NS-EN 197-1), introducing granulated blast furnace slag, a waste product from steel production (Kitazume & Terashi, 2013). Another example is the use of cement kiln dust which is a waste product from cement clinker production. With its significant contents of calcium oxide (quicklime) the agent can successfully replace parts of, or all the "standard" lime usage. Eidsmo (2019b) also presents that the use of cement kiln dust and cement instead of lime and cement have the potential to yield higher shear strength in the soil.

It is also important to specify in what ratio different kinds of binder is to be used. Norsk Geoteknisk Forening (2012) presents a ratio of 50% lime and 50% cement as the default value traditionally used in Norway, but it is important to state that this ratio is usually not optimised for every deep-stabilising project. For instance, one might have big concerns for the short-term stability, maybe in the stabilising process itself with heavy machinery and large mechanical disturbance, causing the project manager to choose a higher percentage of quick reacting lime binder.

2.4 Binder quantity and the w/c-ratio

One of the factors which also is assessed in this thesis is the binder quantity. This is the amount of lime and cement or any other constituent with a strength increasing effect, added to a soil volume. Normally, this measures as a mass unit per volume, but in some cases, the mass percentage of dry soil is used (Statens Vegvesen, 2016). However, for this thesis kilogram (kg) binder per cubic meter (m^3) soil is used as standard.

Traditionally, it is recommended to use binder quantity of at least 75 kg/m^3 for stabilisation depending on the soil and binder type. Sensitive clay, for instance, is often mixed with $90-120$ kg/m^3 binder, while more peaty soils requires even more binder (up to 300 kg/m^3) with a higher cement ratio (Norsk Geoteknisk Forening, 2012). Newer studies, suggest that significantly lower binder amounts yield more than enough strength development for construction purposes.

Paniagua, Bache, Lund, and Karlsrud (2020) indicate that binder quantities down to 30 kg/m^3 shows a noticeable increase in shear strength for very sensitive clays. However, for more stiff clays the effect of the small amount of binder is very limited. It is also worth mentioning that the shear strength development seems increasingly higher for a larger amount of binder used. Nevertheless, minimising the binder amount according to the desired soil strength will play a vital role in decreasing the cost and the environmental impact of the construction industry.

In this thesis the binder quantity varies between 30, 50 and the standardised 80 kg/m^3. Therefore, it is expected that the shear strength varies accordingly, not only because of the binder itself, but also differences in heat generation, affecting the strength development.

In addition to binder quantity used, the strength of stabilised clay will, like normal concrete, depend on the water to cement ratio (w/c-ratio). As commonly recognised, lower w/c-ratios will typically generate higher strengths. This is vital for knowing how adding water will influence the stabilised material. If we further consider the cement stabilised clay as a concrete by definition, one can also assume that the strength is equal as long as the w/c-ratio is the same (Jacobsen et al., 2016). In practice this indicates that by doubling the binder content and the water content, the strength remains the same. This assumption is quite accurate considering figure 2.2 where the unconfined compressive strength seems equal for samples with a water content of 70% and a binder quantity of 100 kg/m^3 compared to samples with binder quantity of 200 kg/m^3 and corresponding water increase (almost up to 140%).

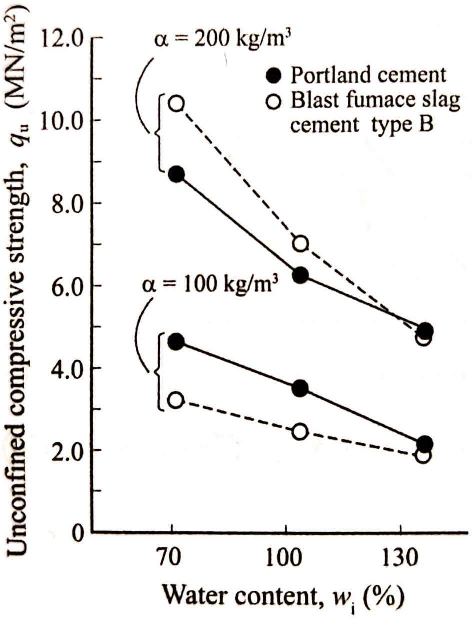


Figure 2.2: Water content effect on shear strength (Kitazume & Terashi, 2013).

If a wet stabilisation method is performed, this implies that one has to compensate by adding extra binder when adding slurry water to not lose strength. This will also be valid in laboratory sample preparation for this thesis as one of the trial methods involve adding water to increase the workability/compactability of the clay.

2.5 Curing temperature and time

The hydration reaction of both quicklime and cement are exothermic reactions. This means that mixing lime and cement into saturated soil will create heat. For quicklime one can estimate a heat production of approximately 15.6 kcal/mol (Kitazume & Terashi, 2013) or approximately 1166 kJ/kg of pure quicklime when fully hydrated. The same number for standard Portland construction cement is approximately 400-500 kJ/kg throughout the curing period (Jacobsen et al., 2016). This will cause a significant heat development in the lime-cement stabilised soil.

Since hydration of concrete tends to speed up with increasing curing temperature, the effect will amplify, creating even more energy. For instance, figure 2.3 shows the relationship of isothermal curing temperature and energy released from a Portland cement over time (Jacobsen et al., 2016). For lime-cement stabilised soil the reaction will be slower, but the tendency recognises to be the same. The lime will cause an immediate response to the temperature in a deep stabilisation column, while the cement will have a more modest influence over a longer time. When using lime and cement together the responses will combine. Figure 2.4 shows the temperature evolution for a Swedish deep mix pile research project involving clay, stabilised with lime (Kalk) and cement. Even though the surrounding soil maintains a temperature of about 7-8°C one can still see a significant higher temperature inside the piles, with increasing effect in group piles (Åhnberg et al., 1995).

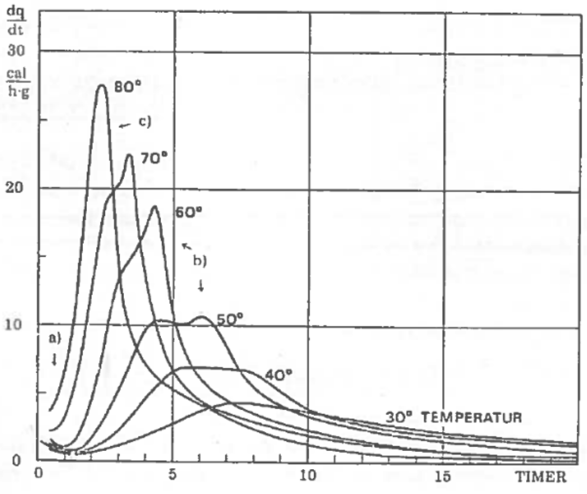


Figure 2.3: Relation of curing temperature and heat generation (Jacobsen et al., 2016)

Wiersholm (2018) tested the same effect on Norwegian soil with temperature sensors at different depths and different placements in the column structure. All the columns were made with binder quantity of 80 kg/m^3 with an equal amount of lime and cement. The columns kept temperatures of around 20°C up to 90 days after the installation, with the most stable and highest temperatures at the highest depths (below about 10 meters). This is an important observation as the strength development of lime-cement columns are highly dependent on temperature.

Temperature regulations is an important factor when laboratory samples are produced and cured to match the in-situ conditions. The Norwegian guideline for lime-cement stabilisation

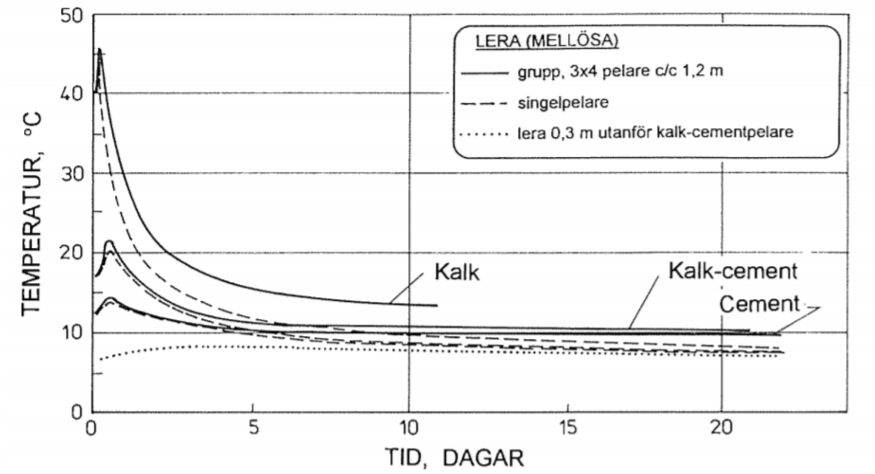


Figure 2.4: Temperature development in lime-cement columns (Åhnberg et al., 1995). Time (days) vs temperature (°C)

Norsk Geoteknisk Forening (2012) recommends curing laboratory samples at refrigerator temperature (6-8°C) which is the assumed ground temperature. This seems correct if the reference field sample situates in the outer periphery or in the shallow depth of the lime-cement column, but as seen in Wiersholm (2018) even here the temperature often is significantly higher than the surrounding soil. The standardised curing temperature for laboratory samples differ somewhat across the international geotechnical societies but is mostly set to about 20°C (Al-Jabban, Laue, Knutsson, & Al-Ansari, 2020). For instance, the Japanese society cures their samples at a standard temperature of $20°C \pm 3°C$ (Kitazume & Terashi, 2013).

It is important when making and curing laboratory samples, that the temperature and curing time are matching the in-situ conditions. As figure 2.5 shows, the temperature and curing time greatly affects the unconfined compressive strength, and therefore also the shear strength. Even though the Norwegian guideline for lime-cement stabilisation suggests curing temperature of 6-8°C the target for this trial is to have a more realistic approach as vital data depends on having matching curing conditions for the field and laboratory samples. An obvious approach is using the concept of maturity which links the curing temperature and curing time, but this requires continuous control of the curing temperature of the columns. Therefore, a curing temperature for in-situ material is chosen based on qualified assumptions. For samples cured longer than 28 days, the mean value (with respect to logarithmic hydration speed) is es-

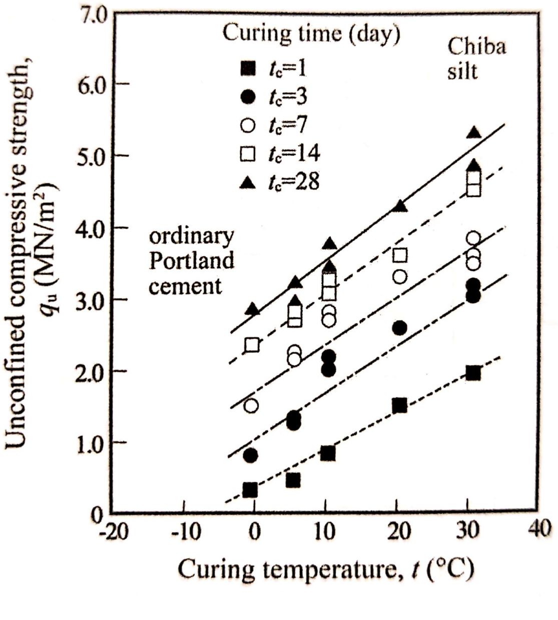


Figure 2.5: Relationship between strength, curing temperature and curing time (Enami et al., 1985)

timated so that laboratory produced samples are expected to match that of in-situ cured field samples which has a mainly logarithmic decreasing curing temperature as seen in figure 2.4. For this thesis, a curing temperature of 20°C (room temperature) is set as standard for the laboratory samples on the assumption of an average temperature for the lime-cement columns in a maturity aspect. However, it is important to remember that this assumption does not consider different binder quantities or qualities.

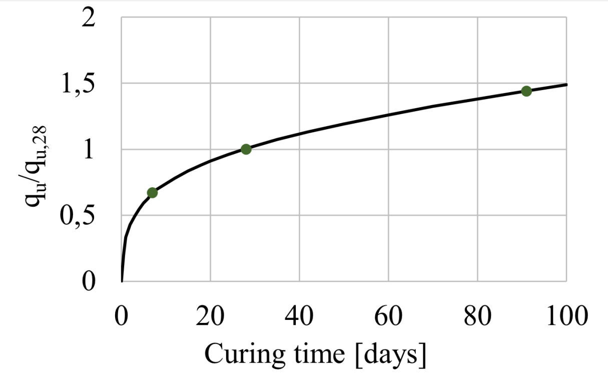


Figure 2.6: Curing time correction curve for unconfined compressive strength (Kitazume & Terashi, 2013)

Typically, 28 days curing time is the standard measurement for obtained strength. This is however based on concrete technology where the strength development is relatively minimal after this period. For cement stabilised clay however, the curing period is typically slower than for normal concretes. Kitazume and Terashi (2013) states that the strength increase for unconfined compression strength is 44% from day 28 to 91. By empirical data, all material cured less or more than 28 days could be compared with respect to real curing time by using the ratio $\mathrm{q_u/q_{28}}$ presented in figure 2.6, and equation 2.5.

$$\frac{q_u}{q_{u,28}} \approx 0.0029 \cdot t + 0.22 \cdot \ln(1.46 \cdot t + 0.63) + 0.1 \tag{2.5}$$

Åhnberg (2006) also had a similar finding and suggested that the unconfined compressive strength for curing time of t days could be described according to equation 2.6. The equation is however more valid for curing times above 7 days.

$$\frac{q_u}{q_{u,28}} \approx 0.3 \cdot ln(t) \tag{2.6}$$

2.6 Curing stress

The curing stress is one of the major differences concerning the comparison of field and laboratory produced lime-cement samples. The basis for any soil element observed is the overburden stress from the soil, depending on the soil density and the depth of the soil element, and the earth pressure coefficient at rest K_0'. For example, this implies that a stabilised soil element in a lime-cement column at 10 meters depth will often experience an overburden pressure of close to 200 kPa. If we add the radial stress from the surrounding clay with an earth pressure coefficient of about 0.5 the horizontal stress also contributes with close to 100 kPa. Statens Vegvesen (2014) also states that due to expansion of the soil volume as a consequence of adding binders to the soil, the horizontal stress upon the column could be considered as equal to the vertical stress. From this, the anticipation is that field stabilised material is or is close to isotropic.

Either way, the overburden curing stress is not taken into account in the production methods for laboratory samples even though they are representing stabilised material for any depth in the lime-cement columns. As presented from a Japanese study on lime-cement deep mix columns in figure 2.7, the unconfined compressive strength clearly increases with the overburden pressure. This means that the obtained field strength might be influenced by the depth in which the strength is measured within the column. This effect could be compensated for by applying curing stress on the laboratory prepared samples, as tested by Engseth (2018) with proven effect, but this requires more extensive laboratory preparation work. Another option is to let the laboratory prepared samples being representative of an average column strength, as suggested by Kitazume and Terashi (2013).

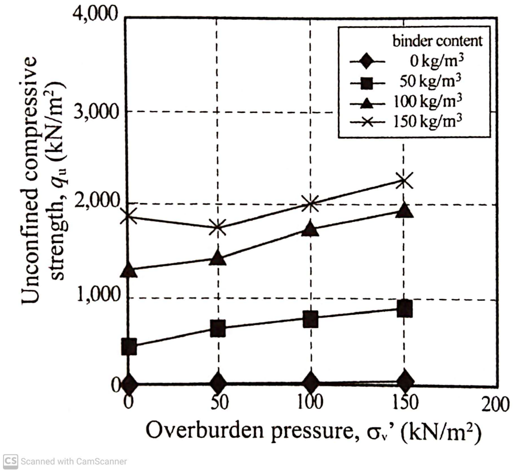


Figure 2.7: Relationship between overburden pressure and unconfined compression strength for clay (Kitazume & Terashi, 2013)

3 | Method

The core objective of this thesis is to study how laboratory samples compares to the properties of in-situ field stabilised soil, and what methods is best suited. For this, three phases were conducted. The first one was the collection and testing of actual field samples with mixed stabilised clay. Thereafter, in the second phase, laboratory samples were made with the given methods for preparation to analyse the applicability to predict the in-situ properties. In the third phase, suggestions for new and improved methods for laboratory preparation were tested.

3.1 Testing procedures

The test methods for this thesis are based on analysis of shear strength with unconfined compression tests in addition to analysis of sample structure with CT-scanning. The testing is done similarly for the field and laboratory samples to be able to compare the results. In addition, some supplementary test data from NGI will also be presented for the field stabilised material.

3.1.1 Initial testing of non-stabilised clay

Clay for use in laboratory sample preparation and as reference material for field stabilised samples are collected and tested for undrained and remoulded shear strength (falling cone test), density, water content, liquid limit, and plastic limit.

3.1.2 Initial testing and inspection of stabilised samples

Before testing all the field and laboratory stabilised samples are measured with respect to dimensions (height and avg. diameter), weighed and photographed. All the samples except the CT-samples are also measured with respect to water content after the unconfined compression test.

3.1.3 CT-analysis

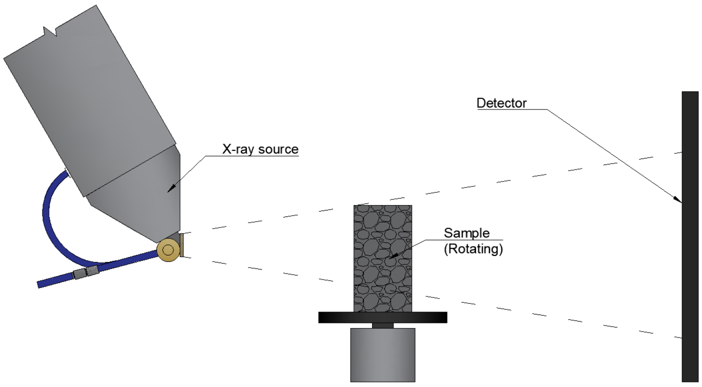


Figure 3.1: Basic setup for CT-scans

Micro computed tomography (μCT or just CT) is performed at a selection of field and laboratory samples by representatives from NTNU Department of Physics. The objective of using the CT-analysis is to clearly separate which factors influence performances of field and laboratory-produced samples on a small millimetre scale. The testing method allows for a unique three-dimensional digital reconstruction of each sample, with x-ray beams mapping areas of samples with differences in density, expectantly uncovering the macro pore structures, binder-clay layers or any other discontinuities. From the image analysis, a porosity estimate is also made by digitally segmenting images to separate solid material and pores. The porosity fineness is limited to the resolution or the voxels size. Detailed information on the CT-setup and porosity estimate is described in appendix B.

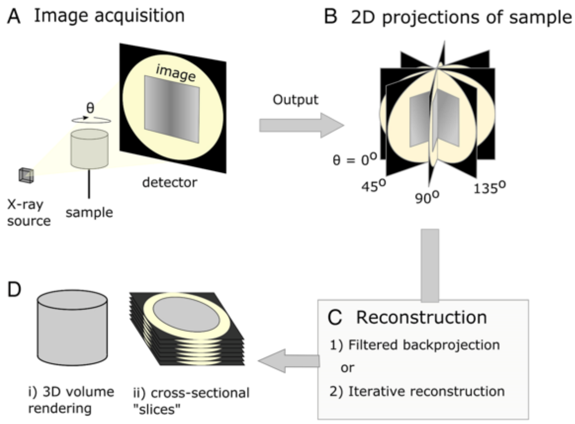


Figure 3.2: Principal of 3D digital reconstruction. Figure as presented by NTNU Physics Department.

Because of the size and the available equipment, to get a good quality image, only a portion of the sample height is imaged. This sets to the top 67 mm for both the field and laboratory samples which combined with the standard diameter of 54mm gives the ideal picture quality for the chosen scale. This should not make any difference for the field samples which assumes to be similar throughout the sample height. For the laboratory samples, on the other hand, the preparation method tends to create layers of varying porosity visible in the CT-analysis. This should nevertheless not be obstructing the ability to analyse the laboratory samples in a representative way.

3.1.4 Unconfined compression test

Unconfined compression test (UCT) is the standard test method of shear strength estimation in lime-cement stabilised soil samples in Norway (Norsk Geoteknisk Forening, 2012) and is therefore the preferred method for this thesis.

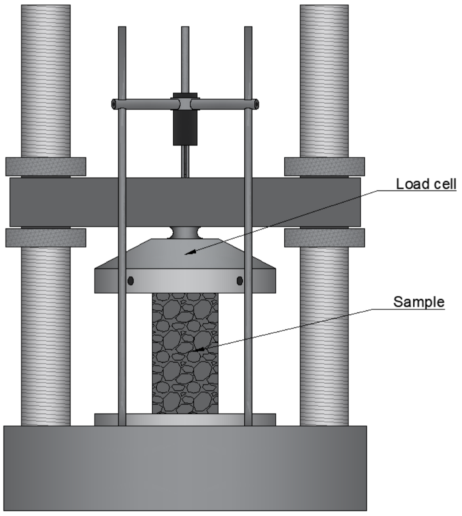


Figure 3.3: Unconfined compression test load cell setup

For the field samples, due to the method of field sample retrieval (see section 3.2.2), no measures to preserve any sample orientation could be made. Therefore, the samples are analysed as an isotropic material even though anisotropy could be a possibility due to potential anisotropic stress situation and mixing process. This might be a problem for the accuracy of the test because of the anisotropic curing stresses which place the vertical stress higher than the horizontal stress. On the other hand, as mentioned in section 2.6, the effect is somewhat limited due to the volume expansion effect of the quicklime component, which will reduce the difference between vertical and horizontal curing stress. In addition, some studies have shown that Norwegian lime-cement stabilised clay shows no sign for anisotropic behaviour (Hanson, 2012). For the laboratory samples, the sample preparation methods (moulding methods) compacts in one (vertical) axis which makes anisotropy inevitable. (Preparation methods in section 3.3.1).

The settings for the test are from the Norwegian standardised UCT method for lime-cement samples which states a deform rate of 1.5% per minute, or specifically 1.5 mm/min for a sample height of 100 mm (Norsk Geoteknisk Forening, 2012). The results of the test were also corrected for variation in diameter. Any deviation in height was, due to fitting in the apparatus, corrected with additional sample trimming right before testing.

3.2 Field samples

3.2.1 Background information

The field stabilised clay is from the Norwegian public road project E6 Kvithammar – Åsen north of Stjørdal in the Trøndelag region. The region is widely known for the large deposits of marine clay, which makes certain areas especially susceptible for occurrences of quick clay zones, making deep stabilising with lime-cement a relevant method for geotechnical ground stabilisation. The stabilised area is a segment of new road in a terrain cut, in need of stabilisation. In addition to the ordinary deep stabilisation piles the project geotechnical consultants, the Norwegian Geotechnical Institute (NGI), has placed scientific research piles as a part of their research on lime-cement deep stabilisation. This results in access to a vast amount of data material.

Figure 3.4: Map and model of the area for field stabilisation. Retrieved from Google Maps and Nye Veger (2020)

A part of NGI's research on deep stabilisation is to examine the possibility and effect of reducing the binder amount when producing the deep stabilisation piles. For instance, the standard "production" piles for the area consist of only 50 kg binder per cubic meter soil. As mentioned in section 2.4 the recommended minimum binder quantity for this soil type is 80 kg/m^3. The research piles also introduce 30 kg/m^3 and the standardised quantity of 80 kg/m^3 to study the binder content effect of both obtained strength as well as the practical aspects of production. As a result of the differ in binder quantities used, the stabilising equipment, built for higher binder quantities, will compensate with increasing the mixing tool retrieval rate to secure a minimum binder flow through the feeder nozzle. This causes a higher retrieval rate for the lower binder quantity piles which potentially could influence the homogeneity of the stabilised material according to section 2.1.

The different piles and their properties are presented in table 3.1. Here type I, II and III indicate binder quantities of 30, 50, and 80 kg/m^3 respectively. "a" and "b" represent different location for retrieval (different piles). The pile number represents a location according to the pile plan in appendix C.1. The retrieval rate, rotation speed and the T-rate from equation 2.1 are also presented in the table. The binder type used is presented in table 3.2.

Table 3.1: Field stabilised material

Types	**Pile numbers**	**Binder type**	**Ratio**	**Binder quantity** [kg/m3]	**Retrieval rate** [mm/rev]	**Rotation speed** [rev/min]	**T-rate** [/m]
Type I-a	Research A1-A10	CEM-B60	50/50	30	37	150	229
Type I-b	Research A11-A20	CEM-B60	50/50	30	37	147	255
Type II	Production piles	CEM-B60	50/50	50	20	170	400
Type III-a	Research A21-A30	CEM-B60	50/50	80	17	135	489
Type III-b	Research A31-A40	CEM-B60	50/50	80	13	138	727

Table 3.2: Binder types

Abbreviation	**Type**	**Description**	**Producer**
CEM	Cement	Standardsement FA (CEM II/B-M 42,5 R)	Norcem (Brevik)
B60	Quicklime	Terraplus B60 (CaO=75% ±5%)	Franzefoss Minerals

The cement used defines as a CEM II/B-M 42,5 R according to the Norwegian Standard NS-EN 206:2013+NA2014, a Portland fly ash cement with approximately 18% fly ash content (Norcem, 2021). The quicklime used contains about 75% active CaO-content (see PDS in appendix C.2). The ratio of lime and cement is 50% each.

The stabilising process took place in January of 2021. The area was stabilised with production piles in parallel double ribs (see appendix C.1). The experimental piles were installed after the main area stabilisation. The area was later set to cure until excavation to the intended terrain geometry. This leaves a curing time before sample retrieval for about nine weeks for the production piles with 50 kg/m^3 binder, and three weeks for the experimental piles with 30 and 80 kg/m^3 binder quantities. All the piles were produced by a mixing tool with 8 blades and diameter 800mm to a target depth of between 10-15 meters. The installation pressure of the binder from the feeder nozzle varied between 4 to 7 bars. The rotation speed and the retrieval rate are displayed in table 3.1 as average values.

3.2.2 Field sampling and preparation

The sampling process started a few days after the excavation. Because of the terrain adjustments the depth was equal to original pile depth of around 5 meters. An excavator was used to break of portions of the piles and later broken into smaller sizes by hand tools. Pieces of about 3-8 kg were later wrapped in double layers of plastic, marked and placed in a warm environment to avoid freezing. About 8-10 pieces were collected from each pile variation. The samples were at the end of the day transported to the laboratory storage and stored.

(a) Contours of lime-cement columns, visible between non-stabilised clay

(b) Typical raw sample piece of stabilised clay

(c) Trimming process of a field sample (F25 II)

Figure 3.5: Retrieval and trimming process of field samples

After the field retrieval the samples were trimmed to the standard cylindrical size with diameter 54mm and height of 100mm. Because of the hard and brittle behaviour of the stabilised soil, the trimming was done by knife. The trimmed samples were wrapped in plastic and alumina foil and marked. To avoid any moisture leakage and potentially drying of the sample, all were placed in plastic bags with a wet paper towel inside.

Because of the curing properties for cement being highly sensitive to temperature, and the theoretical curing temperature for deep stabilisation piles is estimated to at least 20°C, the target value for storage was also set to about 20°C (room temperature). The samples were therefore stored at 20°C (both trimmed and raw samples) until the fourth week after in-situ stabilisation. Because of the extensive work of trimming and testing, some samples were tested up to 32-35 days after installation. To reduce the effect of additional curing after the target 28 days, the samples were then placed in refrigerator (4-6°C). Therefore, based on the sample maturity, the curing time is still estimated to around 28 days at 20°C.

The production piles were installed at an earlier time than the research piles and therefore have a longer curing time. The estimated curing time is therefore set 6 weeks longer, with a curing time of about 70 days, instead of the standard 28 days. The field sample overview, with the basic properties and the intended testing programme, are presented in table 3.3.

3.2.3 Field sample testing

For each variation, a total number of five samples are tested with unconfined compression tests and one sample is tested in CT-scanning. Note that the CT-sample is also tested with unconfined compression test but do not contribute to the obtained average shear strength due to sample drying, which might cause deviation in the results. The basis for testing is at least three samples for each variation for shear strength, but due to extensive variation in the samples, a higher number is chosen. Some of the variation could for example be caused by any inhomogeneities, contents of small rock particles etc.

3.2.4 CPTU and RCP

During the field sample retrieval, cone penetration tests (CPTU) were performed at a selection of the piles. Four piles (A7, A10, A14 and A16) for type I material (30 kg/m^3 binder), two piles for type II material (50 kg/m^3 binder) and four piles (A24, A25, A30 and A31) for type III material (80 kg/m^3 binder). The test was also performed from the excavation, with the initial start depth of 5 meter. Temperature measurement is also logged for each pile. The CPTU shear strength (c_u) is interpreted from corrected tip resistance q_t according to equation 3.1, where s_{v0} is the overburden pressure and the cone factor N_{kt} is assumed equal to 15.

$$c_u = \frac{q_t - s_{v0}}{N_{kt}} \tag{3.1}$$

In addition to the CPTU readings, reverse column penetration tests (RCP) were installed and performed for total a number of six type II piles (standard production piles with 50 kg/m^3 binder) to verify the design shear strength. The RCP shear strength (c_u) is interpreted from the probe resistance q_c according to equation 3.2, where the factor $N_k = 10$.

$$c_u = \frac{q_c}{N_k} \tag{3.2}$$

Table 3.3: Field samples overview

Sample [1]	**Pile Sampled**	**Binder**	**Binder quantity** [kg/m^3]	**Retrieval rate**[2] [mm/rev]	**Rotation speed**[2] [rev/min]	**Curing time**[3] [Days]	**Testing method**[4]
F01 I-a	A1-A10	CEM-B60	30	37	150	~ 28	UCT
F02 I-a	A1-A10	CEM-B60	30	37	150	~ 28	UCT
F03 I-a	A1-A10	CEM-B60	30	37	150	~ 28	UCT
F04 I-a	A1-A10	CEM-B60	30	37	150	~ 28	UCT
F05 I-a	A1-A10	CEM-B60	30	37	150	28/~ 28	CT/UCT
F06 I-a	A1-A10	CEM-B60	30	37	150	~ 28	UCT
F11 I-b	A11-A20	CEM-B60	30	37	147	~ 28	UCT
F12 I-b	A11-A20	CEM-B60	30	37	147	~ 28	UCT
F13 I-b	A11-A20	CEM-B60	30	37	147	~ 28	UCT
F14 I-b	A11-A20	CEM-B60	30	37	147	~ 28	UCT
F15 I-b	A11-A20	CEM-B60	30	37	147	28/~ 28	CT/UCT
F16 I-b	A11-A20	CEM-B60	30	37	147	~ 28	UCT
F21 II	Prod	CEM-B60	50	20	~170	~70	UCT
F22 II	Prod	CEM-B60	50	20	~170	~70	UCT
F23 II	Prod	CEM-B60	50	20	~170	~70	UCT
F24 II	Prod	CEM-B60	50	20	~170	~70	UCT
F25 II	Prod	CEM-B60	50	20	~170	28/~70	CT/UCT
F26 II	Prod	CEM-B60	50	20	~170	~70	UCT
F31 III-a	A21-A30	CEM-B60	80	17	135	~ 28	UCT
F32 III-a	A21-A30	CEM-B60	80	17	135	~ 28	UCT
F33 III-a	A21-A30	CEM-B60	80	17	135	~ 28	UCT
F34 III-a	A21-A30	CEM-B60	80	17	135	~ 28	UCT
F35 III-a	A21-A30	CEM-B60	80	17	135	28/~ 28	CT/UCT
F36 III-a	A21-A30	CEM-B60	80	17	135	~ 28	UCT
F41 III-b	A31-A40	CEM-B60	80	13	138	~ 28	UCT
F42 III-b	A31-A40	CEM-B60	80	13	138	~ 28	UCT
F43 III-b	A31-A40	CEM-B60	80	13	138	~ 28	UCT
F44 III-b	A31-A40	CEM-B60	80	13	138	~ 28	UCT
F45 III-b	A31-A40	CEM-B60	80	13	138	28/~ 28	CT/UCT
F46 III-b	A31-A40	CEM-B60	80	13	138	~ 28	UCT

[1] Sample name explanation: F34 III-a = Field sample number 34 type III location a. The sample names can also just be named just F34 when this is more convenient

[2] The retrieval rate and the rotation speed are calculated as an average of the pile selection. The actual pile among the piles selection used for sample retrieval is not pinned.

[3] The curing time in days is displayed as calculated maturity age.

[4] UCT = Unconfined Compression Test, CT = Computed Tomography

Total number of samples: 30 (CT: 5 / UCT: 25+5)

3.3 Laboratory samples

3.3.1 Sample preparation

The laboratory samples are made to recreate the properties of the field stabilised clay. Therefore, the same cement and lime type are used, as well as non-stabilised clay collected at the same area as the field stabilisation site. Typically, one does not separate the laboratory preparation methods to match the field stabilisation techniques as mixing tool type, rotation speed, retrieval rate etc. Therefore, the mixing energy for the field samples are not represented in the laboratory produced samples. This leaves us with three variations of samples, mainly concerning the different binder quantities of 30, 50, and 80 kg/m^3. The sample overview is presented in table 3.4.

Table 3.4: Laboratory samples overview

Sample[1]	**Binder**	**Binder quantity** [kg/m^3]	**Preparation method**[2]	**Curing time** [Days]	**Curing stress** [kPa]	**Curing temperature** [°C]	**Testing method**[3]
L01-04 I	CEM-B60	30	SVV	28	0	20	UCT
L05-08 I	CEM-B60	30	NGF	28	0	20	UCT
L09 I	CEM-B60	30	SVV	28	0	20	CT
L10 I	CEM-B60	30	NGF	28	0	20	CT
L11-14 II	CEM-B60	50	SVV	28	0	20	UCT
L15-18 II	CEM-B60	50	NGF	28	0	20	UCT
L19 II	CEM-B60	50	SVV	28	0	20	CT
L20 II	CEM-B60	50	NGF	28	0	20	CT
L21-24 III	CEM-B60	80	SVV	28	0	20	UCT
L25-28 III	CEM-B60	80	NGF	28	0	20	UCT
L29 III	CEM-B60	80	SVV	28	0	20	CT
L30 III	CEM-B60	80	NGF	28	0	20	CT

[1] Sample name explanation: L26 III = Laboratory sample number 26 type III. The sample names can also just be named L26 when this is more convenient

[2] SVV-method and NGF-method according to section 3.3.1

[3] UCT = Unconfined Compression Test, CT = Computed Tomography

Total number of samples: 30 (CT: 6 / UCT:24

The mixing process of the samples is the same for each preparation method. A kitchen stand mixer is used for mixing. First, clay is placed in the bowl and mixed for about 30 seconds. Then the correct binder amount is added, and the clay is mixed for another 30 seconds. Thereafter, clay stuck to the whisk is scraped back into the bowl, and the clay is mixed for a final 30 seconds. For the moulding process, cylinder tubes with height of 100mm and inner diameter of 54mm are used for both methods.

Preparation method: Statens Vegvesen (SVV-method)

The method is a static compaction moulding method from the Norwegian Public Road Administration, using an air-pressure compactor (as seen in figure 3.6).

Ready mixed clay is placed with layers of approximately 20mm thickness (total of 5 layers). Compression pressure is 200 kPa (2 bar) for 10 seconds per layer.

Source: (Statens Vegvesen, 2016)

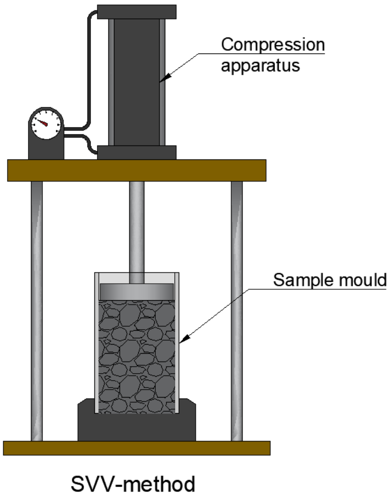


Figure 3.6: Principle of the SVV-method

Preparation method: Norges Geotekniske Forening (NGF-method)

The method is a dynamic compaction moulding method from the Norwegian Geotechnical Association.

Ready mixed clay is placed with layers of 20mm thickness (total of 5 layers). The compaction is executed with a brass cylinder/rod of 1200g with diameter 20mm. The soil is "tapped" with an energy of approximately 0.2-0.25 Nm about 20 times per layer. For this trial a 1200g piston with a falling weight of 800g is used (see figure 3.7) giving a drop height for each tap at between 2.5 to 3.1 cm.

Source: (Norsk Geoteknisk Forening, 2012)

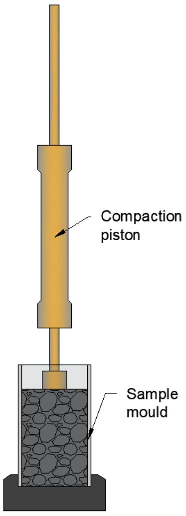


Figure 3.7: Principle of the NGF-method

After the moulding (both methods), all the samples are sealed with vapour tight plastic, marked, and cured. The curing temperature is set to a default of 20°C (room temperature) as the expected in field pile temperature (as discussed in section 2.5). The curing time is set to match that of the field samples. A more detailed description of the sample preparation process is found in appendix B.

3.3.2 Laboratory sample testing

Four samples for each variation of binder amount and preparation method were tested for shear strength in unconfined compression test while one sample were analysed by CT-scans. All the samples were photographed, weighed for density, and measured for water content. As for the laboratory samples, only four samples were tested in opposition to the field samples where five samples for each location were tested in unconfined compression test. This is due to an assumed higher repetitiveness of the sample quality in the laboratory.

3.4 New laboratory preparation methods

After the laboratory samples were tested and analysed, new sample preparation methods were suggested for trial. The objective, based on previous results and experiences, is to reduce the porosity in the samples, increasing the density and expectantly also the shear strength.

A few suggestions were considered but as the dry mixing method most often is used in Norway, the stabilised material is often very stiff and grainy. This typically implies that for example *tapping* and *no compaction* methods according to section 2.2 not is applicable. The *rodding* technique could to a certain degree been used but would require a more powerful "tamping" effort by the operator to achieve a better compaction than the methods already tested on dry mixes. As this method would be very dependant on the operator, and therefore be an uncertainty factor, the method is not tested. This leaves us with the two approaches which are in some way already in use in Norway, the *dynamic compaction* (NGF-method) and the *static compaction* (SVV-method), which will be main basis for the new sample preparation methods.

The first approaches are adjusting the SVV and NGF-method with increasing the compaction effort applied on each sample. This is done i two ways, increasing the compaction energy per layer and increasing the number of compaction layers. Increasing the compaction energy will in practice mean increasing the pressure in the compaction apparatus for the SVV-method (A1) and increasing the impact energy by increasing the drop height for the NGF-method (A2), which in total will increase the work done on each of the five layers per sample. Increasing the number of layers per sample (while maintaining the original compaction energy) will in theory increase the total compaction work performed on each sample for both SVV (B1) and NGF-method (B2) as the compaction cycles increases.

In addition to the mentioned approaches, a larger proctor-hammer was also tested. This is quite similar to the NGF-method, but can produce a higher compaction energy as the equipment is larger with a higher mass. The compression piston end is also larger, covering the whole layer area, which results in a more uniform compaction effort for each layer. The number of impacts per layer are 5, with a impact energy set either 1.0 Nm (C1) or 2.0 Nm (C2).

A final approach for this trial is to reduce the soil resistance against compaction by increasing the flow of the stabilised clay. At first a superplasticising (SP) agent was added to the mixture to hopefully increase the flow while keeping the water-cement ratio stable, but the effect was minimal and the approach was therefore consider unfit. As discussed in section 2.4, an alternative opportunity is to increasing the water and binder amount in the clay which potentially will increase the workability/compactability of the stabilised clay. This is based on the following assumptions:

- The w/c-ratio is the main factor for the strength, not binder amount alone. Any strength increase is therefore a result of a better compacted sample.
- As the quicklime content is kept the same relative to the cement, and therefore also the relative bound water in the sample, it is considered passive in this trial not contributing

to any significant increased long term strength within the reference time period (limited strength increase due to pozzolanic effect).

The water content for the samples is doubled relative to the natural water content of about 30% of the soil while the w/c-ratio is maintained by adding more binder (C2). In addition, to verify the theory, samples with double water content without additional binder is made (C1). For moulding, the "rodding" method is used by softly tamping the soil with a small rod, removing any air pockets. For detailed preparation procedure for all methods, see appendix B.

The new preparation methods used in this trial are as following:

A. Increasing the compaction energy

  A1. SVV-method with increased compression to 400 kPa.

  A2. NGF-method with increased impact energy to 0.50-0.55 Nm.

B. Increased number of layers

  B1. SVV-method with 10 layers of compaction (10mm layer thickness)

  B2. NGF-method with 10 layers of compaction (10mm layer thickness)

C. Dynamic compaction by large proctor-hammer

  C1. Large proctor-hammer with compaction energy of 1.0 Nm

  C2. Large proctor-hammer with compaction energy of 2.0 Nm

D. Increasing water and binder content

  D1. Doubled water content with rodding technique

  D2. Doubled water and binder content with rodding technique

Each of the new methods are tested with two different binder quantities, 30 and 80 kg/m^3. For each variation, three samples were made. The binder type, binder ratio, curing temperature, curing time etc. are set to be equal to the field and laboratory samples tested according to section 3.2 and 3.3. The sample overview is given in table 3.5.

The samples made with the new method are tested for shear strength in unconfined compression test. All the samples were photographed, weighed for density, and measured for water content.

Table 3.5: New trial methods samples overview

Sample[1]	**Binder**	**Binder quantity** [kg/m^3]	**Preparation method**	**Curing time** [Days]	**Curing stress** [kPa]	**Curing temperature** [°C]	**Testing method**[2]
T01-03 A1	CEM-B60	30	A1	28	0	20	UCT
T04-06 A1	CEM-B60	80	A1	28	0	20	UCT
T11-13 A2	CEM-B60	30	A2	28	0	20	UCT
T14-16 A2	CEM-B60	80	A2	28	0	20	UCT
T21-23 B1	CEM-B60	30	B1	28	0	20	UCT
T24-26 B1	CEM-B60	80	B1	28	0	20	UCT
T31-33 B2	CEM-B60	30	B2	28	0	20	UCT
T34-36 B2	CEM-B60	80	B2	28	0	20	UCT
T41-43 C1	CEM-B60	30	C1	28	0	20	UCT
T44-46 C1	CEM-B60	80	C1	28	0	20	UCT
T51-53 C2	CEM-B60	30	C2	28	0	20	UCT
T54-56 C2	CEM-B60	80	C2	28	0	20	UCT
T61-63 D1	CEM-B60	30	D1	28	0	20	UCT
T64-66 D1	CEM-B60	80	D1	28	0	20	UCT
T71-73 D2	CEM-B60	30	D2	28	0	20	UCT
T74-76 D2	CEM-B60	80	D2	28	0	20	UCT

[1] Sample name explanation: T21 B1 = Trial sample number 21 preparation method B1. The sample names can also just be named T21 when this is more convenient

[2] UCT = Unconfined Compression Test, CT = Computed Tomography

Total number of samples: 48

4 | Results

4.1 Initial clay testing

Table 4.1 shows the characteristic index values for the clay used in the laboratory samples and also the reference clay for the field stabilised material.

Table 4.1: Initial test results for non stabilised clay

Initial parameters								
Falling cone, undr.	c_u	52.0 kPa	Density, cylinder	ρ	2.08 g/cm^3	Liquid limit	w_L	32.40%
Falling cone, rem.	c_{ur}	6.0 kPa	Water content	w	30.02 %	Plasticity index	I_P	11.37%
Sensitivity	S_t	8.67	Plasticity limit	w_P	21.03%	Liquid index	I_L	0.79

4.2 Field samples

4.2.1 Description and preparation experiences

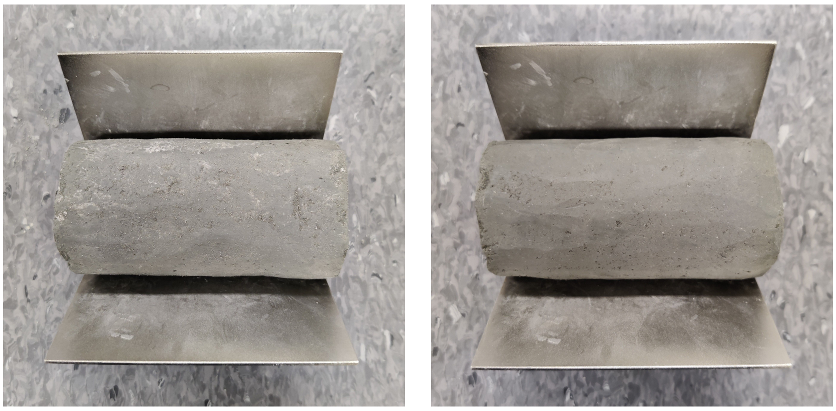
Of all the samples, material from the production piles with 50 kg/m^3 binder (type II) seems the firmest when preparing field samples. This probably has something to do with the longer curing time. It also contained most granules and pebbles, but this could be assumed to be local occurrences and not uniform for all piles. The rock particle content may nevertheless be an influencing factor when testing for unconfined compressive shear strength. Despite this, the sample is considered as relatively homogeneous.

The experimental piles with 30 kg/m^3 binder quantity (type I-a and type I-b), seemed somewhat poorer mixed, with pockets of non-disturbed clay surrounded by accumulations of binder. As shown in figure 4.1, the effect is strongest for the type I-b samples. The poor mixing could be due to a higher retrieval rate than for the 50 kg/m^3 (type II) piles and 80 kg/m^3 (type III) piles.

The experimental piles with 80 kg/m^3 binder quantity had significant difference in appearance. While type III-a seemed similar to the 30 kg/m^3 samples (type I-a and I-b) with respect to the mixing efficiency, the type III-b had no binder accumulations or internal differences in the material. The III-b material was also relatively easier to trim. The type III-a samples behaved more expectantly with respect to binder quantity and firmness. Figure 4.1 shows pictures of representative samples for each field type.

It is also interesting studying the cross sections of the samples. As we see, the pictures of type I-b and type III-a samples in figure 4.2 display how the binder accumulations are impacting the samples, in contrast to the homogeneous type III-b sample. The observation is interesting, revealing important information on the mixing efforts when stabilising clay in-situ. Keep in mind that the cross sections are photographed after the unconfined compression test and the break surface often follows the binder accumulations inside the samples in which makes them extra visible.

(a) Type I-a (F02, 30 kg/m^3) (b) Type I-b (F15, 30 kg/m^3) (c) Type II (F26, 50 kg/m^3)



(d) Type III-a (F34, 80 kg/m^3) (e) Type III-b (F46, 80 kg/m^3)

Figure 4.1: Pictures of a selection of field samples before testing

(a) Type I-b (F12)
Binder 30 kg/m^3
Retrieval rate 37 mm/rev

(b) Type III-a (F32)
Binder 80 kg/m^3
Retrieval rate 17 mm/rev

(c) Type III-b (F43)
Binder 80 kg/m^3
Retrieval rate 13 mm/rev

Figure 4.2: Field samples cross sections

4.2.2 Density and water content

The density of the samples is relatively similar for all types, but with tendency of slightly higher densities for lower binder quantity samples. This could be explained by the rapid reaction of quicklime making the material stiffer for higher amounts added, which will resist compaction and potentially creating more pores.

Table 4.2: Field samples water content and density

Samples			**Water content, w**		**Density, ρ (g/cm^3)**	
ID	Binder	Ret. rate	Avg.[1]	Dev.[2]	Avg.[1]	Dev.[2]
Type I-a	30 kg/m^3	37	23.41 %	0.04	2.10	0.03
Type I-b	30 kg/m^3	37	27.72 %	0.08	2.05	0.02
Type II	50 kg/m^3	20	23.78 %	0.09	2.04	0.04
Type III-a	80 kg/m^3	17	27.08 %	0.08	2.00	0.03
Type III-b	80 kg/m^3	13	26.21 %	0.03	2.02	0.04

[1] The CT-samples contribute to the average density but not water content

[2] Deviation is calculated as the coefficient of variation $COV = \frac{1}{\overline{x}}\sqrt{\frac{1}{n} \times \sum_{i=1}^{n}(x_i - \overline{x})}$

The water content deviates to a larger degree. Type II (production) piles have longer curing time and is expected to have a lower water content due to hydration. By that argument, one can also expect lower water content for a higher amount of binder because of more extensive hydration reaction. This is not the case as the 80 kg/m^3 samples (type III) have among the highest water content levels. For the 30 kg/m^3 samples, type I-a has a lower water content than type I-b. This could be explained by the poor mixing of type I-b samples, causing a lot of non-hydrated material and thus higher average water content.

4.2.3 Shear strength

The average obtained shear strength and the strain at failure from the unconfined compression tests, along with estimated stiffness module are presented in table 4.3. Figure A.1 to A.5 in the appendix A presents raw data for unconfined compression test for each sample. Please note that the results are corrected for a hypothetical zero strain at full sample contact at test start. This is done as the contact area between the load cell and the samples are not in fully engaged with respect to the cross-section area for the sample at start of the test due to irregularities at the sample top surface, which is causing an unnatural high increase rate for strain.

Type II samples, with 50 kg/m^3 binder quantity, seems to yield the highest shear strength with an average value close to 400 kPa. The deviation on the other hand is very high but is highly influenced by single samples. Primarily we would expect the samples with the higher binder amounts, as type III samples with 80 kg/m^3, to produce the highest shear strength. For type III-a this theory fits quite good even though the difference is not that high. An important factor is that the type II samples (50 kg/m^3) has at least ten weeks curing time, which is expected to give a notable higher shear strength than the standard 28 days curing.

Table 4.3: Field Samples Unconfined Compressive Shear Strength

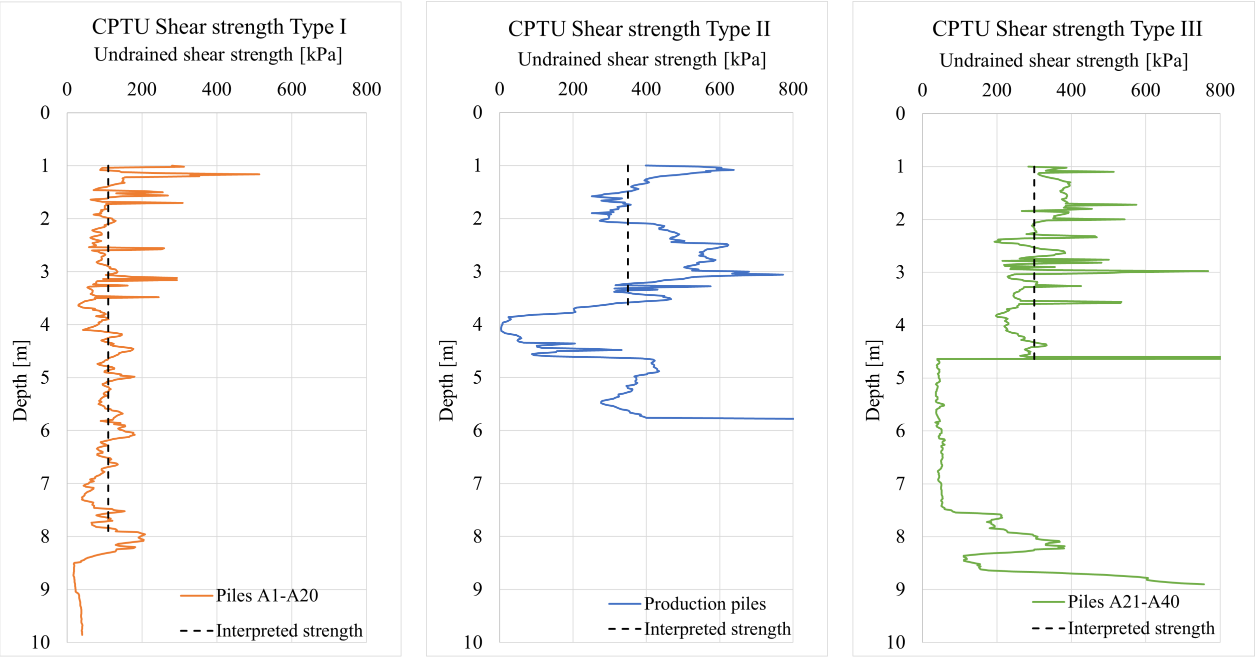
Sample type			**Shear strength, c_u**		**Strain at failure, ε_v**		**Stiffness, E_{50}**	
ID	Binder	Ret. rate	Avg.[1]	Dev.[2]	Avg.[1]	Dev.[2]	Avg.[1]	Dev.[2]
Type I-a	30 kg/m^3	37	222.3 kPa	0.16	1.30 %	0.51	60.6 MPa	0.48
Type I-b	30 kg/m^3	37	228.2 kPa	0.17	0.90 %	0.18	58.2 MPa	0.17
Type II	50 kg/m^3	20	394.1 kPa	0.31	1.27 %	0.34	80.0 MPa	0.47
Type III-a	80 kg/m^3	17	323.4 kPa	0.16	0.89 %	0.28	116.2 MPa	0.41
Type III-b	80 kg/m^3	13	183.5 kPa	0.42	1.47 %	0.42	42.6 MPa	0.71

[1] The CT-samples (F05, F15, F25, F35, F45) does not contribute to the average values

[2] Deviation is calculated as the coefficient of variation $COV = \frac{1}{\bar{x}}\sqrt{\frac{1}{n} \times \sum_{i=1}^{n} (x_i - \bar{x})}$

For the type III-b samples, the shear strength is on average 140 kPa lower than the type III-a samples, which should be equal. This is hard to explain. A theory is local differences in the original clay material when stabilising. Another theory is inconsistent stabilising work, for instance with lower or wrong binder feeding. Either way, the III-b samples are not considered representative for a binder content of 80 kg/m^3 in comparison with laboratory produced samples.

The type I samples with 30 kg/m^3 binder did on the other hand achieve an expected lower value of about 220 kPa for both a and b locations. This shows, despite the poor mixing observed at preparation, that the samples values have a lower spread, which strengthens the believe of representative values.



(a) Type I (30 kg/m^3, 4 piles)
(b) Type II (50 kg/m^3, 2 piles)
(c) Type III (80 kg/m^3, 4 piles)

Figure 4.3: CPTU readings from stabilised field columns

NGI also performed cone penetration tests (CPTU) at the field stabilisation site. The results from the test are presented as shear strength by cone resistance in figure 4.3. All the data is corrected for curing time with formula $\tau_f/\tau_{f,28days} = 0.3 \cdot ln(t)$ for t days (Åhnberg, 2006), giving a correction factor of 0.913 for 30 kg/m^3 (type I) and 80 kg/m^3 (type III) material, and 1.243 for the 50 kg/m^3 (type II) material.

Please note that the field samples for the unconfined compression test and the CT analysis are retrieved between 0 and 1 meter with respect to the CPTU graphs. Therefore, from the CPTU readings we could expect a shear strength of about 110 kPa for the 30 kg/m^3 (type I) samples, 350 kPa for the 50 kg/m^3 (type II) samples and around 300 kPa for 80 kg/m^3 (type III) samples. The strength readings are in the same range for the unconfined compressive strength of type II samples and the type III-a samples. As for the type I samples with 30 kg/m^3 binder, the measured shear strength from unconfined compression test is on average about 100 kPa higher, around double the values of the CPTU readings. However, as seen in figure 4.4, the variation of CPTU for type I is high, which also indicates a high variation in the data set for the different piles tested. For the 50 and 80 kg/m^3 (type II and III) piles the data seems slightly more stable.

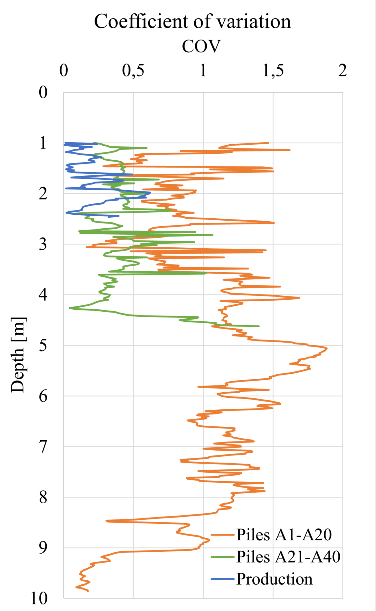


Figure 4.4: Coefficient of variation

For the RCP test, also performed by NGI, a design shear strength for the type II samples is set to 260 kPa uniformly throughout the columns. It is important when assessing the values for the RCP that the test is conducted two to three days after installation and therefore not representative for a 28-days strength. Compared to the CPTU readings the curing time is almost nine weeks difference for those piles and therefore the obtained shear strength is large.

Evaluated field material strength

Based on the obtained information the 28 day strength of the field stabilised material in this section a design shear strength is chosen and displayed in table 4.4.

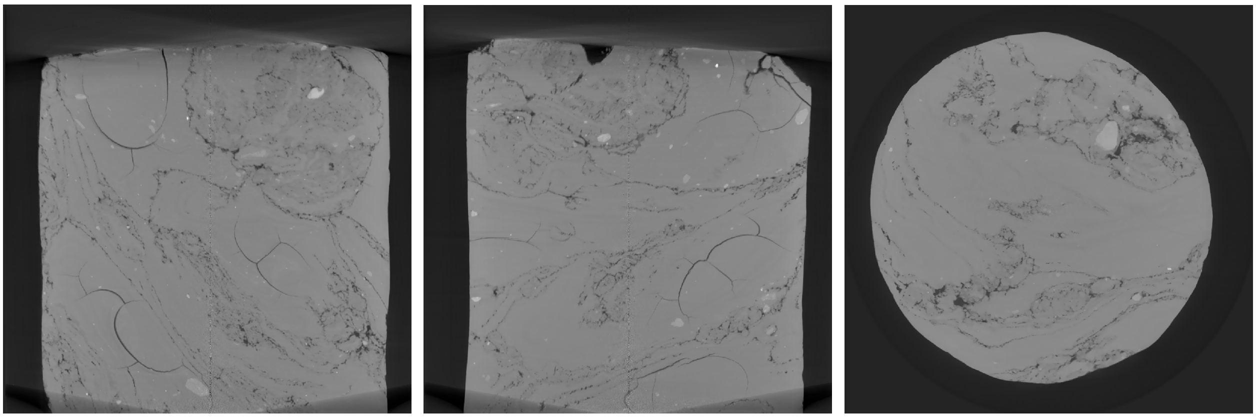
Table 4.4: Evaluated field design strength

Samples type			**Shear strength**
Type	Binder	Ret. rate	$c_{u,28}$
Type I	30 kg/m^3	37 mm/rev	220
Type II	50 kg/m^3	20 mm/rev	300
Type III	80 kg/m^3	17 mm/rev	320

The different values are mostly weighted with respect to the obtained unconfined compressive strength results. This is due to too few (especially for type II), or too high deviation in CTPU readings (especially for type I). For the type II material, the shear strength from the unconfined compression test was close to 400 kPa. However, when correcting for a curing time of 70 days, by equations 2.5 and 2.6 in section 2.5, the unconfined compressive shear strength is then 298 kPa or 309 respectively. For the type III strength, both CPTU strength and unconfined compressive strength are used to define the strength. However, the type III-b variation unconfined compressive strength measurements are excluded as this deviates with unrealistically low strength compared to the expected values. The RCP readings are not included as these are hard to convert due to very low curing time, and should only be used for verify a minimum strength value.

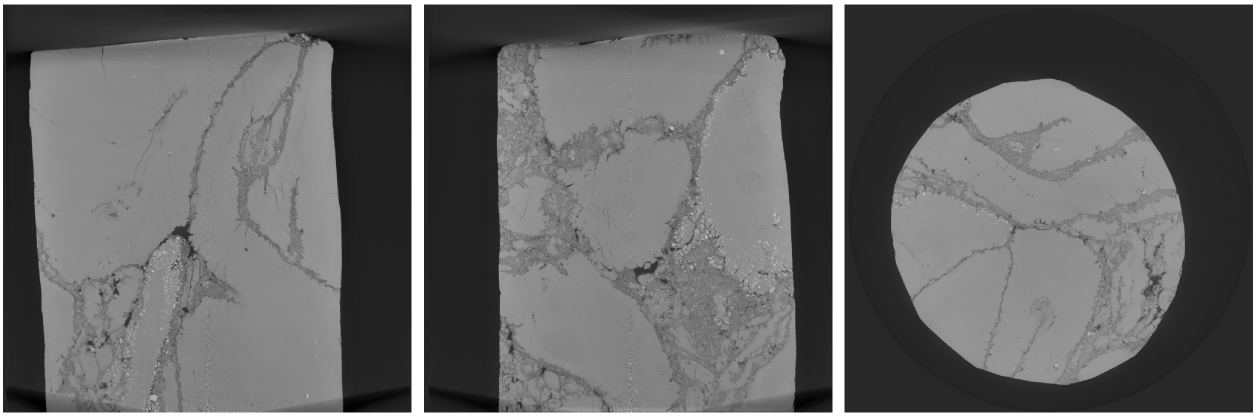
4.2.4 CT-analysis

The scans are performed on five field samples, one for each type with a representative (assumed non-deviating) sample. In the images from the CT-reconstructions (in figures 4.5 to 4.9), denser areas seems to appear brighter, which makes small stones the lightest spots in the images. Pores seems to appear darker. By that, density differences are visible within the images. Binder accumulations, which are considered more porous, are therefore appearing darker than clay material. The less internal colour differences in the samples, the more homogeneous the samples are. This observation fits good with the image of the type III-b samples which already is observed as more homogeneous. For the type I-b samples, which have a known prominent binder accumulations (e.g. seen in figure 4.1), the CT-images also shows the non-homogeneous features. The accumulation can clearly be found in every samples except for type III-b (sample F45). For sample F15 (figure 4.6), white spots along the accumulations are spotted, most likely being high density grains of binder.



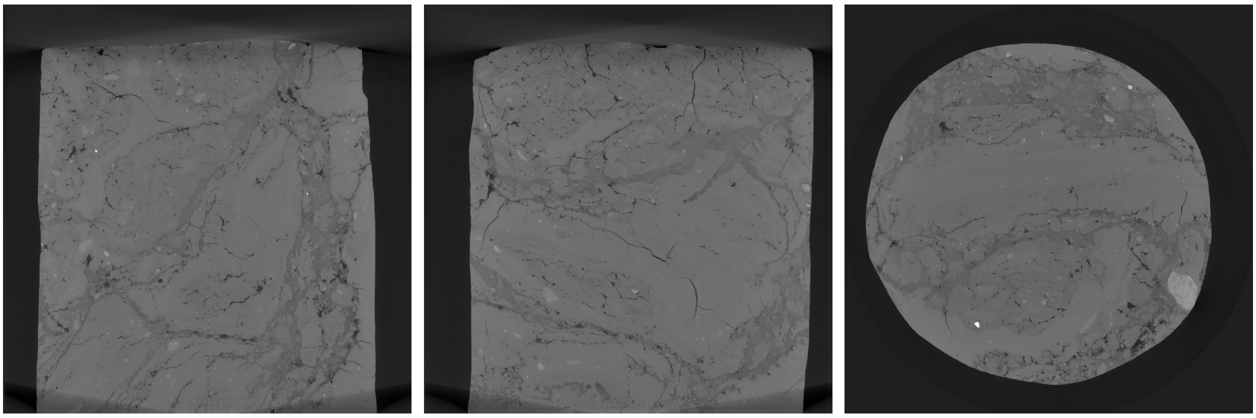
(a) xz-plane (b) yz-plane (c) xy-plane

Figure 4.5: CT-images F05 I-a (30 kg/m^3, ret. rate 37mm/rev, filtered)



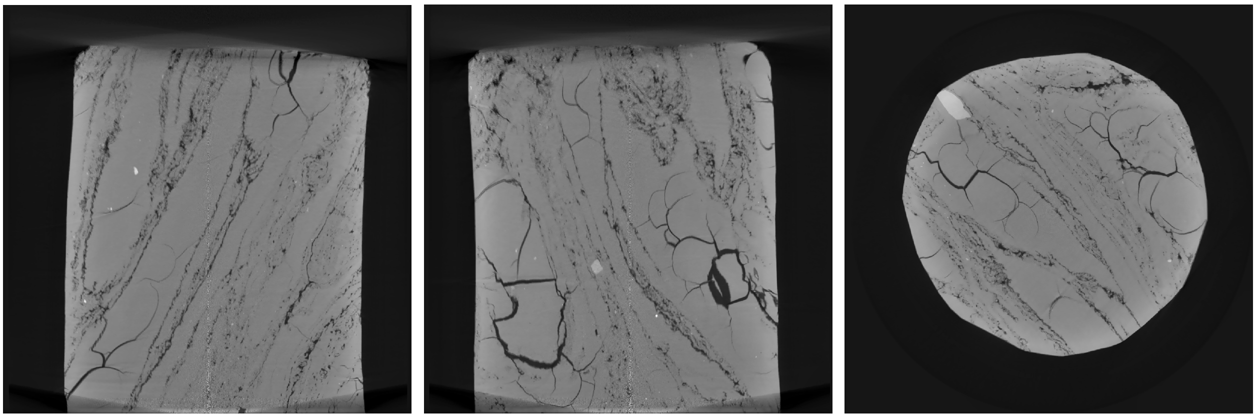
(a) xz-plane (b) yz-plane (c) xy-plane

Figure 4.6: CT-images F15 I-b (30 kg/m^3, ret. rate 37mm/rev, filtered)



(a) xz-plane (b) yz-plane (c) xy-plane

Figure 4.7: CT-images F25 II (50 kg/m^3, ret. rate 20mm/rev, filtered)



(a) xz-plane (b) yz-plane (c) xy-plane

Figure 4.8: CT-images F35 III-a (80 kg/m^3, ret. rate 17mm/rev, filtered)

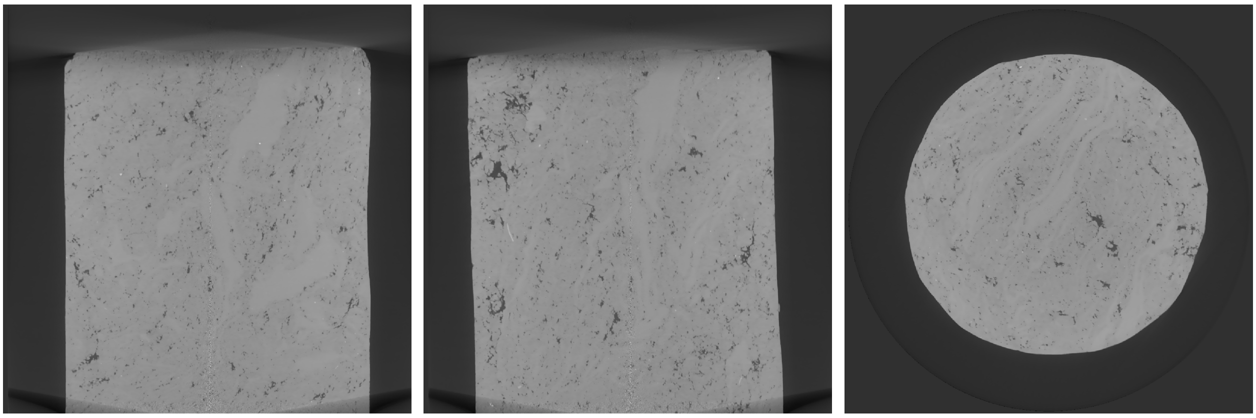

(a) xz-plane (b) yz-plane (c) xy-plane

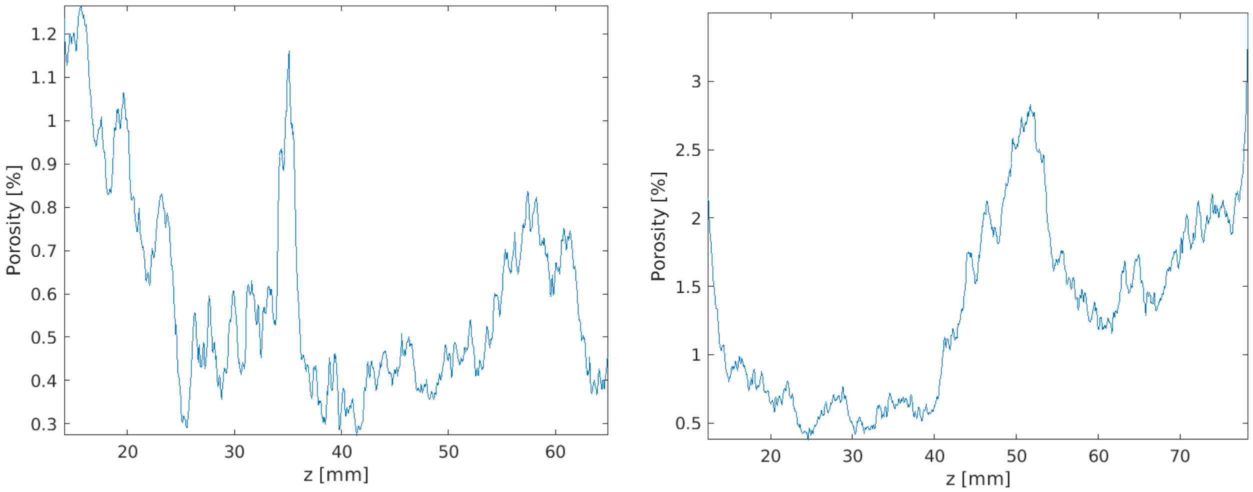
Figure 4.9: CT-images F45 III-b (80 kg/m^3, ret. rate 13mm/rev, filtered)

One of the more defining factors when comparing the field and laboratory sample is the porosity and density of the samples. From the images all the samples seems relative dense with no clear extensive pore systems. However, the images show a very fine distributed set of pores in the type III samples with the highest binder amount. This is a very interesting observation, especially if you consider the density measurements for the samples from table 4.2, which stated that the density on average is a few percent lower for the type III samples than the others.


Figure 4.10: CT-scans sample drying. F15 before and after CT

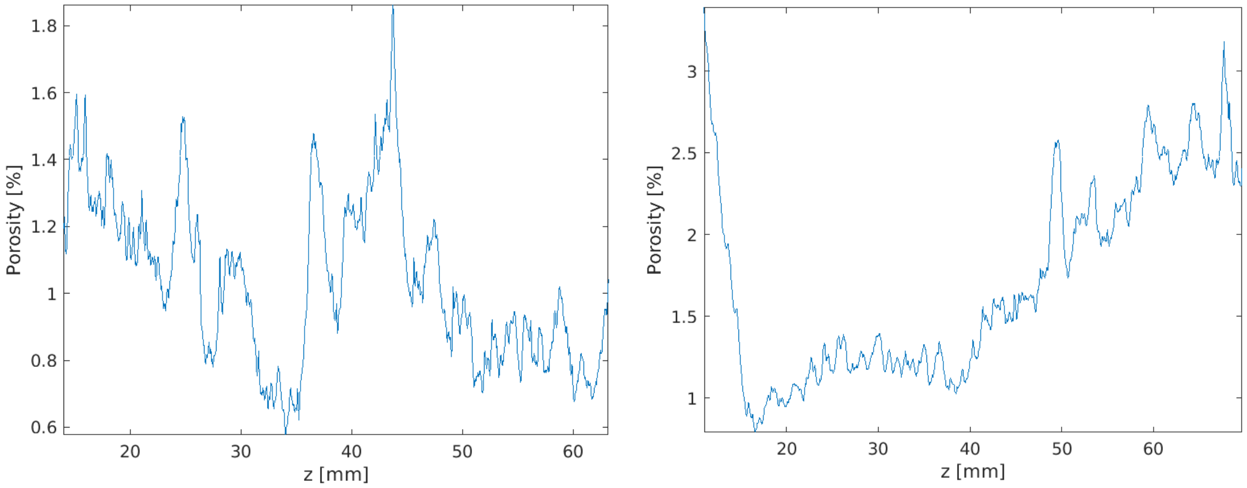
We can also spot some cracks in the samples. Some are potentially caused by drying of the samples inside the CT-scanner, especially those close to the sample surface. The samples are scanned for about two hours (some even longer) which may cause shrinkage cracks, as well as potentially influencing the image quality. The crack formation is clearly shown in figure 4.10 where they are visible at the surface of the sample after the scan. It is also an interesting observation that cracks seem to appear along the brittle (transition) area of binder accumulations. Despite that, F15 seems very influenced by cracks on the surface and the CT-images does not reveal any internal cracking. This is however not the case for samples F05 and F35 which CT-images clearly shows internal cracks while the surface seemed non-influenced. This might indicate that cracking of concrete does not only propagate from the surface from drying, but also appears in the weaker layers of the sample. Larger images from the CT-scans are presented in appendix A.3.1.

Porosity estimations done by analysing the CT-images (filtering and segmenting, see methods in appendix B), porosity estimations reveals very low values for all the scans. It is observed that the porosity is slightly higher for the high binder quantities samples than for the low binder. This correlates good with the obtained densities from earlier. The graphs in figure 4.11 may give the impression of huge variations throughout the samples height but the porosity interval in each samples only varies within 1 or 2 percentages, where peaks being very sensitive to small single pores.



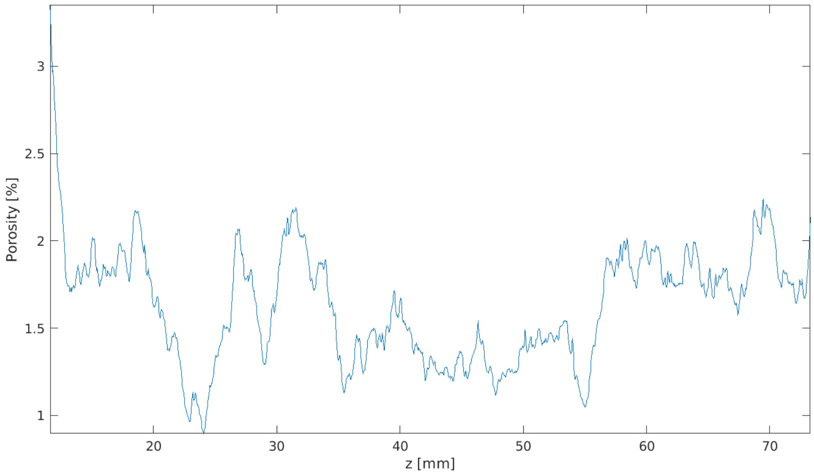
(a) F05 I-a (30 kg/m^3) Porosity ≈ 0.9%

(b) F15 I-b (30 kg/m^3) Porosity ≈ 0.6%



(c) F25 II (50 kg/m^3) Porosity ≈ 0.9%

(d) F35 III-a (50 kg/m^3) Porosity ≈ 1.7%



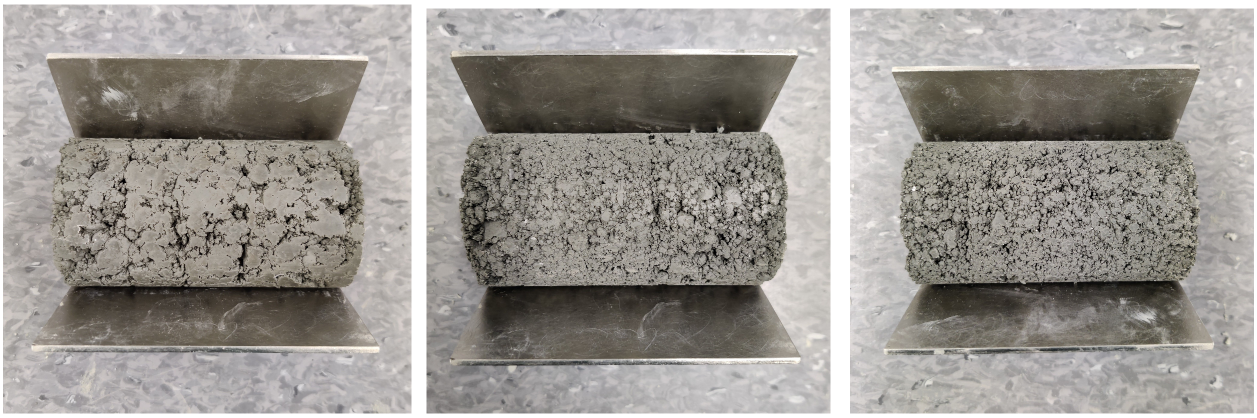
(e) L29 III (80 kg/m^3 - SVV) Porosity ≈ 1.6%

Figure 4.11: Field samples porosity estimations

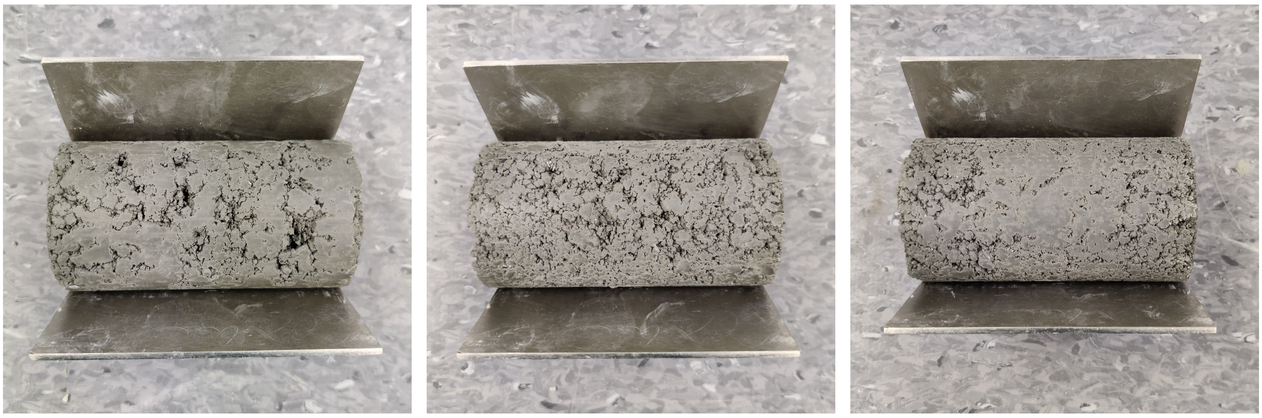
4.3 Laboratory samples

4.3.1 Description and preparation experiences

When preparing laboratory mixed samples, huge variation in material consistency is experienced for the changing binder amounts. Typically, the clay with the most binder behaves more brittle, causing a finer grained material, which on one hand is easier to handle during moulding but on the other hand seems stiffer to compact. Pictures from the samples (figure 4.12) clearly shows how the "clay grains" becomes smaller when adding more binder. This causes the pores to be finer and more evenly distributed trough out the sample. The type I samples with the lowest binder content (30 kg/m^3) have larger pores, also revealing the moulding layers to a higher extent.



(a) L04 I (30 kg/m^3 SVV) (b) L11 II (50 kg/m^3 - SVV) (c) L24 III (80 kg/m^3 - SVV)



(d) L06 I (30 kg/m^3 - NGF) (e) L16 II (50 kg/m^3 - NGF) (f) L27 III (80 kg/m^3 - NGF)

Figure 4.12: Laboratory sample pictures after curing

The pictures (figure 4.12) also reveal how the moulding technique affects the structure of the samples. The NGF-method (dynamic compression) seems to create a more compact sample than the SVV-method (static compression) for any binder amount.

4.3.2 Density and water content

The density measurements, presented in table 4.5, confirms the observation already mentioned in the last section. It can be observed that both higher binder content and use of the SVV moulding method gives lower density.

Table 4.5: Laboratory samples water content and density

Samples			**Water content, w**		**Density, ρ (g/cm^3)**	
ID	Binder	Method	Avg.[1]	Dev.[2]	Avg.[1]	Dev.[2]
L01-04 I	30 kg/m^3	SVV	29.49 %	0.01	1.71	0.03
L05-08 I	30 kg/m^3	NGF	30.01 %	0.01	1.82	0.02
L11-14 II	50 kg/m^3	SVV	26.90 %	0.02	1.64	0.03
L15-18 II	50 kg/m^3	NGF	28.52 %	0.02	1.80	0.01
L21-24 III	80 kg/m^3	SVV	27.81 %	0.02	1.61	0.01
L25-28 III	80 kg/m^3	NGF	28.02 %	0.03	1.79	0.02

[1] The CT-samples contribute to the average density but not water content

[2] Deviation is calculated as coefficient of variation $COV = \frac{1}{\bar{x}}\sqrt{\frac{1}{n} \times \sum_{i=1}^{n}(x_i - \bar{x})}$

The relationship between binder quantity and density is also shown in figure 4.13 for the two preparation methods. The results clearly indicates that density of the laboratory samples are highly dependent on the binder amount used. The reason for this might be the quick reaction between water and binder, especially quicklime, as discussed in section 2.3, which will quickly dry the clay before the moulding process has started. This explains that increasing binder amount yields a more brittle and fine-grained clay sample, which is harder to compact. Figure 4.13 also clearly illustrates the differences of the two preparation methods for laboratory samples, where the NGF-method on average has between 9-10% higher density than the SVV-method.

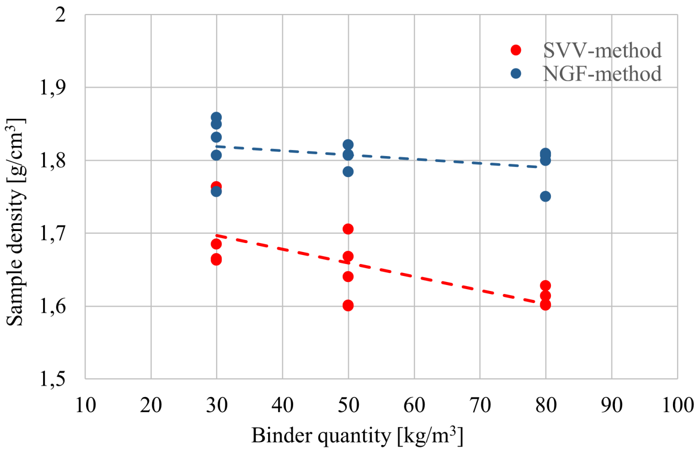


Figure 4.13: Laboratory samples - Density vs binder quantity

Compared to the "equivalent" field samples the density of the laboratory samples are relatively lower. For the SVV-method the laboratory samples are on average about 19.0% less dense, while for the NGF-method the same number is 11.7%.

4.3.3 Shear strength

The shear strength of the laboratory samples are given in table 4.6. As of previous experiences, it is expected that the shear strength should to a certain degree reflect the density observation and the binder amount with the shear strength being higher for more compact samples with high binder amount.

Table 4.6: Laboratory Samples Unconfined Compression Shear Strength

Sample type			**Shear strength, c_u**		**Strain at failure, ε_v**		**Stiffness, E_{50}**	
ID	Binder	Method	Avg.	Dev.[1]	Avg.	Dev.[1]	Avg.	Dev.[1]
L01-04 I	30	SVV	111.2 kPa	0.11	1.70 %	0.41	19.7 MPa	0.32
L05-08 I	30	NGF	159.5 kPa	0.26	1.72 %	0.46	34.0 MPa	0.53
L11-14 II	50	SVV	145.3 kPa	0.11	1.08 %	0.18	34.1 MPa	0.16
L15-18 II	50	NGF	235.4 kPa	0.15	1.20 %	0.04	54.0 MPa	0.16
L21-24 III	80	SVV	190.1 kPa	0.17	0.94 %	0.10	53.9 MPa	0.20
L25-28 III	80	NGF	303.7 kPa	0.09	1.74 %	0.36	71.9 MPa	0.37

[1] Deviation is calculated as coefficient of variation $COV = \frac{1}{\overline{x}}\sqrt{\frac{1}{n}\times\sum_{i=1}^{n}(x_i-\overline{x})}$

The unconfined compression strength is as expected increasing by the binder amount used. The strength increase is about 71% from 30 to 80 kg/m^3 for the SVV-method, while the same number is a bit higher for the NGF-method with about 90% strength increase.

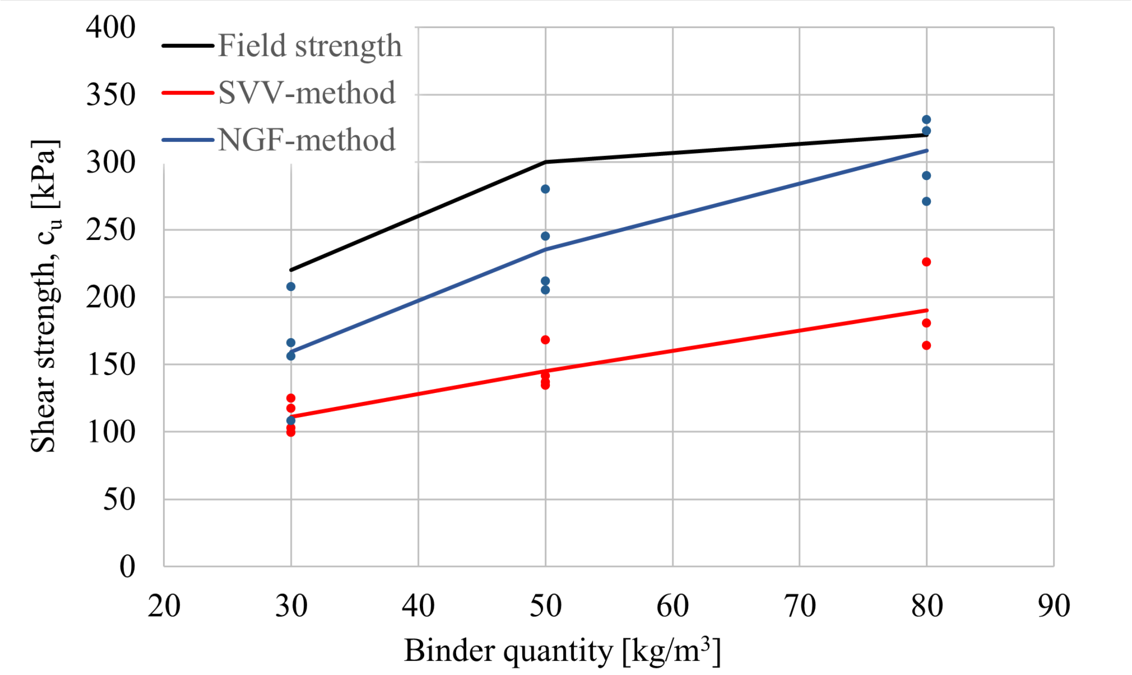
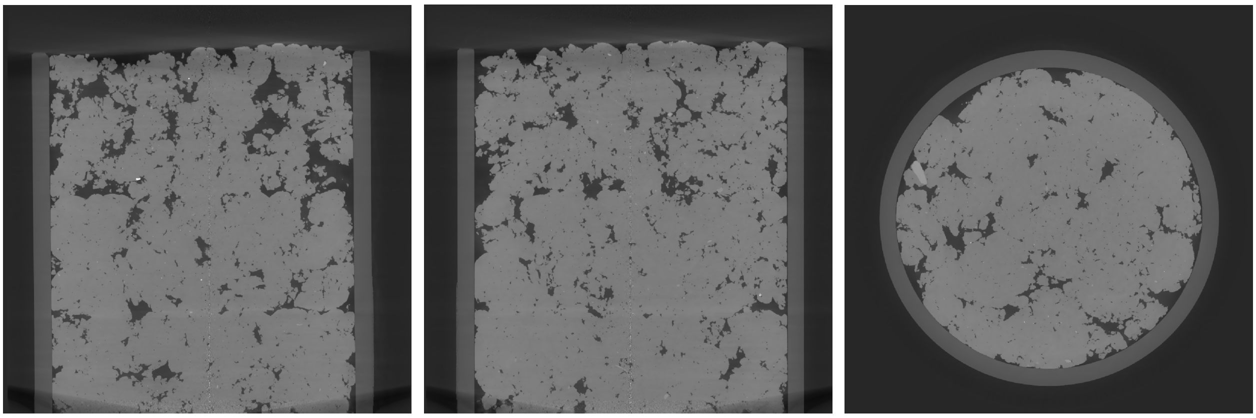


Figure 4.14: Laboratory sample unconfined compression shear strength vs binder quantity

Furthermore, the NGF-method on average yields 40-60 % higher strength than the SVV-method. The difference is clearly visible by figure 4.14 where it also can see how the strength of the laboratory samples compares to the field samples strength. The NGF-method more successfully represent the strength of the field conditions compared to the SVV-method. However, the NGF-method does not produce samples which describes field material with lower binder quantities good. For 80 kg/m^3 binder content the NGF-method creates samples with about 95% shear strength of the estimated field strength, while the same number is 78% for the 50 kg/m^3 samples and 72% for the 30 kg/m^3 samples. The obtained laboratory strength is therefore not sufficient to be used as accurate field design values.

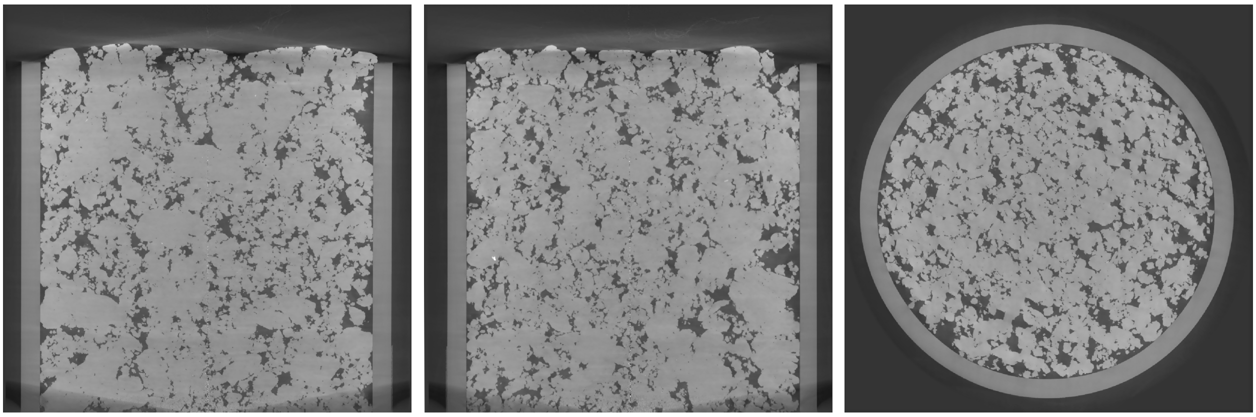
4.3.4 CT-analysis

The CT-images from the standardised SVV-method are presented in figures 4.15 to 4.17 (respectively 30, 50 and 80 kg/m^3 binder quantity) and for NGF-method in figures 4.18 to 4.20.



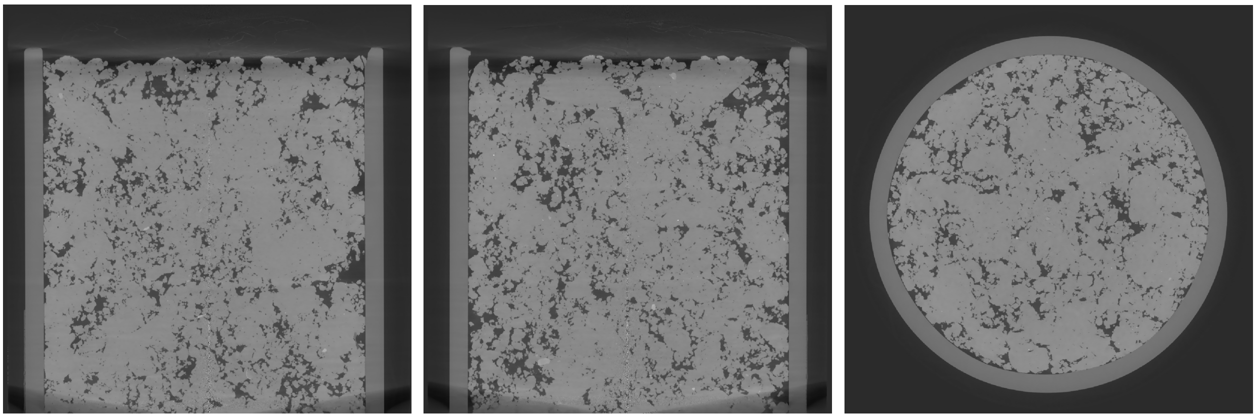
(a) xz-plane (b) yz-plane (c) xy-plane

Figure 4.15: CT-images L09 I (SVV-method, 30 kg/m^3)



(a) xz-plane (b) yz-plane (c) xy-plane

Figure 4.16: CT-images L19 II (SVV-method, 50 kg/m^3)

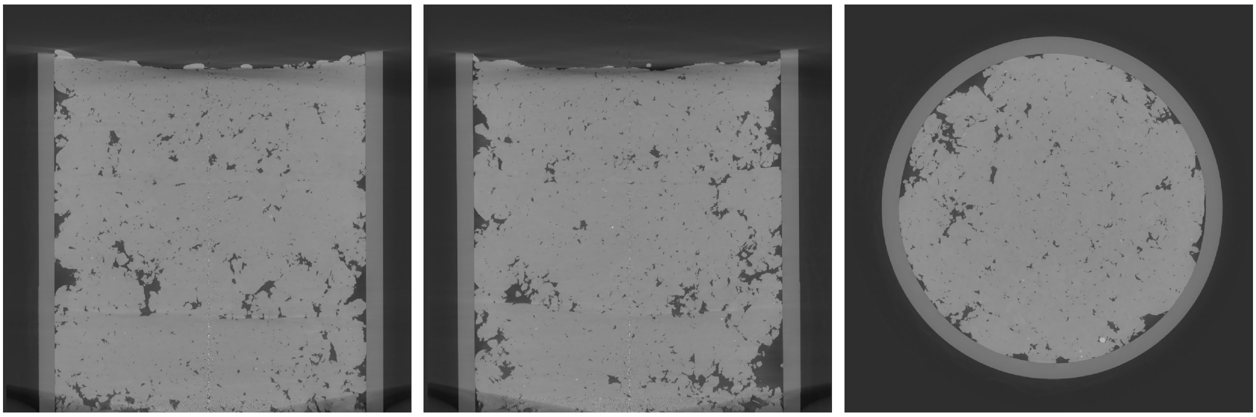


(a) xz-plane (b) yz-plane (c) xy-plane

Figure 4.17: CT-images L29 III (SVV-method, 80 kg/m^3)

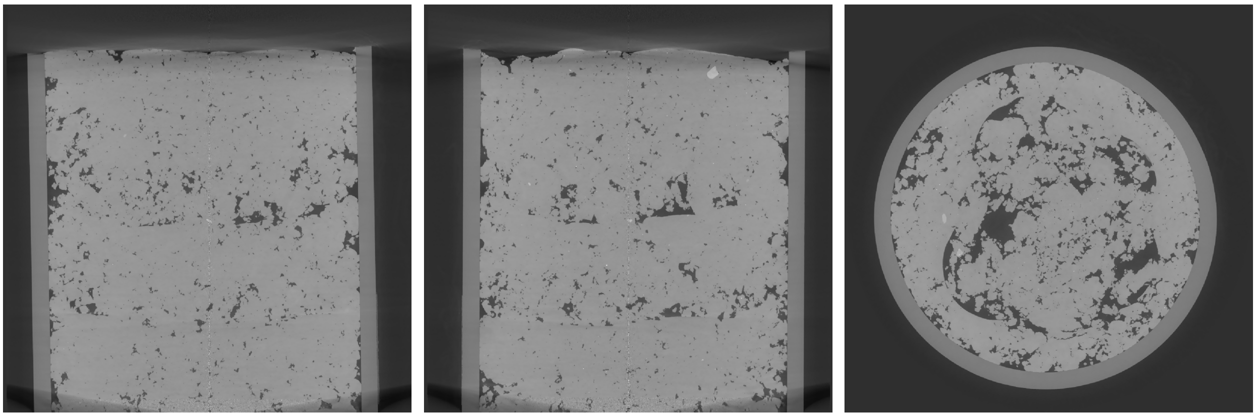
By studying the CT-images for the SVV- and NGF-methods a clear observation is made on the visual macro porosity; lower binder quantities yields larger pores with more inconsistent spread throughout the sample in contrast to higher binder quantity samples where the pore structures are finer and more spread. This observation is compliant with experiences from the visual inspection of the samples, described earlier. In the SVV-images especially, it is visible how the clay "grains" affect porosity with large grains for lower binder contents creates larger pores when compressed, and opposite for higher binder amounts.

Another important observation, which is best visualised in the NGF pictures (figures 4.18 to 4.20), is the detection on layering from the moulding process when preparing the samples. This can be seen as an apparently larger porosity in the bottom of each layer, most distant to the contact plane between clay and compression equipment. It is also visible, from images for both SVV- and NGF-method, how the moulding cylinders wall friction resist the compaction efforts creating samples with larger porosity along the sample edges. This should be considered when studying the sample images in figure 4.12.



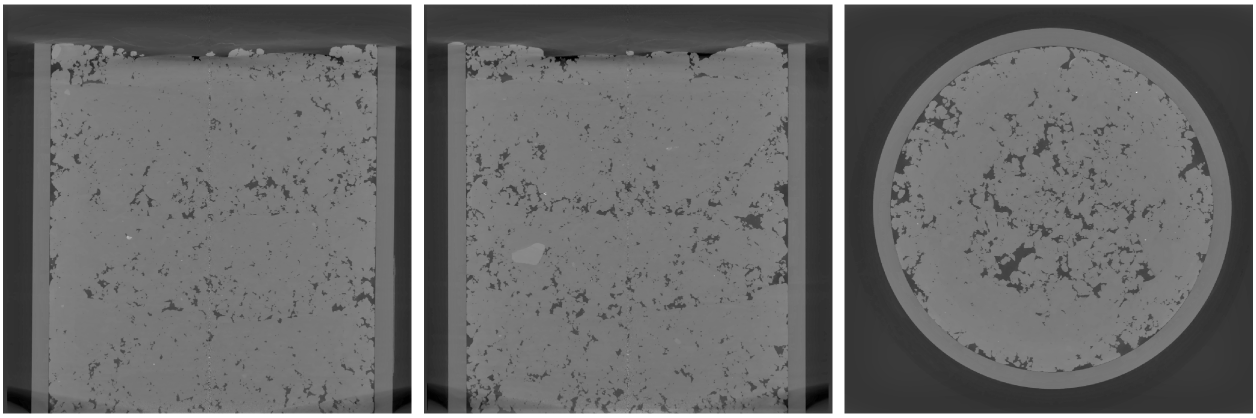
(a) xz-plane (b) yz-plane (c) xy-plane

Figure 4.18: CT-images L10 I (NGF-method, 30 kg/m^3)



(a) xz-plane (b) yz-plane (c) xy-plane

Figure 4.19: CT-images L20 II (NGF-method, 50 kg/m^3)

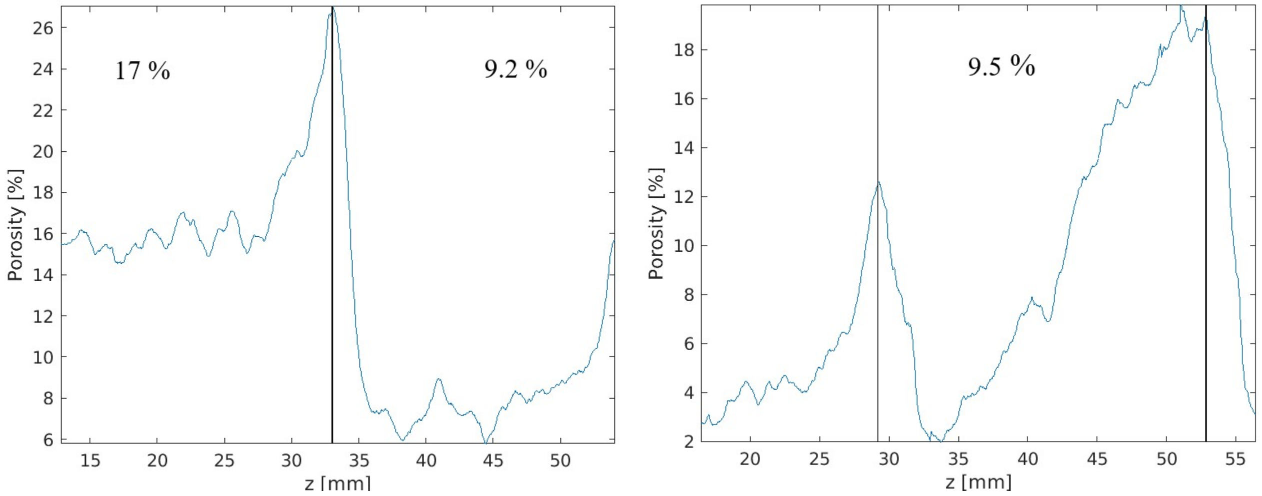


(a) xz-plane (b) yz-plane (c) xy-plane

Figure 4.20: CT-images L30 III (NGF-method, 80 kg/m^3)

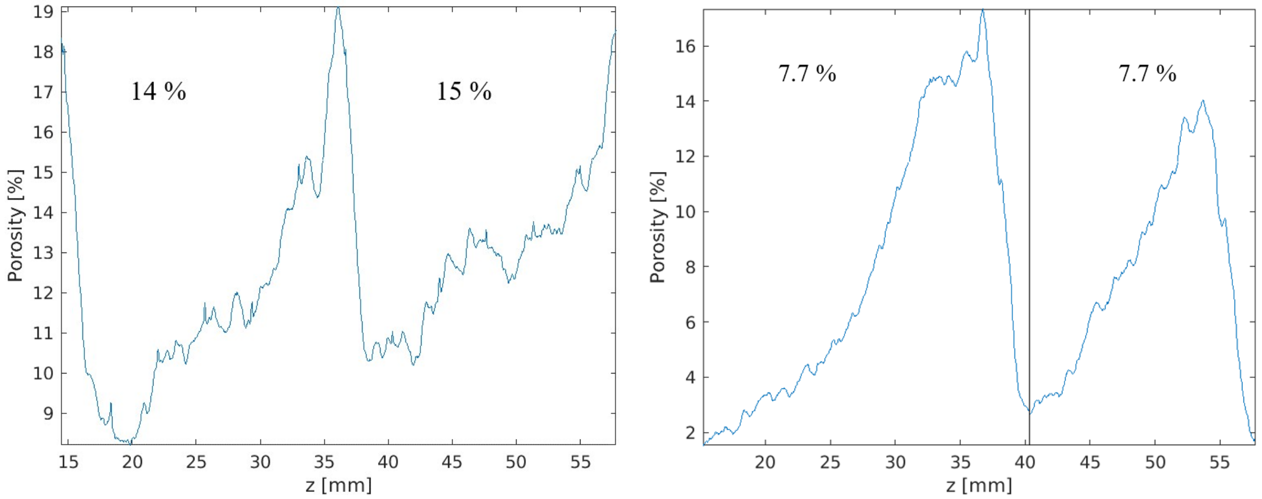
The sample layering is clearer when viewing the porosity estimation done by analysing the CT-images. Figure 4.21 displays the estimated porosity for the laboratory samples by millimetres from the top surface. The analysis aims to estimate porosity of at least two layers by adjusting the reference area between curve extremes. This to recreate the expected sample average porosity. In this analysis, the estimated error for the porosity values is less than 10% of the obtained results.

The SVV-method samples (L09, L19, L29) seems to yield a higher porosity overall, especially compared to the NGF-method samples (L10, L20, L30) where the porosity seems more modest and stable among the different binder quantities. For the SVV-samples the variation is large and distributions of porosity is very different implying poor layer repetitiveness by compaction. Nevertheless, for both methods, the porosity distribution curves clearly shows how the compaction creates internal differences within the samples with low porosity in top of each layer, closest to compaction surface, and very high porosity in the bottom of each layers.



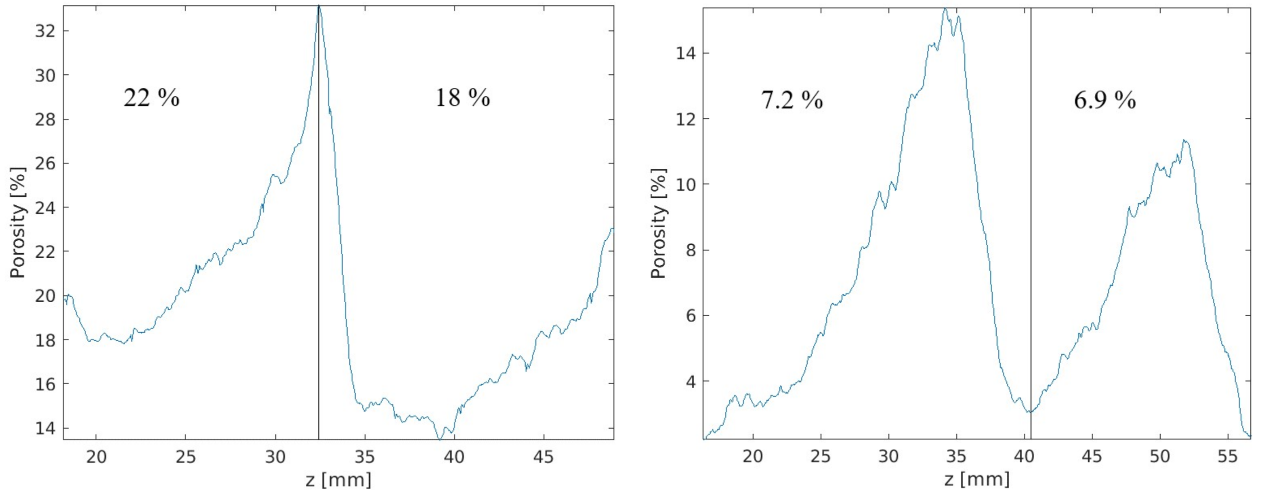
(a) L09 I (30 kg/m^3 - SVV) Porosity ≈ 15.0%

(b) L10 I (30 kg/m^3 - NGF) Porosity ≈ 7.7%



(c) L19 II (50 kg/m^3 - SVV) Porosity ≈ 13.0%

(d) L20 II (50 kg/m^3 - NGF) Porosity ≈ 8.5%



(e) L29 III (80 kg/m^3 - SVV) Porosity ≈ 20.0%

(f) L30 III (80 kg/m^3 - NGF) Porosity ≈ 7.1%

Figure 4.21: Laboratory samples porosity estimations

4.4 Trial laboratory methods

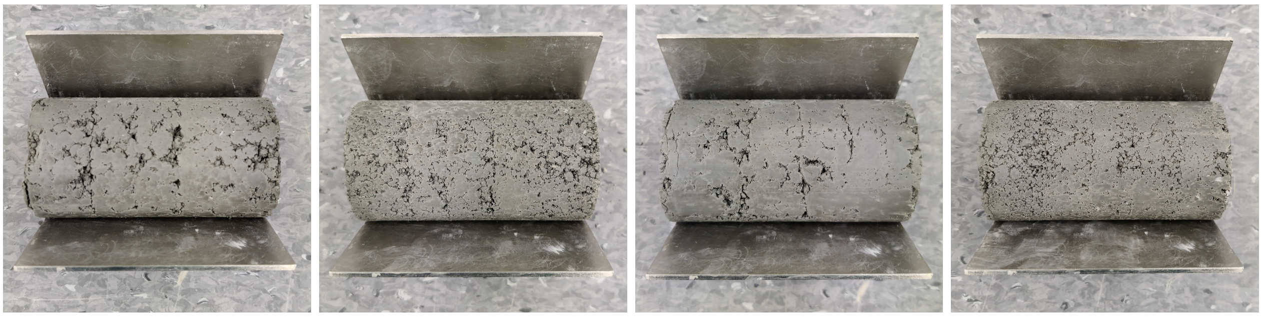
4.4.1 Approach A: Increasing compaction energy

A1: SVV-method with increased compression to 400 kPa.
A2: NGF-method with increased impact energy to 0.50-0.55 Nm.

Table 4.7: Results trial methods approach A

Sample type			**w**		ρ $[g/cm^3]$		$\mathbf{c_u}$ [kPa]		$\varepsilon_\mathbf{v}$		$\mathbf{E_{50}}$ [MPa]	
ID	Binder	Method	Avg.	Dev.[1]	Avg.	Dev.[1]	Avg.	Dev.[1]	Avg.	Dev.[1]	Avg.	Dev.[1]
T01-03	30	A1	28.0 %	0.01	1.89	0.03	201.6	0.15	1.72 %	0.29	66.4	0.32
T04-06	80	A1	26.6 %	0.01	1.80	0.04	358.4	0.10	1.06 %	0.26	115.6	0.29
T11-13	30	A2	28.1 %	0.01	1.93	0.01	198.1	0.07	2.04 %	0.29	37.6	0.16
T14-16	80	A2	26.5 %	0.01	1.91	0.01	354.0	0.14	2.37 %	0.33	60.1	0.18

[1] Deviation is calculated as coefficient of variation $COV = \frac{1}{\overline{x}}\sqrt{\frac{1}{n} \times \sum_{i=1}^{n}(x_i - \overline{x})}$



(a) Method A1 - 30 kg/m^3 (b) Method A1 - 80 kg/m^3 (c) Method A2 - 30 kg/m^3 (d) Method A2 - 80 kg/m^3

Figure 4.22: Sample images - Trial method A

By increasing the compaction energy for both SVV-method and NGF-method the density as well as the shear strength is significantly increased. For the A1-method, by increasing the compaction energy, the shear strength is on average over 80% higher than for the standard SVV-method. The strength increase for the A2 is about 20% higher than the standard NGF-method. Comparing the results with the field strength, as in figure 4.23, also reveals how both A1 and A2 methods fits the shear strength approximation better.

Another interesting find is that A1 and A2-methods yields very similar values for shear strength, in large contrast to the standard SVV- and NGF-methods. This despite that the A2-method, like NGF-method, produce denser samples. The densities are also greatly improving towards the actual field samples densities, but are on average still around 10% lower for the A1-method and 6% lower for the A2-method. However, both A1 and A2 methods shows potential of describing field samples more successfully than the existing methods.

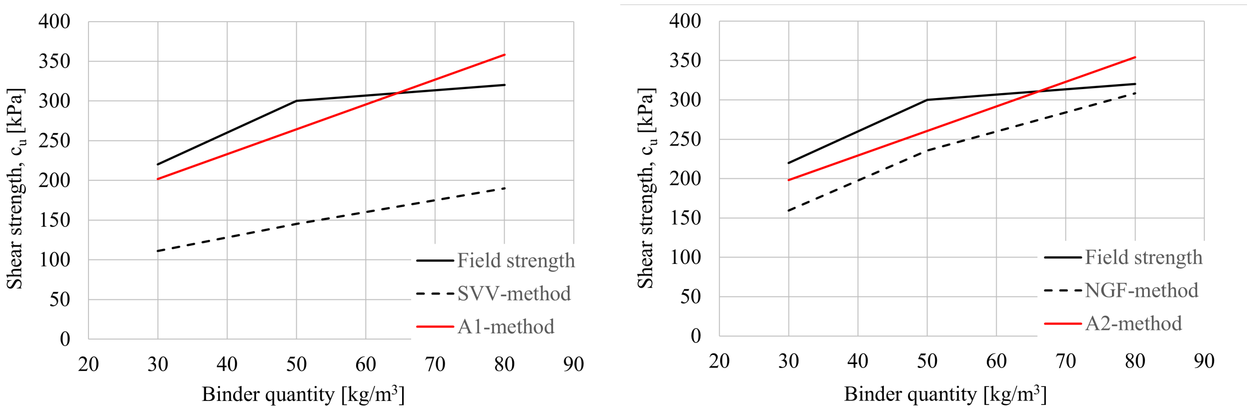


Figure 4.23: Method A1 and A2: Shear strength vs binder quantity

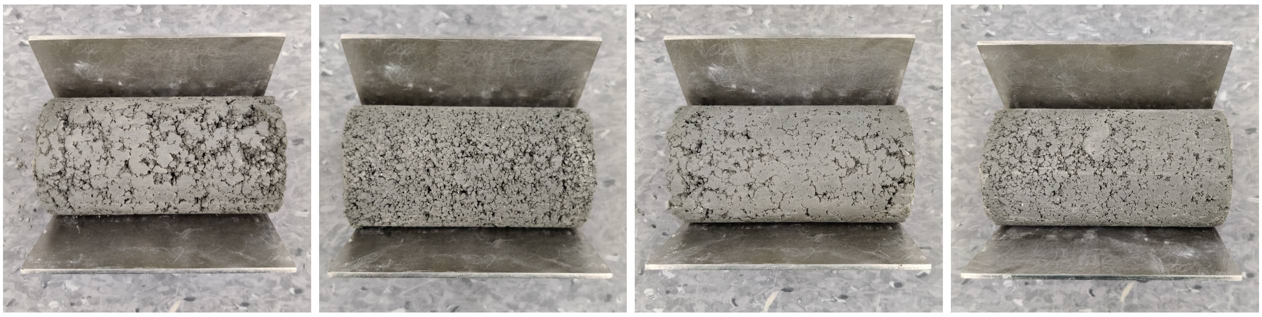
4.4.2 Approach B: Increasing number of layers

B1: SVV-method with 10 compaction layers.
B2: NGF-method with 10 compaction layers.

Table 4.8: Results trial methods approach B

Sample type			**w**		ρ $[g/cm^3]$		c_u [kPa]		ε_v		E_{50} [MPa]	
ID	Binder	Method	Avg.	Dev.[1]	Avg.	Dev.[1]	Avg.	Dev.[1]	Avg.	Dev.[1]	Avg.	Dev.[1]
T21-23	30	B1	28.5 %	0.01	1.77	0.01	156.4	0.12	1.08 %	0.13	42.1	0.29
T24-26	80	B1	26.8 %	0.02	1.67	0.03	228.5	0.10	1.44 %	0.29	45.3	0.41
T31-33	30	B2	28.3 %	0.01	1.89	0.01	215.4	0.13	1.83 %	0.31	35.1	0.28
T34-36	80	B2	26.6 %	0.02	1.87	0.03	452.3	0.20	1.50 %	0.21	96.7	0.47

[1] Deviation is calculated as coefficient of variation $COV = \frac{1}{\overline{x}}\sqrt{\frac{1}{n} \times \sum_{i=1}^{n}(x_i - \overline{x})}$



(a) Method B1 - 30 kg/m^3 (b) Method B1 - 80 kg/m^3 (c) Method B2 - 30 kg/m^3 (d) Method B2 - 80 kg/m^3

Figure 4.24: Sample images - Trial method B

Even though both B1 and B2 methods succeeds to increase the unconfined shear strength of the samples both fails to create samples that match that of the field material. As we can see in figure 4.25 the B1 method increases the shear strength of the laboratory samples with 20-40 % (depending on binder amount) compared to the standard SVV-method, but are still significantly lower than the field samples strength. Also, the B1-method almost totally fails to increase the sample densities, as were one of the main the intentions of increasing the number of compaction layers. This indicates that the compaction process is somewhat limited by the material resistance, and an insufficient compaction energy.

For the B2-method, the method also fails to fit the strength versus binder trend. Using 10 compaction layers seems reasonable for samples with lower binder content (more plastic clay behaviour), but as binder quantities increases, the shear strength of the laboratory samples becomes too high compared to the field samples. Despite this, it seems like different binder quantities requires a non uniform number of compaction layers to fit the field samples. This observation can be seen figure 4.25 where laboratory samples with 30 kg/m^3 binder correlates good with 10 compaction layers while laboratory samples with 80 kg/m^3 binder correlates good with 5 compaction layers from the standard NGF-method. This indicates that to create laboratory samples by the NGF-method approach, one has to increase the number of compaction layers when reducing the binder quantities.

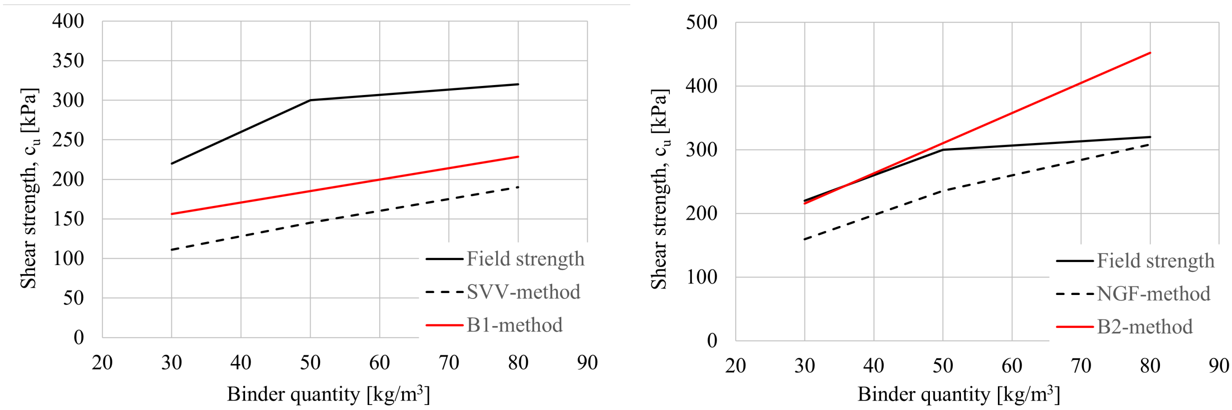


Figure 4.25: Method B1 and B2: Shear strength vs binder quantity

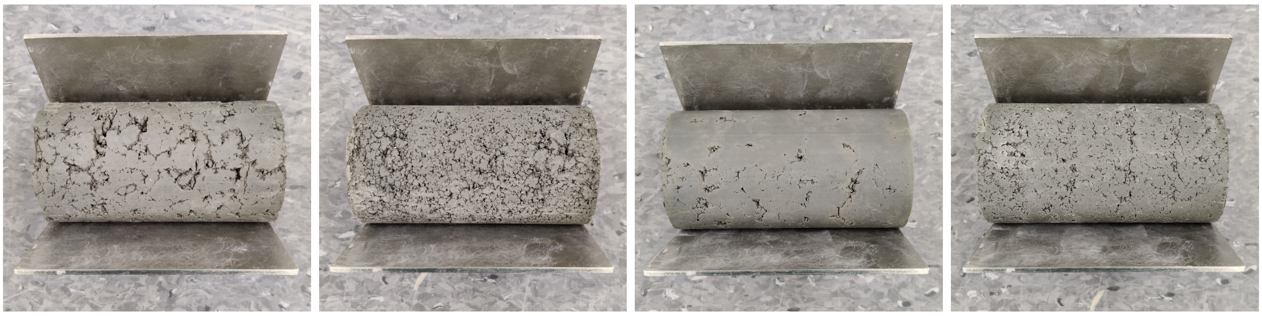
4.4.3 Approach C: Dynamic compaction with proctor-hammer

C1: Large Proctor-Hammer with compaction energy of 1.0 Nm.
C2: Large Proctor-Hammer with compaction energy of 2.0 Nm.

Table 4.9: Results trial methods approach C

Sample type			**w**		ρ $[g/cm^3]$		**c$_u$** [kPa]		ε_v		**E$_{50}$** [MPa]	
ID	Binder	Method	Avg.	Dev.[1]	Avg.	Dev.[1]	Avg.	Dev.[1]	Avg.	Dev.[1]	Avg.	Dev.[1]
T41-43	30	C1	29.5 %	0.01	1.84	0.02	194.1	0.19	1.48 %	0.34	34.4	0.41
T44-46	80	C1	27.1 %	0.04	1.82	0.01	415.9	0.17	1.34 %	0.07	85.1	0.20
T51-53	30	C2	28.5 %	0.03	1.92	0.02	203.8	0.05	1.76 %	0.08	37.7	0.29
T54-56	80	C2	27.9 %	0.01	1.93	0.01	498.5	0.17	1.57 %	0.04	90.7	0.14

[1] Deviation is calculated as coefficient of variation $COV = \frac{1}{\bar{x}}\sqrt{\frac{1}{n} \times \sum_{i=1}^{n}(x_i - \bar{x})}$



(a) Method C1 - 30 kg/m^3 (b) Method C1 - 80 kg/m^3 (c) Method C2 - 30 kg/m^3 (d) Method C2 - 80 kg/m^3

Figure 4.26: Sample images - Trial method C

For lower binder contents the fitting against field samples is relatively good, surprisingly similar for both 1 and 2 Nm compaction energy. Larger compaction energy will create samples with higher densities, and also higher shear strength with increasing binder content. Considering shear strength both compaction energies 1 and 2 Nm seems to high to match the field material properties for the highest binder content. For use of this method, a solution would be to find a relation to reduce the compaction energy with increasing binder content.

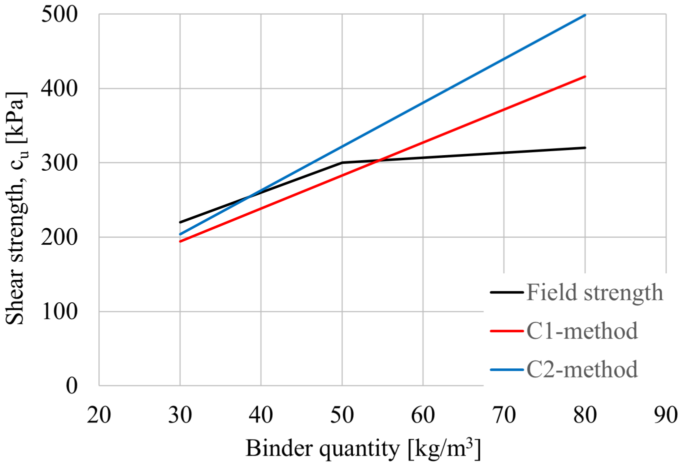


Figure 4.27: Method C1 and C2: Shear strength vs binder quantity

On the other hand, the method does not offer any new advantages on the results compared to the already existing static compaction NGF-method, neither in form of better fit in shear strength, density or the respective deviation in the samples. The method is on the other hand easier and more efficient to manually execute in the laboratory.

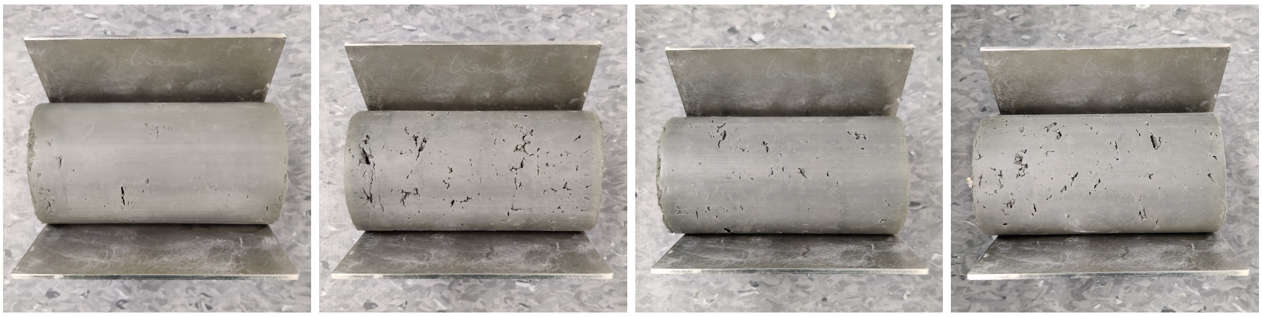
4.4.4 Approach D: Increasing water and binder content

D1: Doubled water content with rodding technique.
D2: Double water and binder content with rodding technique.

Table 4.10: Results trial methods approach D

Sample type			**w**		ρ $[g/cm^3]$		$\mathbf{c_u}$ [kPa]		$\varepsilon_\mathbf{v}$		$\mathbf{E_{50}}$ [MPa]	
ID	Binder	Method	Avg.	Dev.[1]	Avg.	Dev.[1]	Avg.	Dev.[1]	Avg.	Dev.[1]	Avg.	Dev.[1]
T61-63	30	D1	41.1 %	0.01	1.82	0.00	74.7	0.06	3.44 %	0.14	6.7	0.01
T64-66	80	D1	40.3 %	0.01	1.82	0.00	200.0	0.08	1.94 %	0.14	35.5	0.27
T71-73	30	D2	41.5 %	0.00	1.81	0.00	140.8	0.07	2.60 %	0.32	15.2	0.49
T74-76	80	D2	39.4 %	0.01	1.82	0.00	276.0	0.04	1.81 %	0.17	63.0	0.51

[1] Deviation is calculated as the mean absolute deviation coefficient of variation $COV = \frac{1}{\bar{x}}\sqrt{\frac{1}{n} \times \sum_{i=1}^{n}(x_i - \bar{x})}$



(a) Method D1 - 30 kg/m^3 (b) Method D1 - 80 kg/m^3 (c) Method D2 - 30 kg/m^3 (d) Method D2 - 80 kg/m^3

Figure 4.28: Sample images - Trial method D

By adding water to the clay material during mixing, the consistency changes drastically. For both with and without increased binder content it were easier to create samples with less macro porosity. However, the compactibility of the material decreases with more liquid behaviour. This is also shown in the density measurements in table 4.10 where the density seems to settle around 1.82 g/cm^3, indicating that the increased water content is controlling the largest obtainable density.

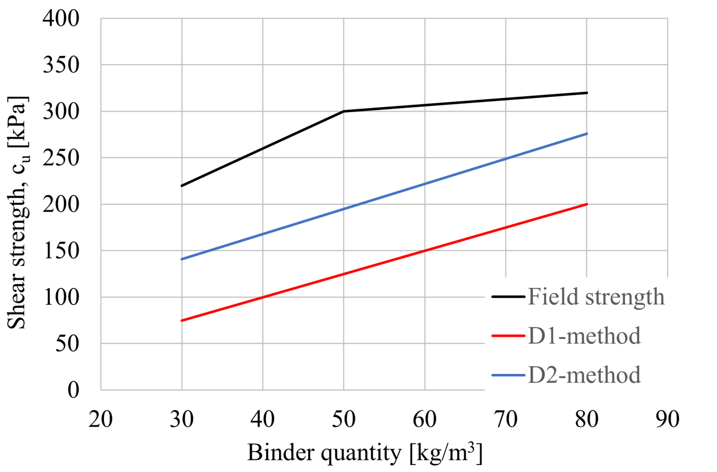


Figure 4.29: Method D1 and D2: Shear strength vs binder quantity

For the strength measurements, figure 4.29 shows a bad correlation for both D1 and D2 samples with the field strength for all binder contents. The observation is expected for the D1 samples, but as the graph shows, keeping the w/c-ratio equal to the original for D2 samples does not improve the shear strength even though the sample seemed less porous. With that in mind, considering the density and the shear strength of the D2 samples, they are in the range of what be expected relative to the other methods with no larger deviation. This strengthens the theory on the importance of the water-binder ratio being vital for the strength. Still, this theory is yet to be proper confirmed.

5 | Discussion

5.1 Field material and mixing quality

As discovered, the field material is highly influenced by inconsistent material properties. This is supported by large deviation in strength measurements from unconfined compression tests and CPTU readings, as well as preparation observations and CT-images. It seems like the mixing effort, and the ability to mix the binder and clay will influence the material properties values and the uniformity of them.

Table 5.1: Field material T-rate

Type	T-rate	
	Avg.[1]	Dev.[2]
Type I-a	229	42
Type I-b	255	94
Type II [3]	(400)	-
Type III-a	489	35
Type III-b	727	281

[1] Based on average column retrieval rate
[2] Mean absolute deviation
[3] Actual mixing data not available

Both the field sample pictures in section 4.2.1 and the CT-images in section 4.2.4 clearly shows the inefficiency of the in-situ mixing efforts. For the field samples examined in this thesis, the T-ratio as described in the theory section 2.1 is presented in table 5.1. Larsson et al. (2005a) stated that material stabilised with T-rate above 400 could be expected homogeneous, with lower variations. The values displayed in the table seems quite accurate concerning this statement. However, as seen for type III-a (80 kg/^{3}, location a), accumulations are still prominent in the material, but to a lower extent. Since the mixing quality varies that much, and as mentioned most likely is a major factor for the strength in which laboratory prepared samples are to match, making prediction models on the strength of field stabilised clay becomes even harder. As stated in Larsson et al. (2005b), an important step for stable strength predictions is to increase the T-rate as observations points towards a linear decrease of coefficient of variation with log(T). This means that the obtained strength in each column is deviating less when the T-rate is high, which argues that the highest stabilising potential is reached. This does however not imply that the type III-b (80 kg/m^3, location b) field material, with the highest T-rate, is better representing the highest potential strength as the believe is still that the obtained strength is increasing with higher mixing effort (Kitazume & Terashi, 2013) and type III-b has very low shear strength on the binder amount used.

For this thesis, stabilised material for a soil level with very stiff clay is examined. This may have enhanced the effect of poor mixing as the remoulded shear strength of the clay is relatively high ($\approx$ 6 kPa). Normally, deep stabilisation with lime and cement is performed on far more sensitive clay with remoulded shear strength often below 1 kPa. This implies that the clay stiffness may have been a limiting factor on the mixing process. Nevertheless, the mixing equipment is also limiting the mixing quality as the retrieval rate had to be increased to compensate for a minimum binder flow rate through the mixing tool nozzle. This may explain the extensiveness of the binder accumulations in the samples with lowest binder quantities. With that said, binder accumulations are not a rare observation. For instance both Larsson et al. (2005a) and Larsson (2001) also states how binder accumulations and differences in binder contents across piles cross sections are prominent in Swedish soil stabilisation projects. A factor that may be important is that both Norway and Sweden uses the same dry deep stabilisation method as standard.

5.2 Comparison of field and laboratory produced samples

It is obvious when studying the results that the preparation methods used for laboratory trial mixing in Norway have greatly limited abilities to represent the real qualities in field stabilised material. In addition to the issues addressed in the previous section, a few other observations are also made.

One interesting observation is the strongly deviating densities in the field and laboratory produced samples. As field samples seems more solid, with relatively low porosity for all binder quantities used, the laboratory samples are subject to an extensive pore structure, where binder quantities used are governing both the porosity and the fineness of the pores. For both field and laboratory produced samples the densities are larger for the low binder quantities used. However, the effect for the laboratory samples are much greater. The reasons could be many, but are mainly seen as a result of the mixing process. Since the clay stabilised in field is below the surface at the mixing time, the availability of air is lower than for laboratory stabilised clay. As observed in laboratory, clay stabilised with binder has the tendency to stiffen and crumble into grains when mixing. The effect increases with the binder content giving smaller and even stiffer grains. When trying to compact the clay, the moulding methods are not able to compress the grains and removing any air. For field samples, the air is no present in which clay material is naturally compressed together.

In figure 5.1, all laboratory produced samples (except wet preparation methods D1 and D2) and average field samples are plotted with densities against shear strength for binder quantities 30 and 80 kg/m^3. As seen, the tendency of the laboratory samples are linear correlation between density and shear strength. The field samples on the other hand seems to deviate from this trend. This suggest that the shear strength potentially could be higher for the field sample than is obtained during field mixing. Alternatively, poor mixing might be the cause of unnaturally high field material density. An additional option is sample disturbances from the preparation and the influence on testing method, especially considering unconfined compression test. In field samples, weak accumulation surfaces may yield lower strength in field samples compared to in situ strength where the material is confined. Either way it is believed that the field material strength is severely influenced by poor mixing.

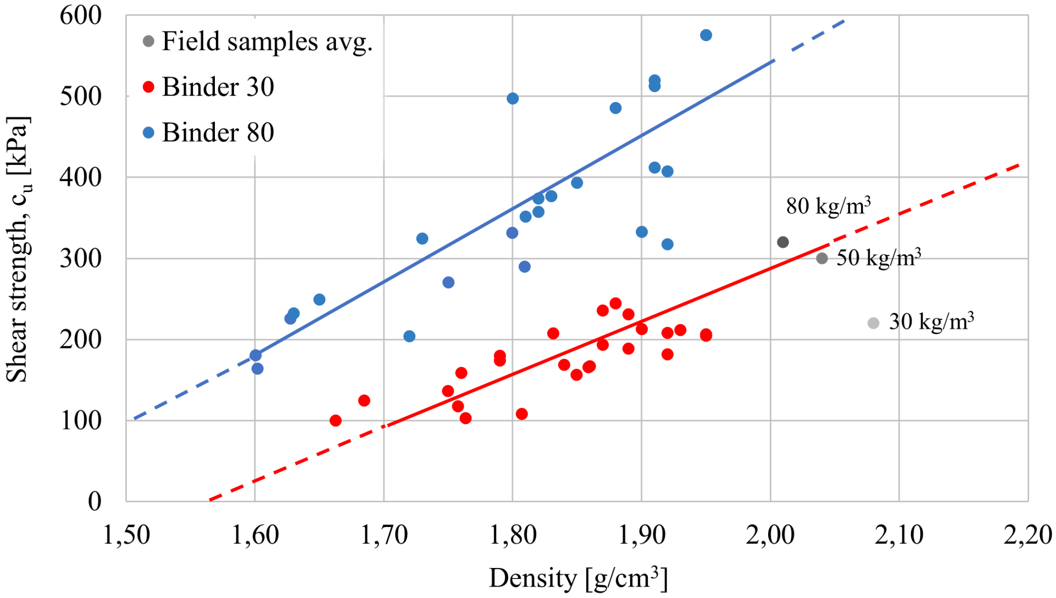


Figure 5.1: Laboratory and field samples - Shear strength vs density fitting for different binder quantities.

Another major concern is that the material variation is also very large for the field material and could limit the representativeness of the laboratory trial mix. As observed in chapter 4.2.3 the coefficient of variation is from 0.16 to 0.42 for the field unconfined compressive shear strength, which is relatively high. The same is for instance observed in the piles CPTU readings where the coefficient of variation for the readings is even higher. This makes it hard to settle on a characteristic strength for field stabilised material. As seen for the difference in unconfined compressive shear strength of type III-a and type III-b samples, material with seemingly similar configuration can vary a lot in performance both in strength, but also in mixing quality. This uncertainty would be limited when increasing the number of sampled columns and samples per column. For this paper, only one or two columns are tested per variation for the unconfined compression test which is causes large uncertainties. In addition, the field type II (50 kg/m^3) samples have considerably longer curing time than the other samples. This raises the uncertainties in the real 28-days strength for this material, especially when observing that the strength of the 50 kg/m^3 binder quantity samples is equal to, or even higher than the 80 kg/m^3 samples (especially CPTU readings). This may be raise concerns on the accuracy of curing time to strength correction models used for both CPTU and unconfined compressive strength. Still, other explanations as local differences in stabilised clay material or similar may be also be the cause.

When it comes to the laboratory samples, we see that the preparation method obviously impacts the properties of the laboratory samples. As for the SVV-method versus NGF-method, the NGF-method is clearly better at describing the field properties, not only in sense of density and shear strength, but also that the porosity is lower and more evenly distributed in the samples. How the layering in the samples is impacting the fracture mechanisms of the samples when testing in unconfined compression test is still unclear but as experiences from laboratory work shows, samples often break in the transition planes between to layers. This indicates that zones with higher porosity acts as weaknesses in the samples, at least as part of transition zones with denser material. On the other hand, as the preparation methods are made with intention of testing uniaxial by unconfined compression test, the assumption is that the sample layering pose little or no impact on the obtained strength as no fracture surfaces from any unconfined compression tests seemed influenced by horizontal weak layers. However, it is imaginable that test where anisotropic behaviour is important, for instance for triaxial compression tests, the strength might be somewhat influenced by the preparation methods.

Among other findings worth mentioning, the stiffness (from unconfined compression test) in both the field and laboratory samples seems linearly increasing with increasing binder quantity used for stabilising. This is clearly visible in figure 5.2. Note that the deviating field value for type III-b samples, which showed large deviation in strength properties, do not follow the trend for stiffness either.

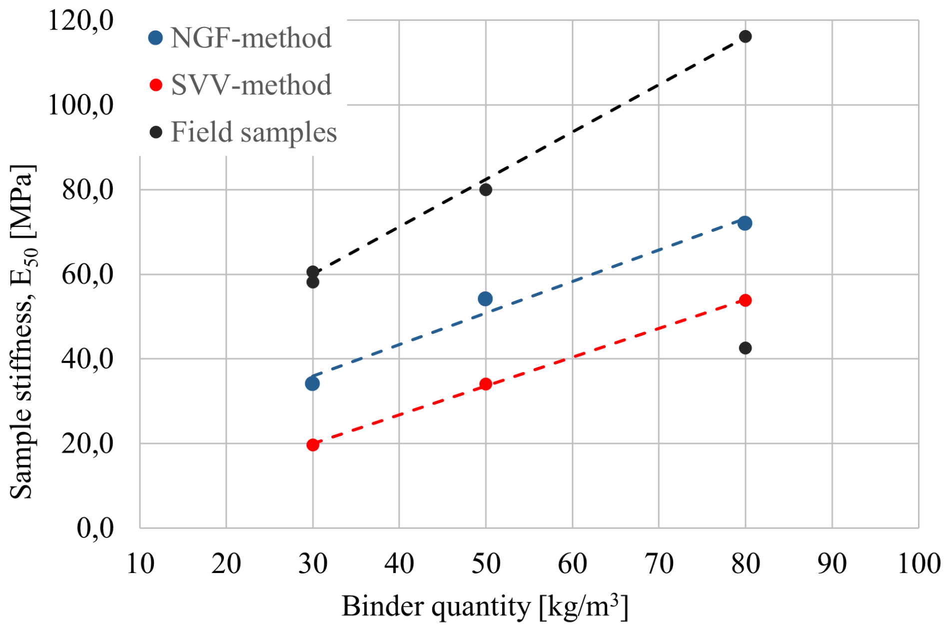


Figure 5.2: Sample stiffness versus binder quantity

Another observation concerning sample stiffness is that it seems linearly increasing with the sample density. This is clearly visible in figure 5.3. Unlike shear strength versus density (in figure 5.1) the field samples stiffness versus density seems to follow the same linear trend as the laboratory samples. This observation is done even with known binder accumulations and poor mixing of the field samples. It is important to notice that the accumulation material is to a certain degree hydrated as they seem stiff and brittle, and therefore most likely will contribute to a certain average stiffness in the field samples.

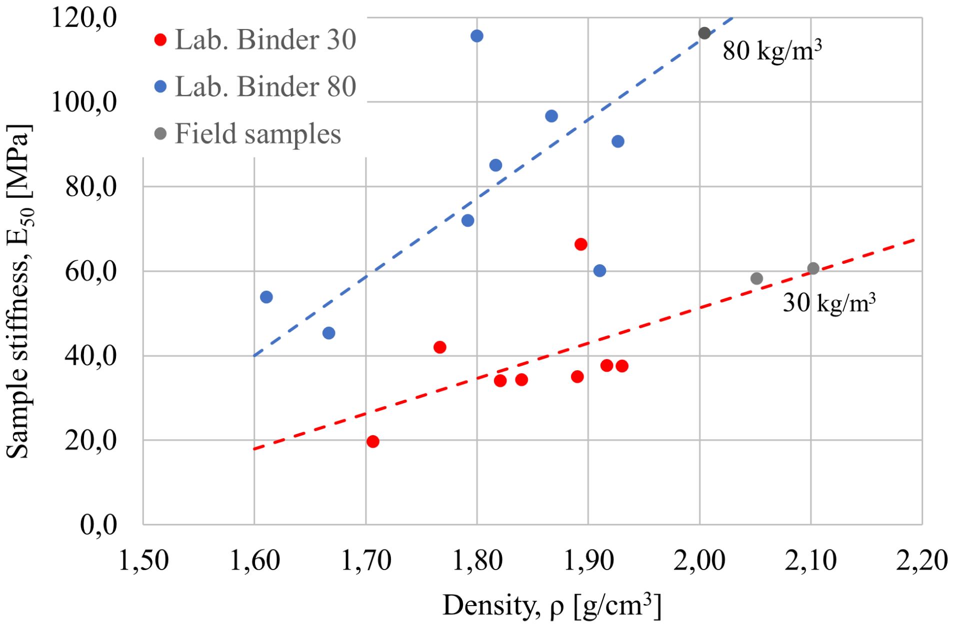


Figure 5.3: Sample stiffness versus density

One of the most obvious differences between field and laboratory samples is the porosity content. Where the measured porosity is estimated to be lower than 2% for all field samples, the same number for the laboratory samples varies from about 13 to 20% for the SVV-method, and 7 to 9% for the NGF-method. As mentioned, the reason for this is most likely related to both how the material is mixed and compacted in field and laboratory, as well as some effect from the larger curing stress in the field. Since NGF-method with lower porosity (and higher density) yields higher sample strength than the SVV-method, it is reasonable to believe that porosity plays a vital role for the material strength. However, the porosity is only part of the strength determination along with the mixing quality and the binder quantity. For instance, laboratory samples with 80 kg/m^3 binder quantity prepared with the NGF-method obtained almost the same unconfined compressive shear strength as the field samples even though the density was only 1.79 g/cm^3 and the porosity was about 7% (relative to field samples with density 2.01 g/cm^3 and porosity 1.7%).

5.3 Field strength estimation and ideal laboratory preparation model

When studying all the "dry" laboratory preparation methods (excluded D1 and D2), figure 5.1 proves that when using the same material and binder quantities the obtained strength seems to follow a linear trend to the density. This indicates that the compacting abilities to the moulding technique will determine the obtained sample strength within the same type of stabilised material (soil type, binder quantities, type and ratio etc.) and same curing conditions. The SVV-methods and NGF-methods in this case will therefore not deviate from following this trend among other preparation method.

Ideally, obtaining the expected field strength from laboratory test mixing trough a fitting model would be a great asset for use in deep mix stabilising design but as already proven this is not easy as field material quality is highly variable and often poor mixed compared to laboratory stabilised material. However, it would be interesting exploring how the field stabilised material would relate to the density-strength relation discovered for the laboratory stabilised material if the mixing process for field was improved.

As for suggestions for improved or new laboratory preparation methods, table 5.2 displays the match between the laboratory samples and the field samples results for shear strength, density and stiffness for all laboratory preparation methods studied in this thesis. The reference value used for field samples, for the different binder quantities of 30, 50, and 80 kg/m^3 respectively, are 220, 300, and 335 kPa for shear strength, 2.08, 2.04, and 2.01 g/cm^3 for density, and 59.4, 116.2, and 80.0 MPa for stiffness.

Table 5.2: Preparation method precision

Method	**Shear strength**			**Density**			**Stiffness**			**Total**
	30	50	80	30	50	80	30	50	80	**Precision**
SVV	50.5 %	48.4 %	59.4 %	82.0 %	80.5 %	80.1 %	33.2 %	42.6 %	46.4 %	58.1 %
NGF	72.5 %	78.5 %	94.9 %	87.5 %	88.5 %	89.1 %	57.3 %	90.0 %	61.9 %	80.0 %
A1	91.6 %		88.0 %	91.0 %		89.6 %	88.3 %		99.5 %	91.3 %
A2	90.0 %		89.4 %	92.8 %		95.0 %	63.3 %		51.7 %	80.4 %
B1	71.1 %		71.4 %	84.9 %		82.9 %	70.8 %		39.0 %	70.0 %
B2	97.9 %		58.7 %	90.9 %		92.9 %	59.0 %		83.2 %	80.4 %
C1	88.2 %		70.0 %	88.5 %		90.4 %	57.9 %		73.2 %	78.0 %
C2	92.6 %		44.2 %	92.1 %		95.9 %	63.4 %		78.0 %	77.7 %
D1	33.9 %		62.5 %	87.3 %		90.4 %	11.3 %		30.6 %	52.7 %
D2	64.0 %		86.3 %	86.9 %		90.4 %	25.6 %		54.2 %	67.9 %

Precision is calculated as the absolute percent accuracy $(1 - |(x_F - x_L)/x_F|)$

By looking at the shear strength fit between laboratory and field samples, increasing the compaction energy (on the standardised SVV and NGF-method) which done for A1 and A2-methods seems to create samples with the best fit. By also considering density and stiffness the A1 method (SVV-method with increased compression energy) clearly is the best choice for producing samples which reflects the field material properties. This is interesting, as the SVV-method, in which the A1-method is based on, has one of the poorest match with the field properties. This indicates that the static compression moulding SVV-method has a high potential if the compression pressure is increased. The A2-method (NGF-method with increased impact energy) also shows great potential in representing both shear strength and density, but fails to match the stiffness properties of the field material. Still, both A1 and A2 seems to be better choices than the recommended methods used in the Norwegian geotechnical society today.

It is important to remember that these results are only valid for the specific clay used for this project, and further investigation on different clay must be conducted. It is also important remembering that the mixing efforts in the field can vary, and therefore potentially could yield completely different field properties than what is achieved in this project. This limits the findings done on the improvement of the laboratory sample preparation method. Still, it can be stated that the existing standardised methods (NGF-method and SVV-method) does not produce laboratory prepared samples in which properties can be directly used as design values to describe the field stabilised material properties. Any use of laboratory test mix with these method should persistently only be used to predict minimum values for field stabilised soil strength.

5.4 Limitations of the findings

As briefly stated earlier, a great limitation to any findings in the thesis relates to the amount of data available. All though it applies to both laboratory and field data, the material from field is more greatly influenced by large variations. Basically, to improve the accuracy for the measurements, a higher number of stabilised columns should be examined both with in-situ surveys and sampling for laboratory tests. This argument is both based on a relative high coefficient of variation in laboratory test which implies huge variation within samples from the same pile, but also high variation in the CPTU readings which indicates great variation between different piles. This is obviously proven by the unexpected low unconfined compression strength in the type III-b pile as discussed in section 4.2.3.

The same goes for the laboratory produced samples, where a higher number of samples per variation (binder quantity and moulding method) would decrease the variation. This goes especially for the trial methods in which only three samples were made per variation. Also, for the trial methods, making samples for binder quantity of 50 kg/m^3 would give valuable information on the properties trends relative to binder amount (in addition to the 30, and 80 kg/m^3 which were already produced). With that said, the laboratory samples seemed to yield samples with significantly lower deviation than the field samples which had a larger sample selection.

As already mentioned, all research is made on material from the same project and same soil. This means that the findings in this thesis will be bound to this project or projects with similar soil properties. There is also a certain limitation of the validity of the findings towards other kind of binder types and ratios.

As of the field stabilisation works, the mixing equipment settings made little room for change in certain settings as binder flow and retrieval rate in which sufficient mixing for lower binder quantities was hard to achieve. This is thought to be the main reason for poor mixing especially for the low binder quantity samples.

5.5 Further work

Based on the obtained results and the material variation obtained for the field material in this thesis, further investigation on the mixing quality should be explored for other material. A quantification an analysis on how the mixing efforts affects the field material variation and repetitiveness of both material from same lime-cement column and material from different piles would be very useful information, especially when trying to predict field strength by laboratory trial mixing.

By changing the methods for laboratory sample preparation, the potential for obtaining similar results with field samples may increase, especially with method based on the SVV and NGF-method with increased compaction energy (A1 and A2 methods). However, this finding is only valid for field material used in this project. Further research should be tested for other deep stabilisation projects, with different soils while also comparing to the mixing energy applied when stabilising the soil.

6 | Conclusion

From the findings in this thesis the following conclusion could be made:

1. The field stabilised material showed large variation in properties (especially shear strength and stiffness) both from samples within the same pile column, as well as between piles.
2. Binder accumulations in the field stabilised samples were highly visible both with visual observation and from CT-images. Binder accumulations in stabilised material from piles with lower binder quantities (30 kg/m^3) and lower blade rotation number (T-rate) seemed more prominent, all though binder accumulation also were somewhat present in some of the samples with higher binder quantities and higher blade rotation number.
3. For the same type of stabilised material (soil properties, binder types and ratio, binder quantity etc.) and the same curing conditions (temperature, time etc.), the laboratory samples strength seems to be linearly increasing with the sample densities, independent on which moulding method is used. The field sample strength does not fit the trend, which likely is due to poor mixing. Binder accumulations are probably a factor for lower strength. Laboratory samples stiffness seems to be linearly increasing with the density, and the field samples seems to fit this trend.
4. Comparison of the static compaction SVV-method and the dynamic compaction NGF-method, both used in laboratory sample production in Norway today, shows quite significant differences in sample properties. The SVV-method produces samples with significantly lower shear strength, lower stiffness and higher porosity (lower density) compared to the NGF-method.
5. Of the standardised laboratory sample preparation methods used in Norway today, the NGF-method is better suited to describe the field strength, stiffness, and density for field stabilised clay studied in this thesis. However, the existing standardised methods still does not produce laboratory samples which properties describe the field material properties sufficiently to be directly used as design values.
6. Increasing the compaction energy seems to be most effective measure to better describe the field strength. The SVV-method and NGF-method with doubled compaction energy had significantly better accuracy predicting field strength. This observation is yet to be confirmed to be valid for other types of clay in other projects with other initial settings. However, the methods (A1 and A2) seems overall more precise.

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A | Laboratory data

A.1 Unconfined compression results

In this section data from the unconfined compression test is displayed for all samples. Please note that the raw data is corrected for a hypothetical zero strain at full sample contact at test start. This means that any readings, where strain is unrealistically large due to insufficient contact between the sample area and the piston, is not displayed.

The samples scanned with CT are presented in the graphs, but do not contribute to average values as they were influenced by drying during the scanning process, which made the strength typically higher.

A.1.1 Field samples

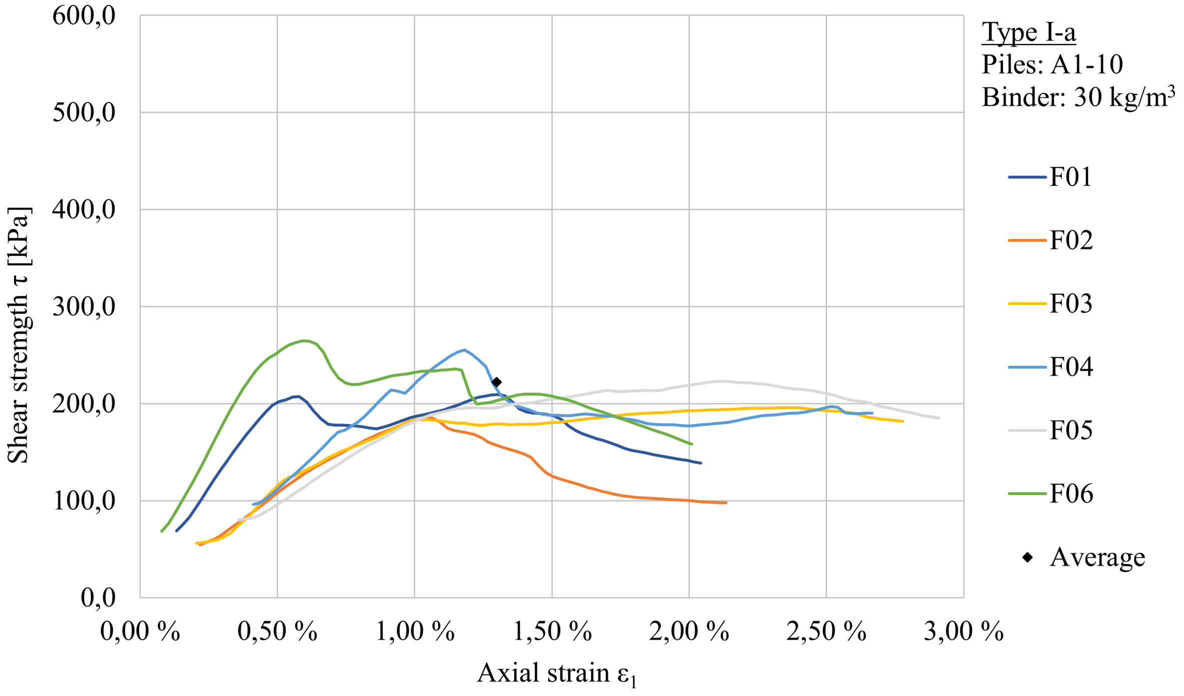


Figure A.1: Field Type I-a - Unconfined compression shear strength

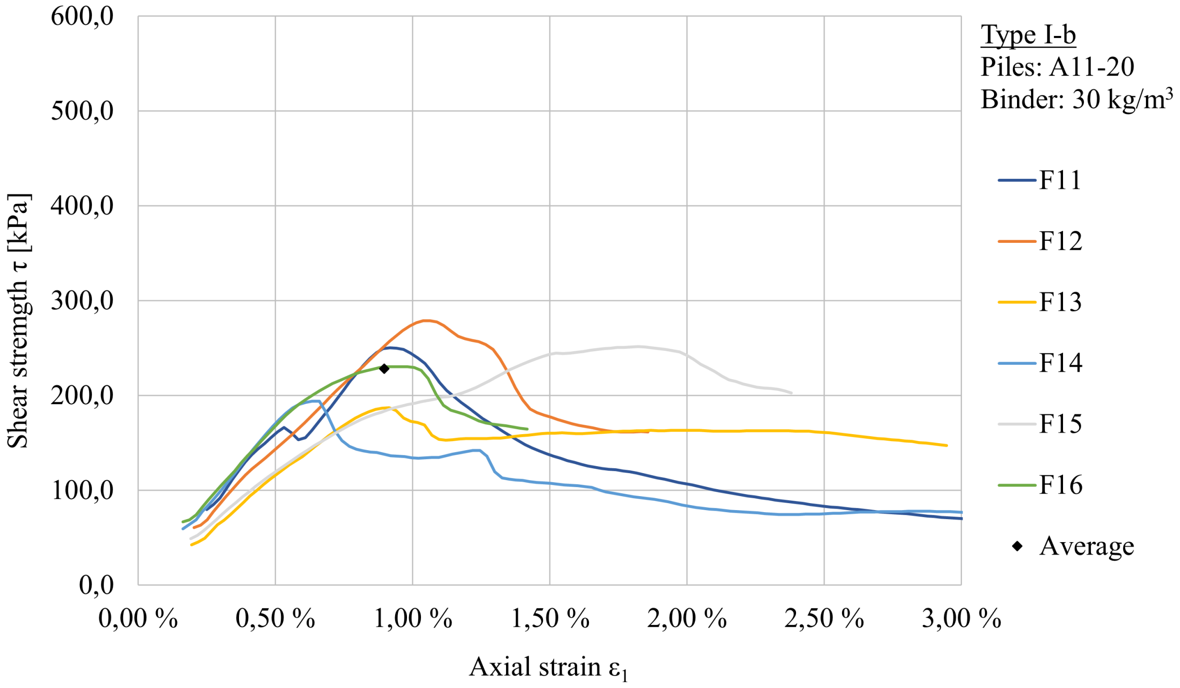


Figure A.2: Field Type I-b - Unconfined compression shear strength

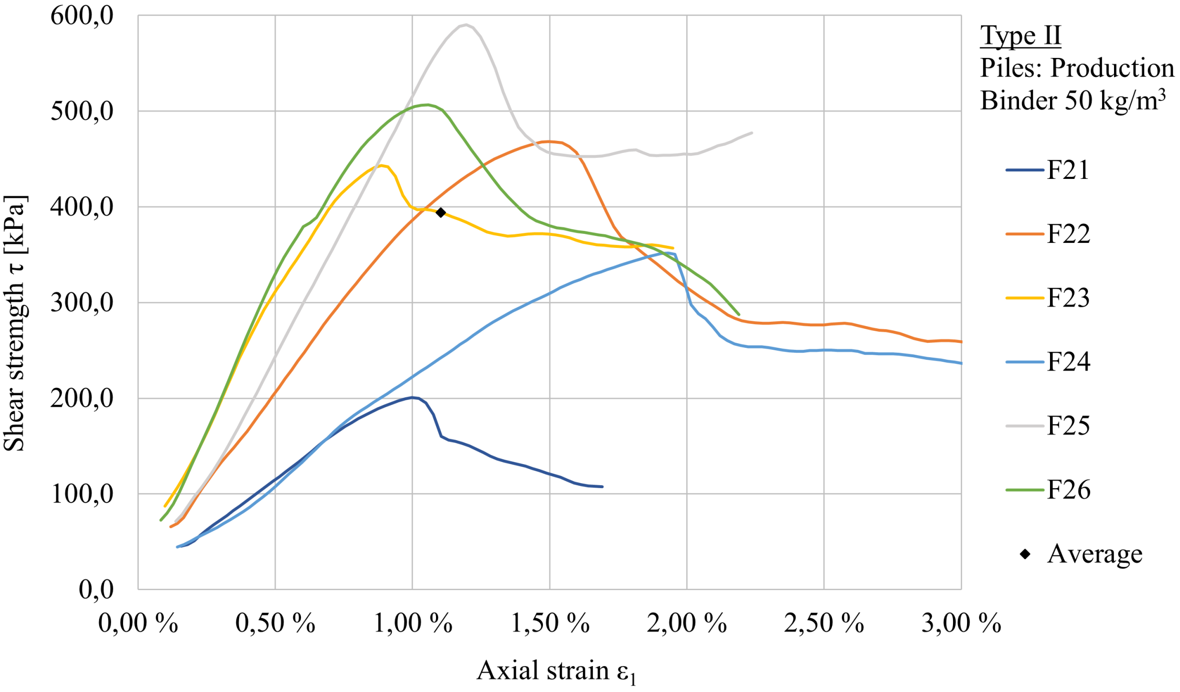


Figure A.3: Field Type II - Unconfined compression shear strength

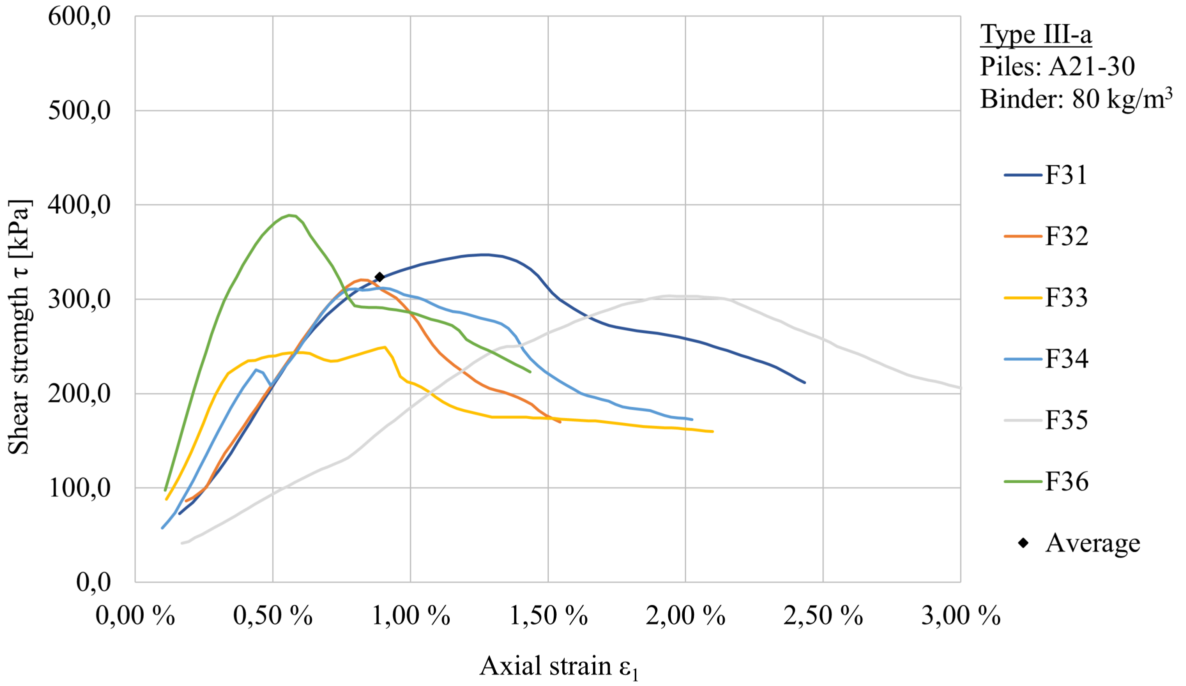


Figure A.4: Field Type III-a - Unconfined compression shear strength

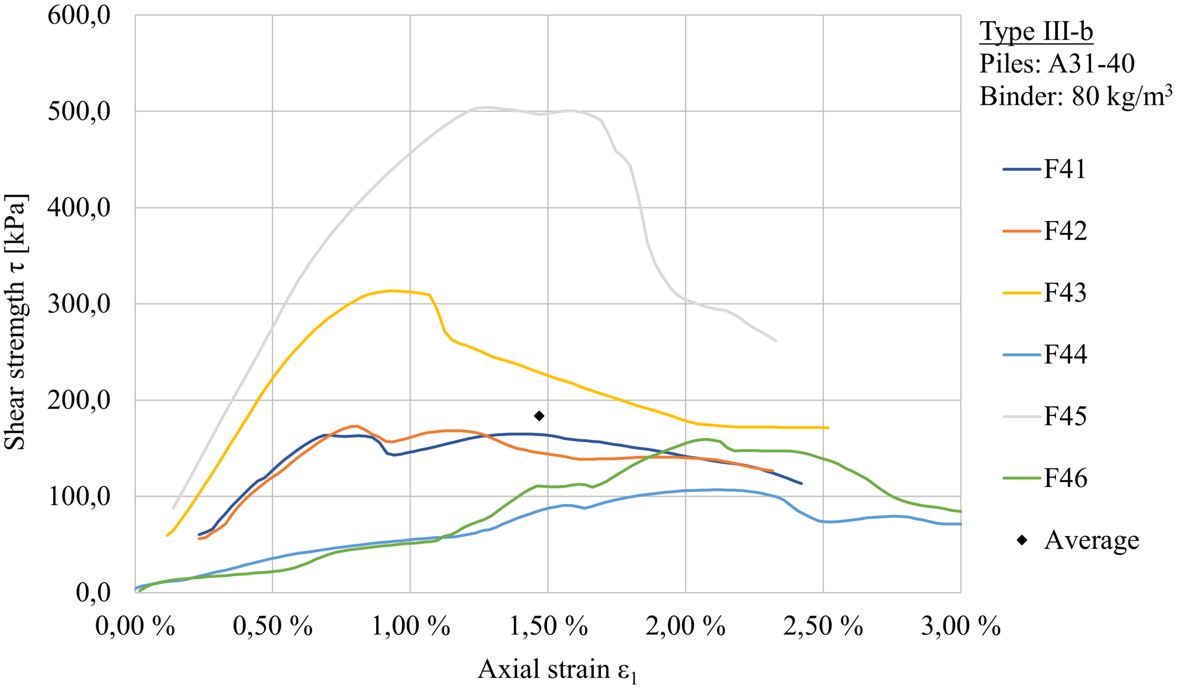


Figure A.5: Field Type III-b - Unconfined compression shear strength

A.1.2 Laboratory samples

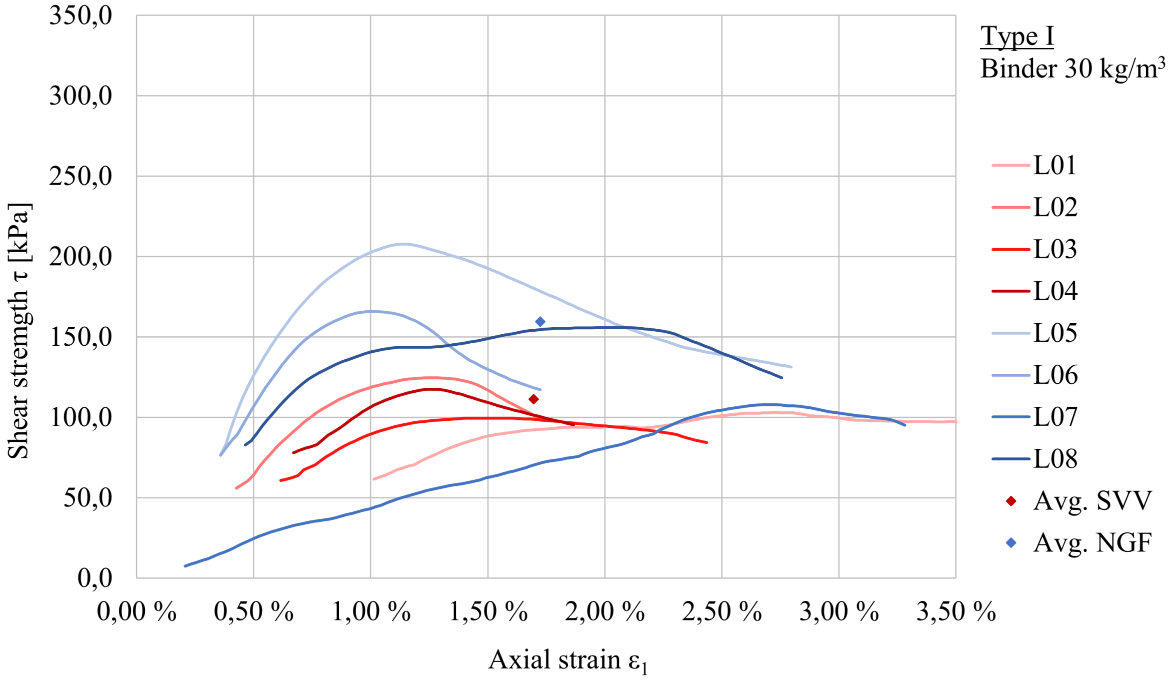


Figure A.6: Laboratory Type I - Unconfined compression shear strength

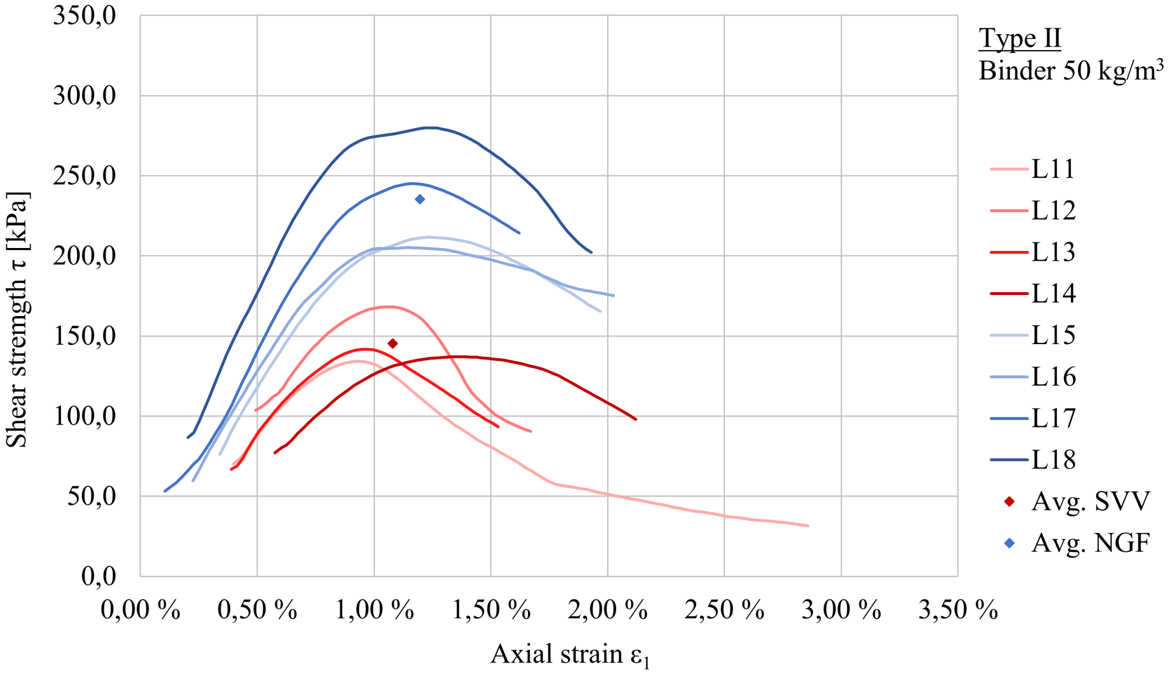


Figure A.7: Laboratory Type II - Unconfined compression shear strength

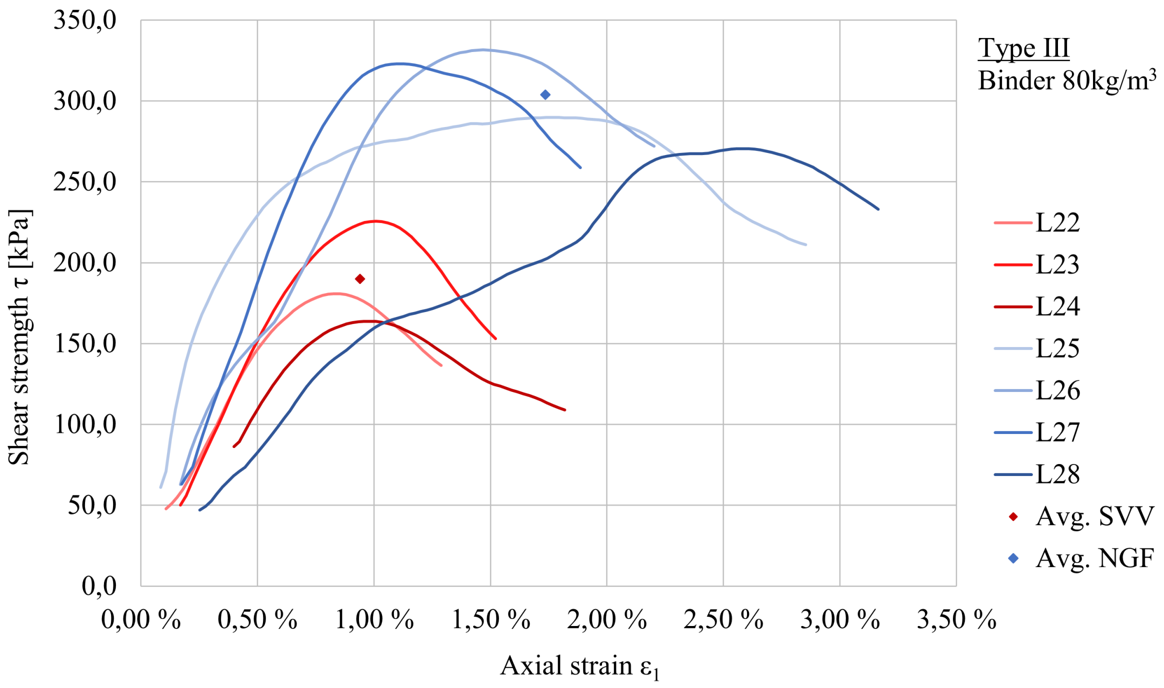


Figure A.8: Laboratory Type III - Unconfined compression shear strength

A.1.3 Trial laboratory samples

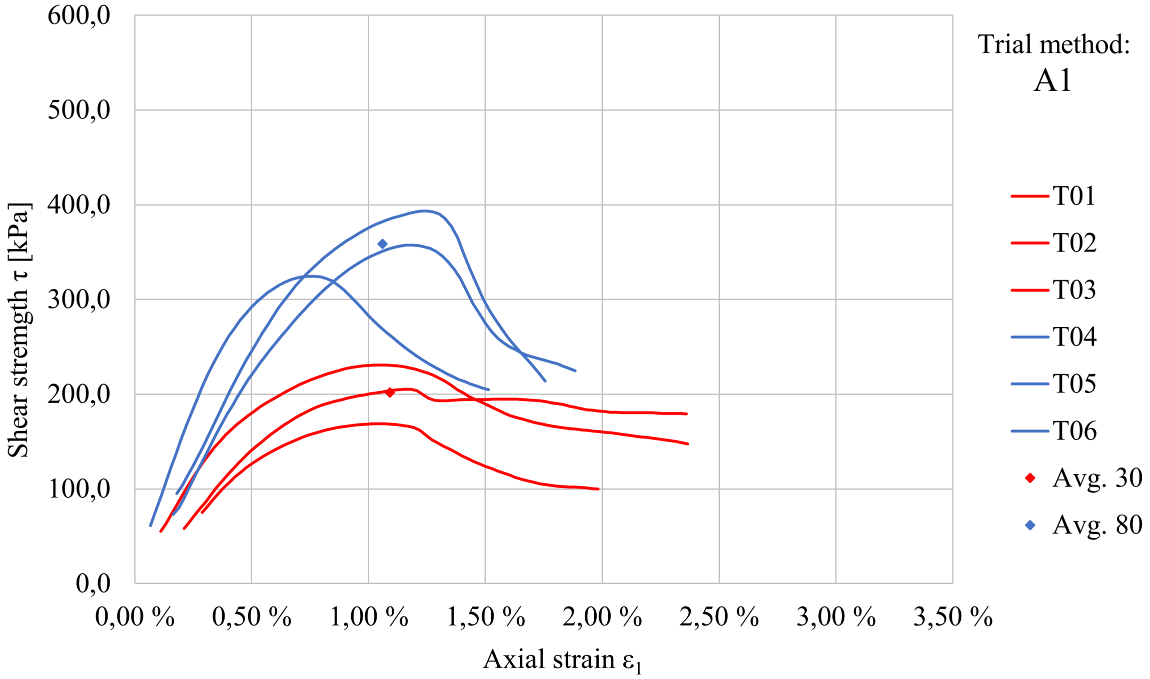


Figure A.9: Trial method A1 - Unconfined compression shear strength.

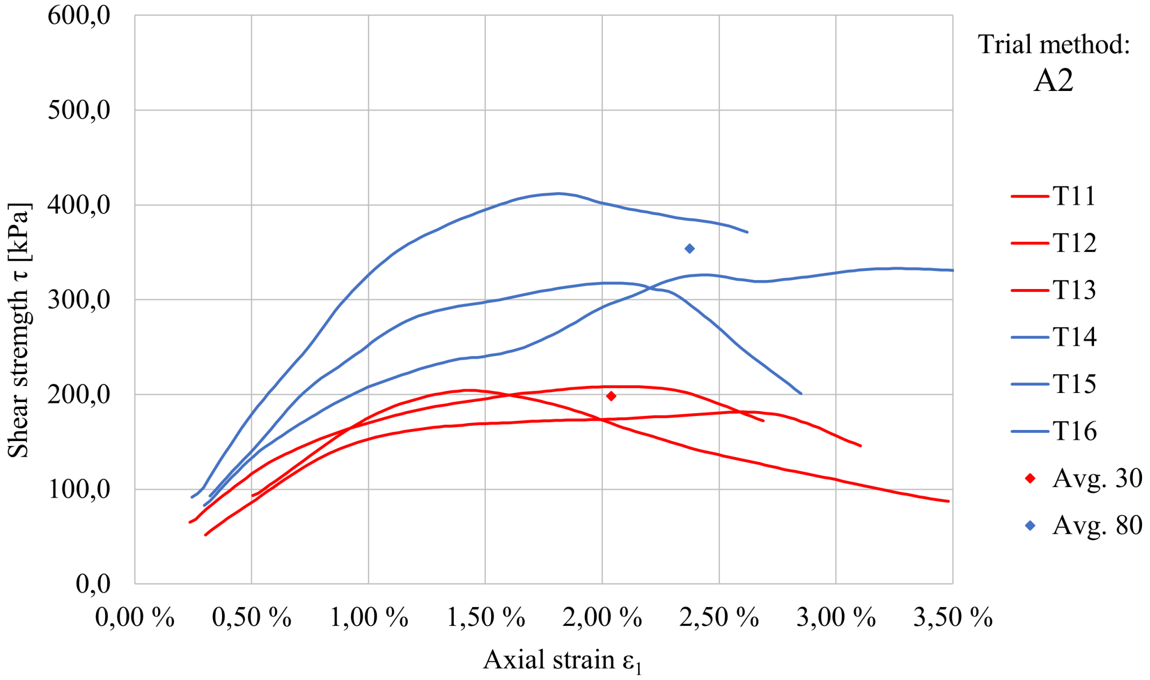


Figure A.10: Trial method A2 - Unconfined compression shear strength.

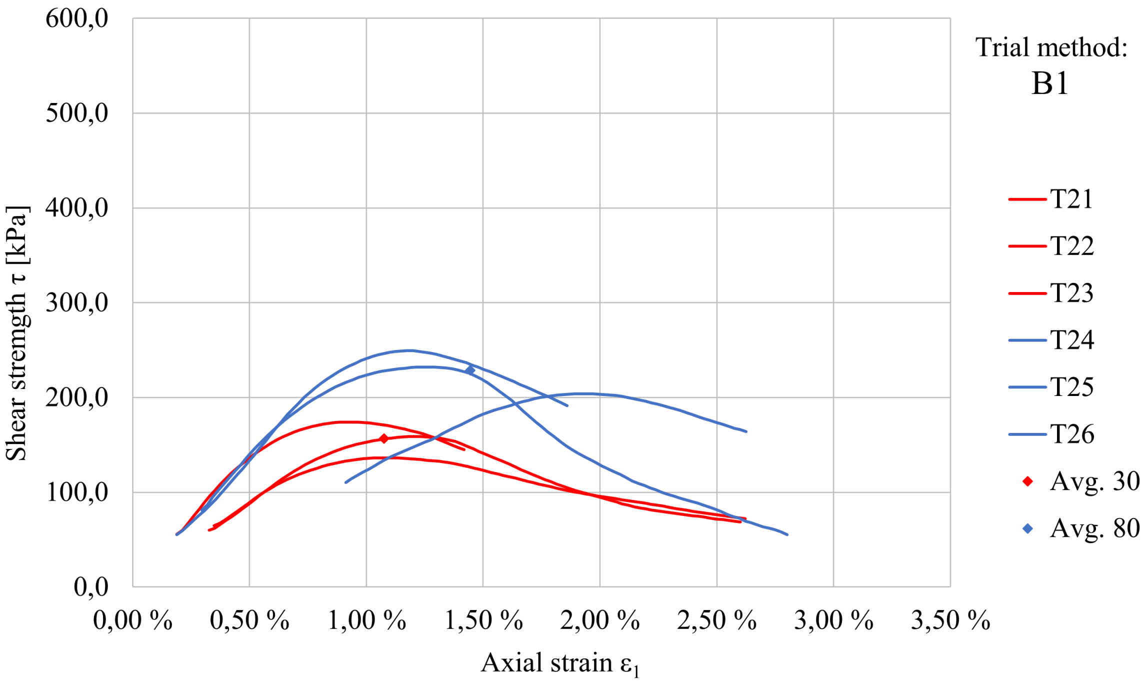


Figure A.11: Trial method B1 - Unconfined compression shear strength.

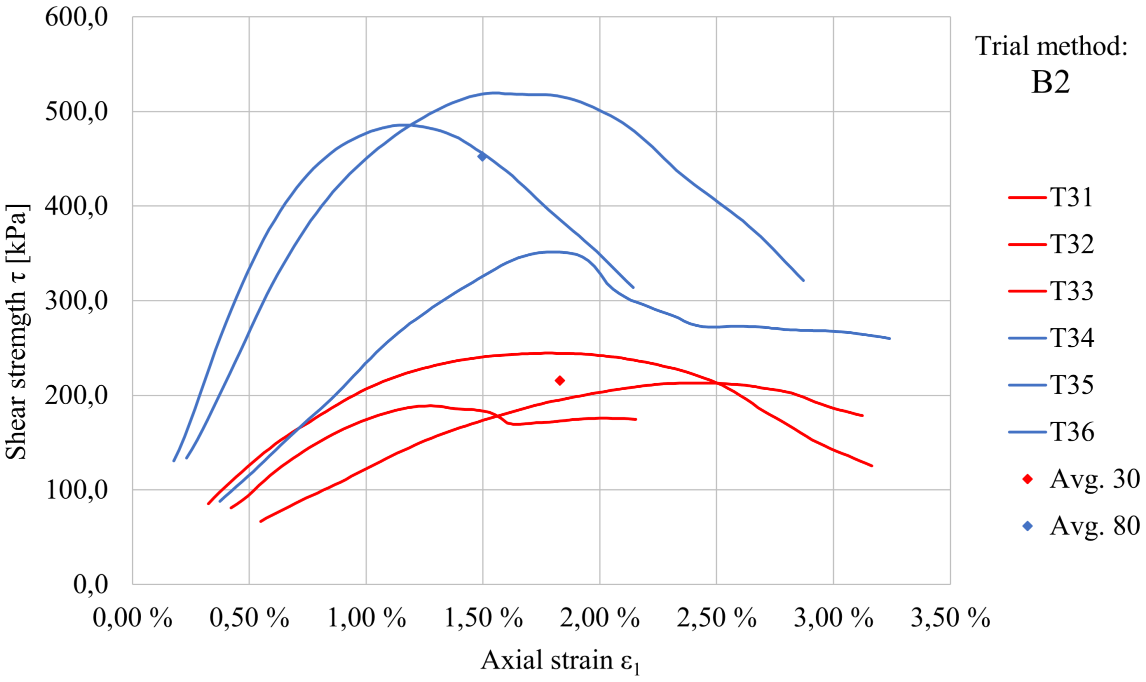


Figure A.12: Trial method B2 - Unconfined compression shear strength.

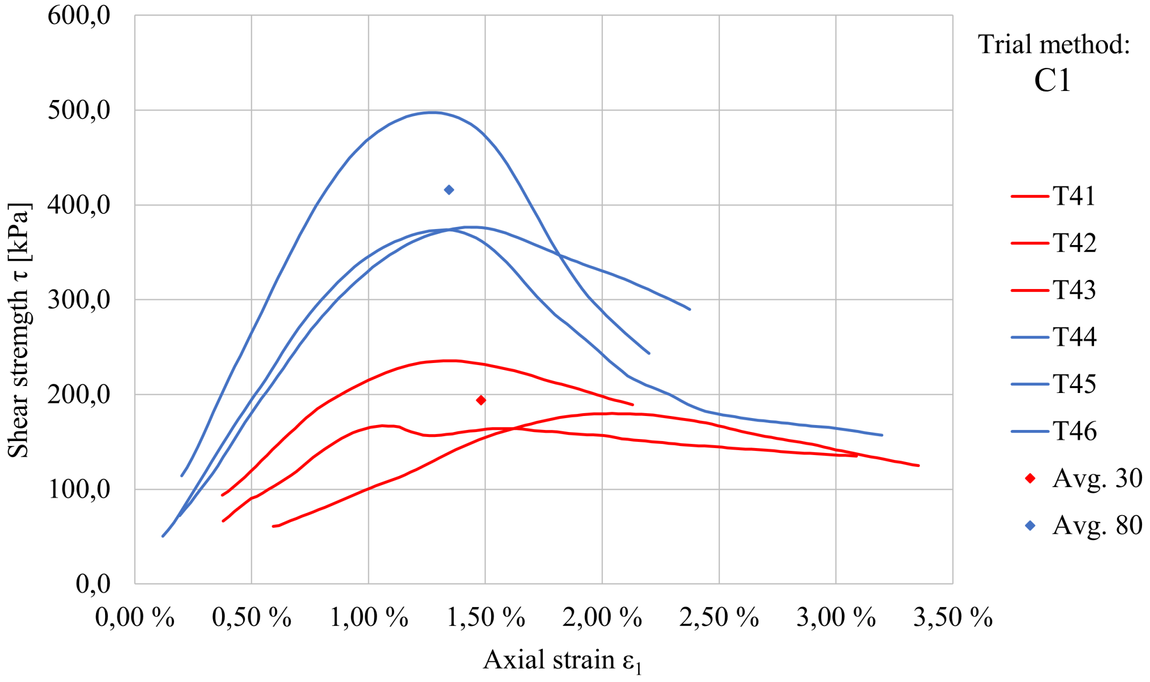


Figure A.13: Trial method C1 - Unconfined compression shear strength.

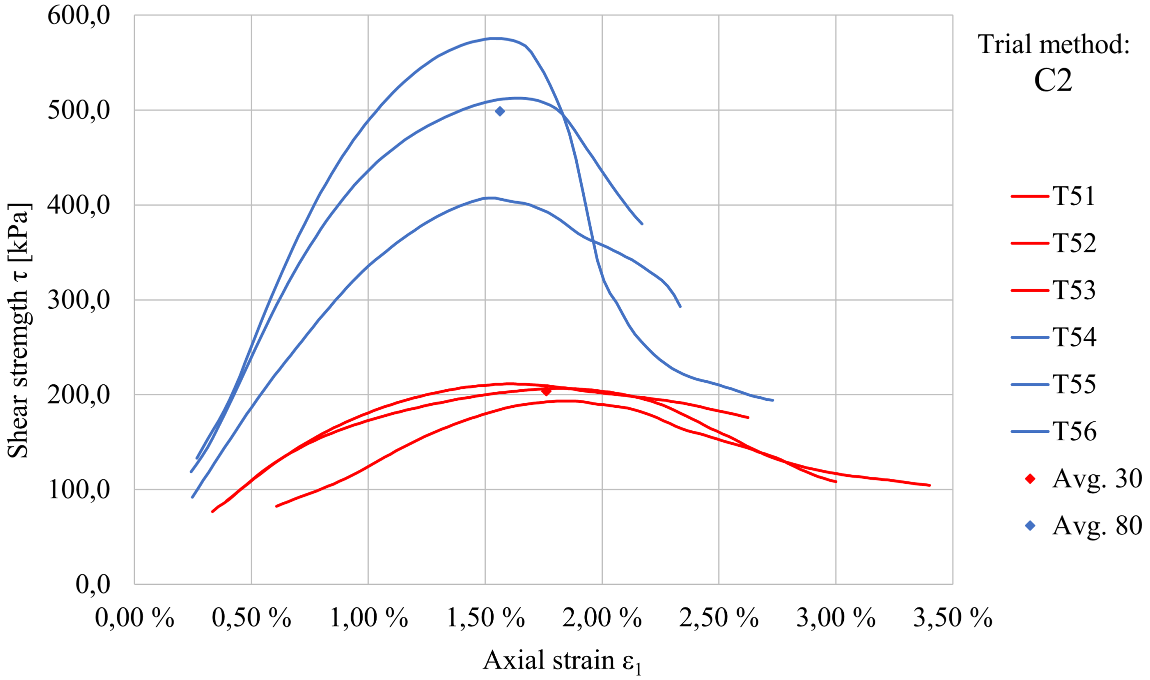


Figure A.14: Trial method C2 - Unconfined compression shear strength.

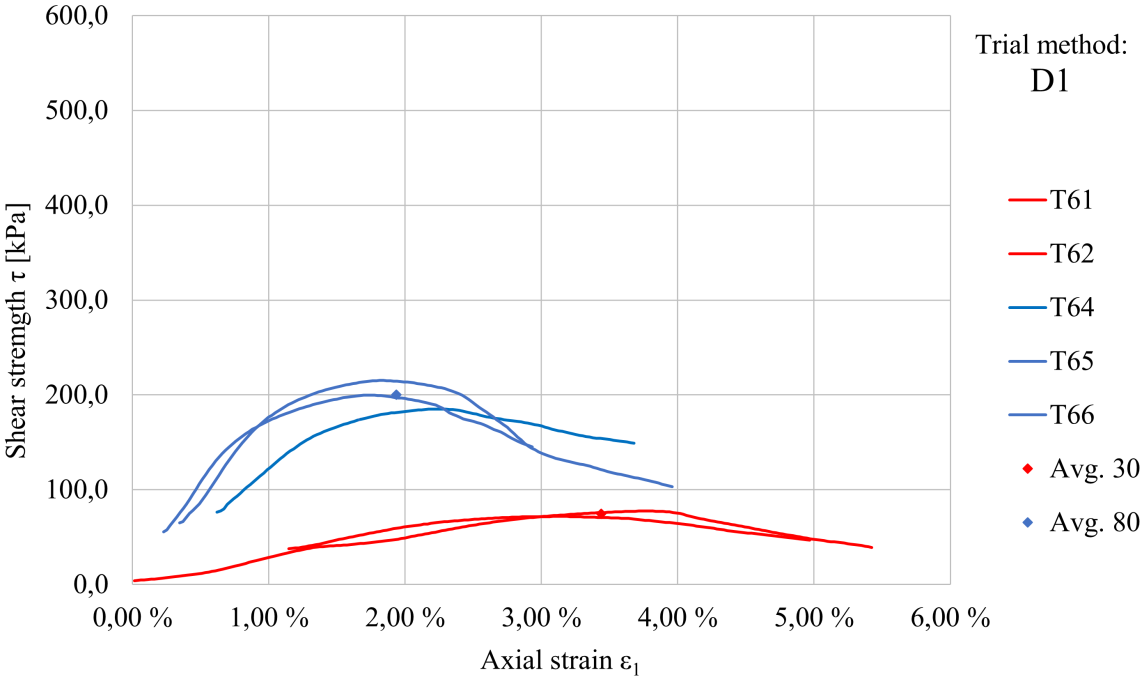


Figure A.15: Trial method D1 - Unconfined compression shear strength.

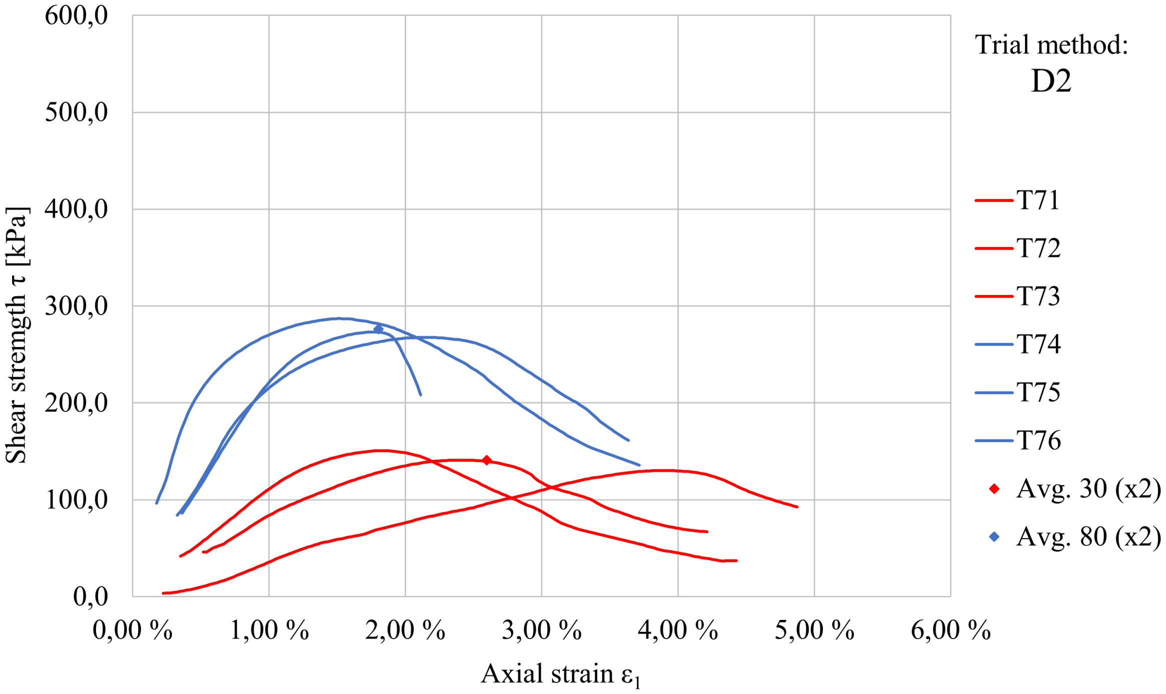
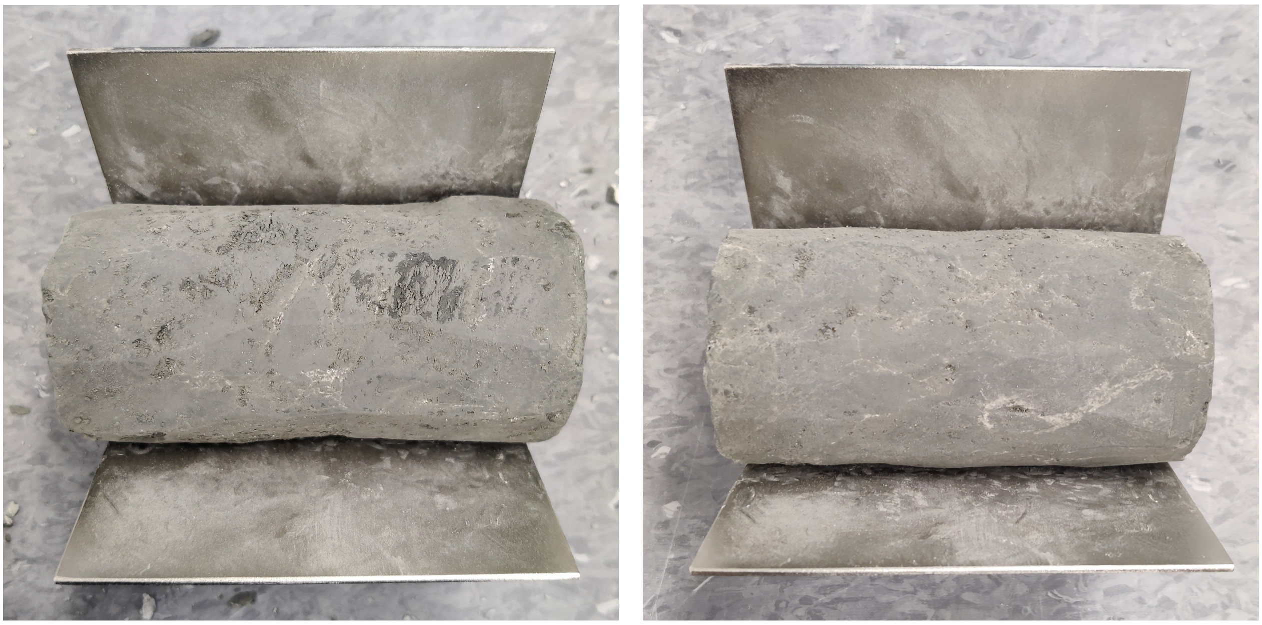


Figure A.16: Trial method D2 - Unconfined compression shear strength.

A.2 Sample pictures

A.2.1 Pictures - Field samples



(a) F02 I-a

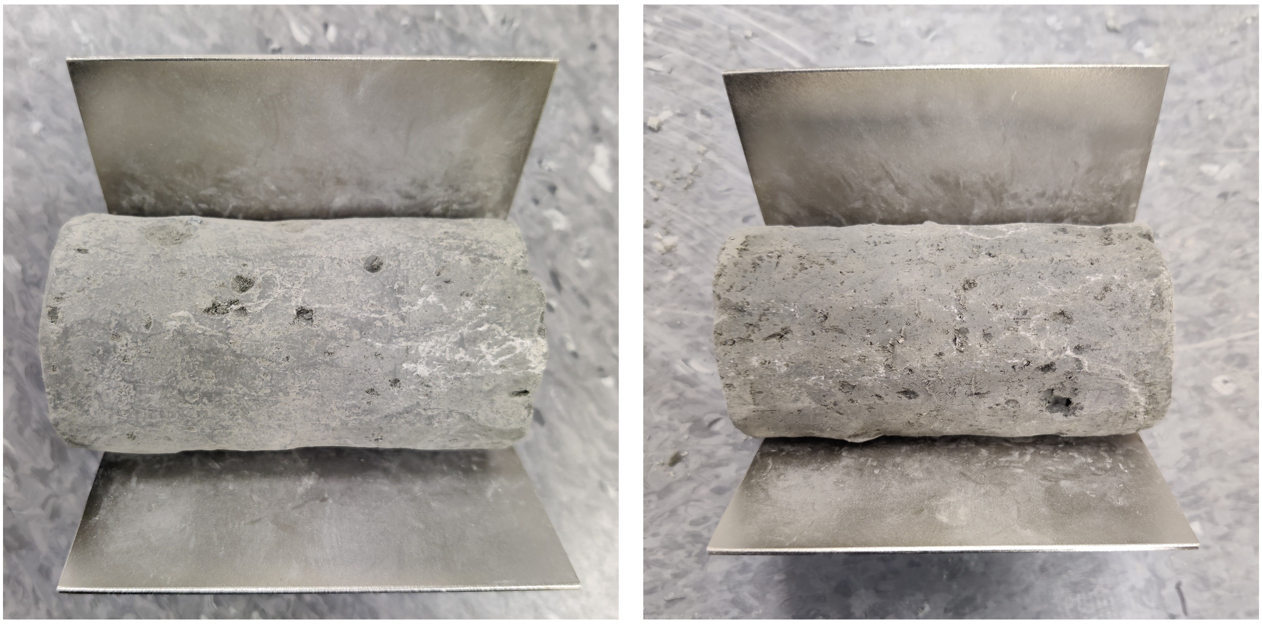
(b) F04 I-a

Figure A.17: Pictures - Field type I-a (30 kg/m^3, location a)

(a) F12 I-b

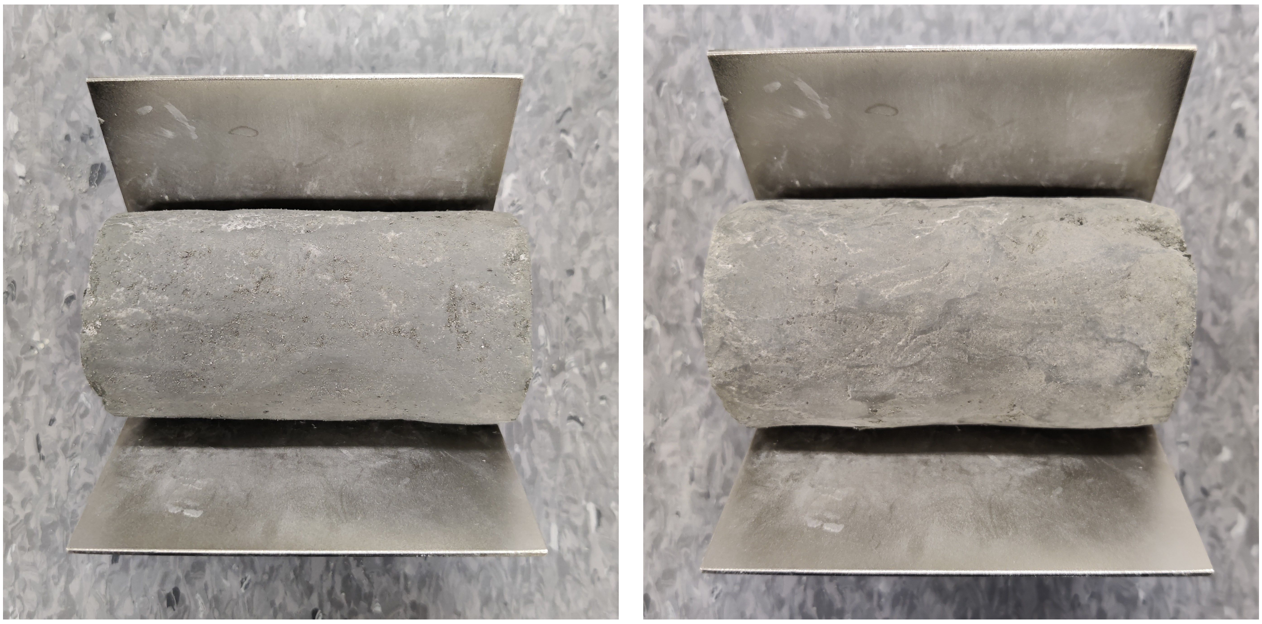
(b) F15 I-b

Figure A.18: Pictures - Field type I-b (30 kg/m^3, location b)



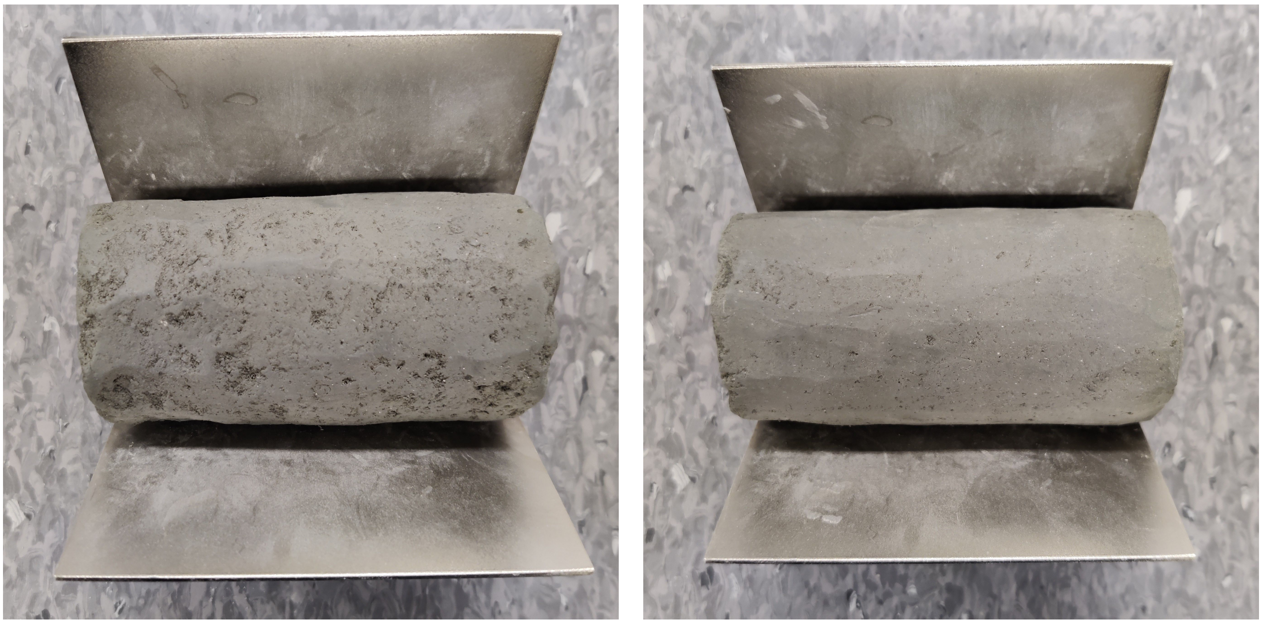
(a) F24 II (b) F26 II

Figure A.19: Pictures - Field type II (50 kg/m^3)



(a) F34 III-a (b) F36 III-a

Figure A.20: Pictures - Field type III-a (80 kg/m^3, location a)

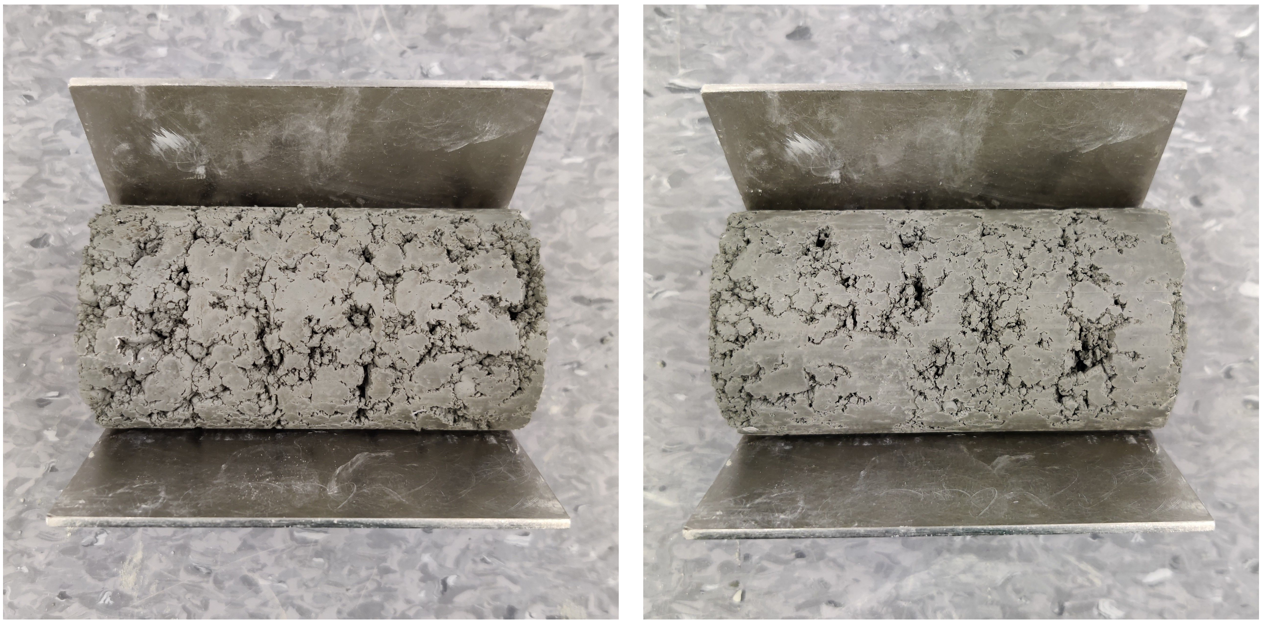


(a) F44 III-b

(b) F46 III-b

Figure A.21: Pictures - Field type III-b (80 kg/m^3, location b)

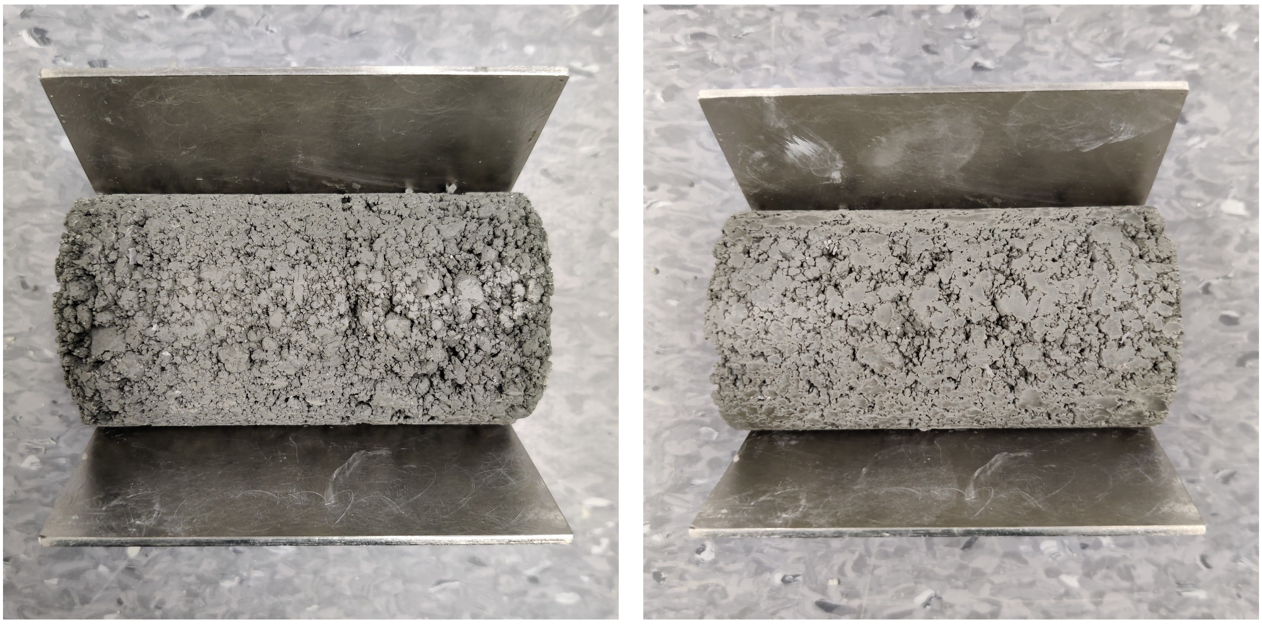
A.2.2 Pictures - Laboratory samples



(a) L04 I (30 kg/m^3 SVV)

(b) L06 I (30 kg/m^3 - NGF)

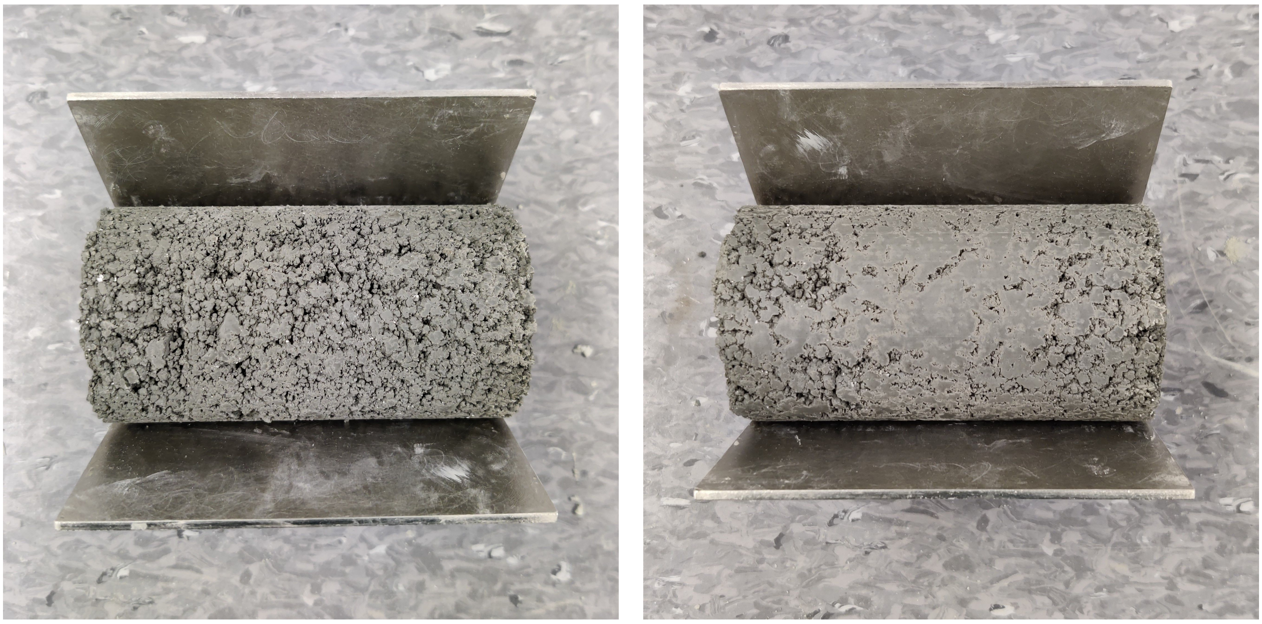
Figure A.22: Pictures - Laboratory samples - Binder 30 kg/m^3 (SVV- and NGF-method)



(a) L11 II (50 kg/m^3 - SVV)

(b) L16 II (50 kg/m^3 - NGF)

Figure A.23: Pictures - Laboratory samples - Binder 50 kg/m^3 (SVV- and NGF-method)



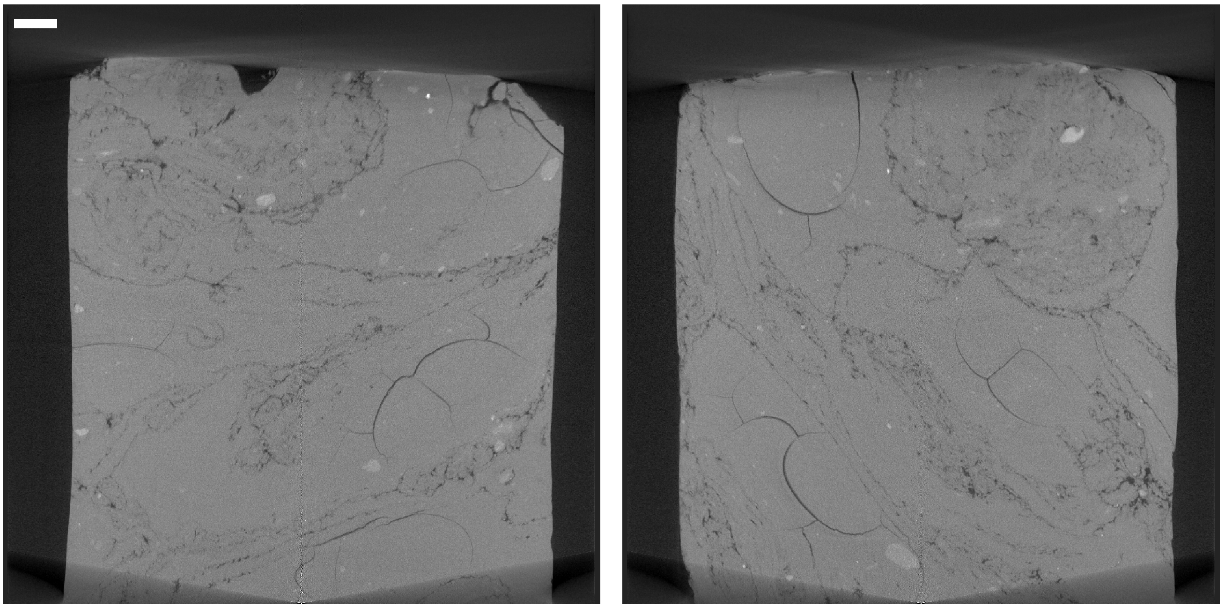
(a) L24 III (80 kg/m^3 - SVV)

(b) L27 III (80 kg/m^3 - NGF)

Figure A.24: Pictures - Laboratory samples - Binder 80 kg/m^3 (SVV- and NGF-method)

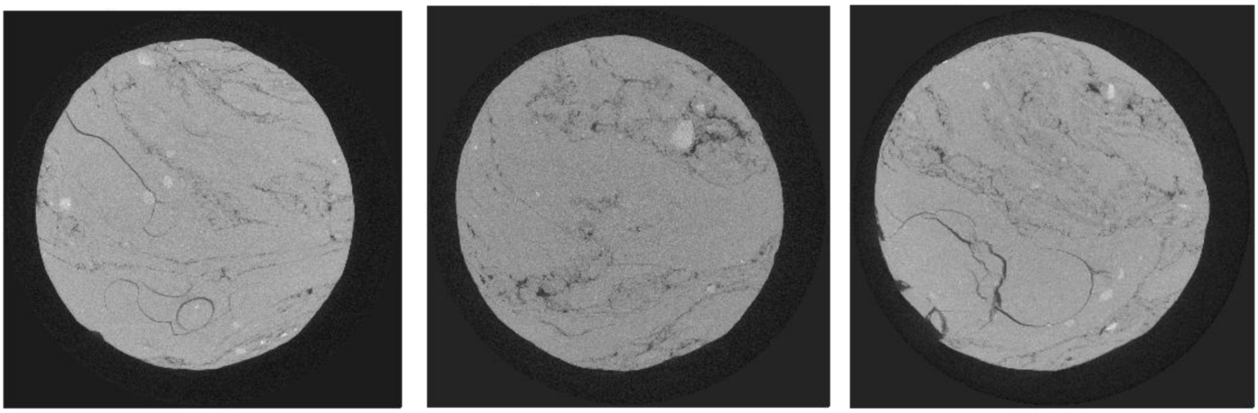
A.3 CT-images

A.3.1 CT-images - Field samples



xz-plane centre

yz-plane centre

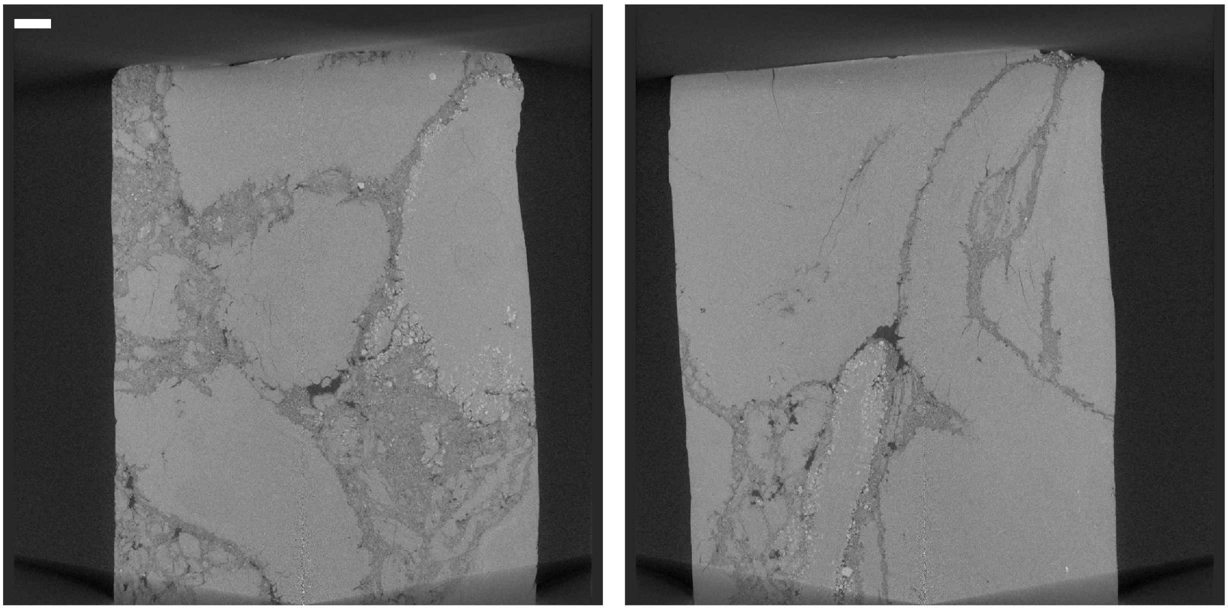


xy-plane (depth=1.2cm)

xy-plane (depth=3.5cm)

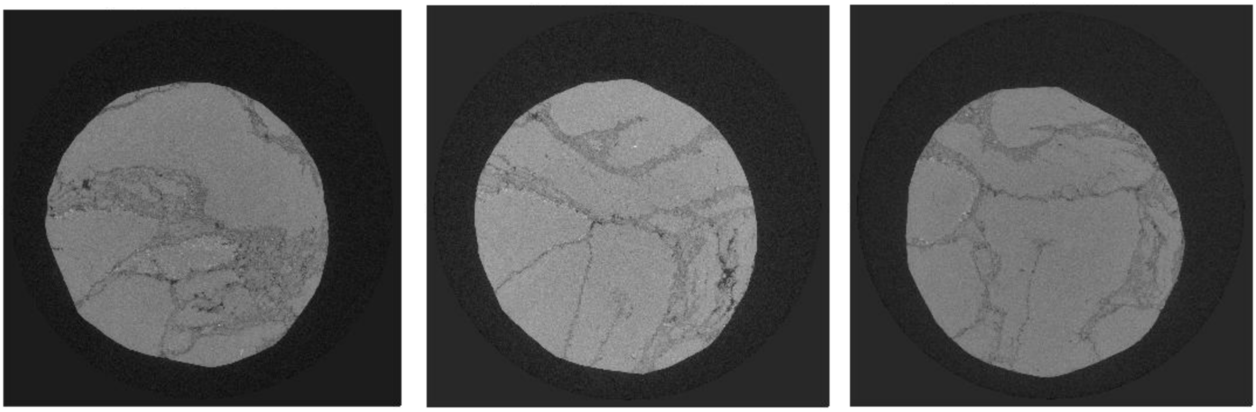
xy-plane (depth=5.2cm)

Figure A.25: CT-images field type I-a (30 kg/m^3 binder, sample F05 I-a)



xz-plane centre

yz-plane centre

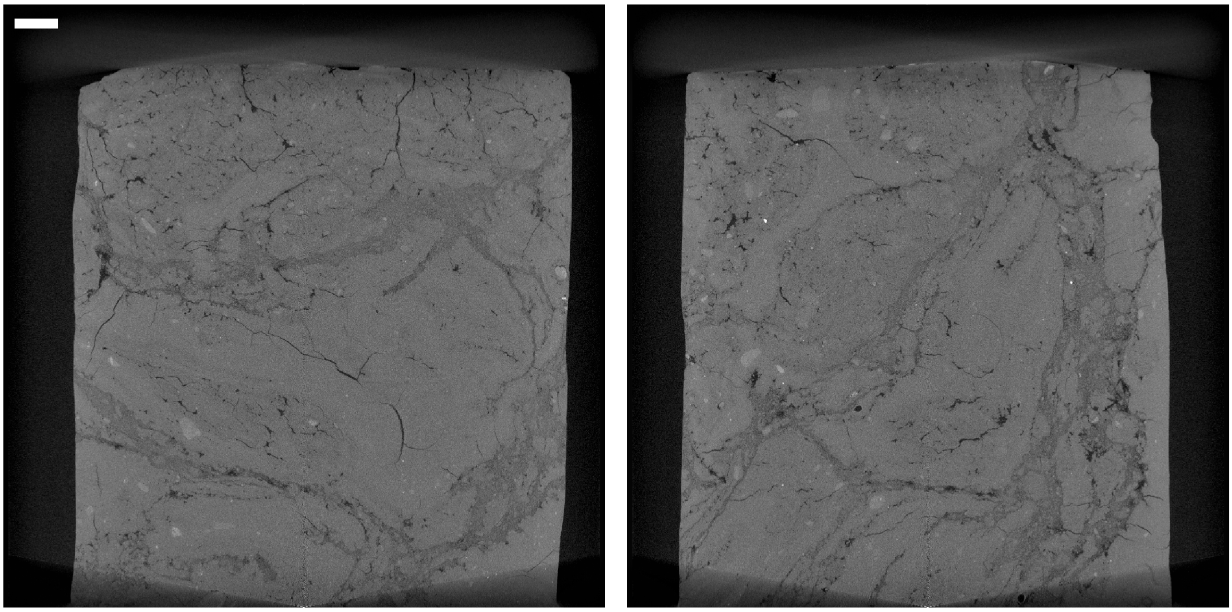


xy-plane (depth=1.2cm)

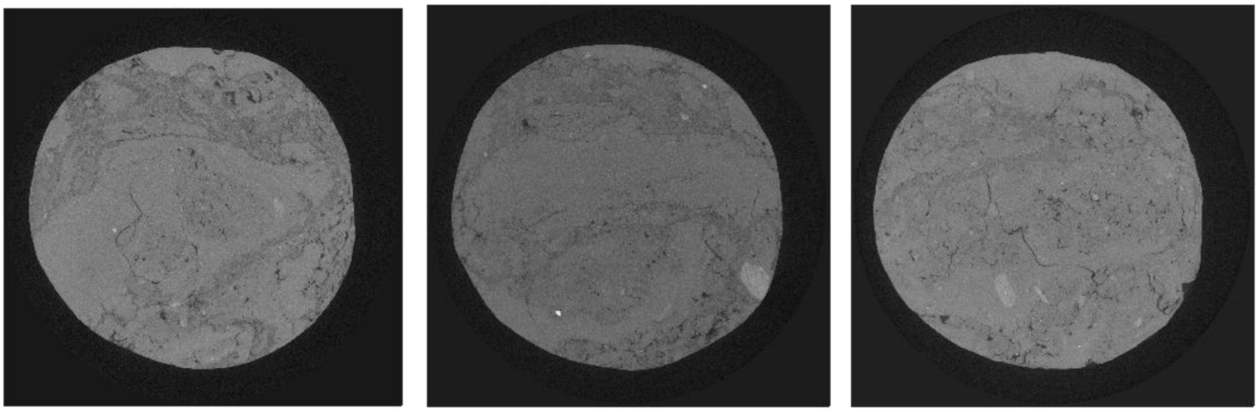
xy-plane (depth=3.5cm)

xy-plane (depth=5.2cm)

Figure A.26: CT-images field type I-b (30 kg/m^3 binder, sample F15 I-b)

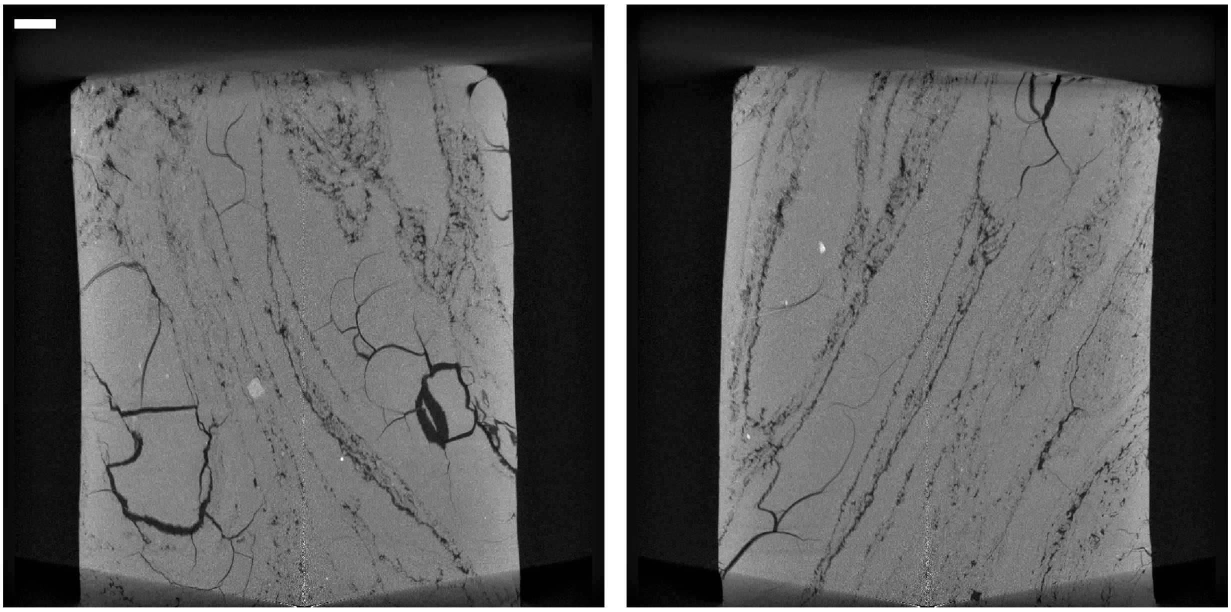


xz-plane centre yz-plane centre



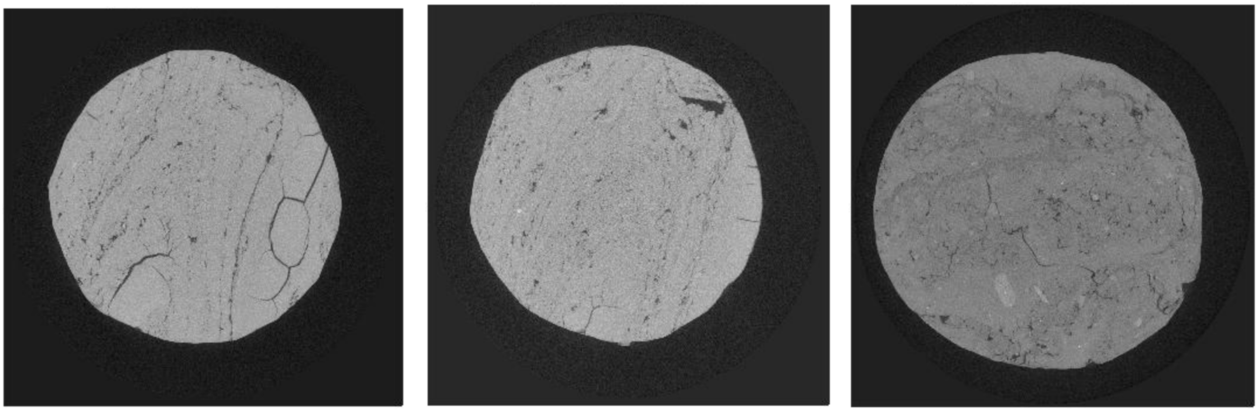
xy-plane (depth=1.2cm) xy-plane (depth=3.5cm) xy-plane (depth=5.2cm)

Figure A.27: CT-images field type II (50 kg/m^3 binder, sample F25 II)



xz-plane centre

yz-plane centre

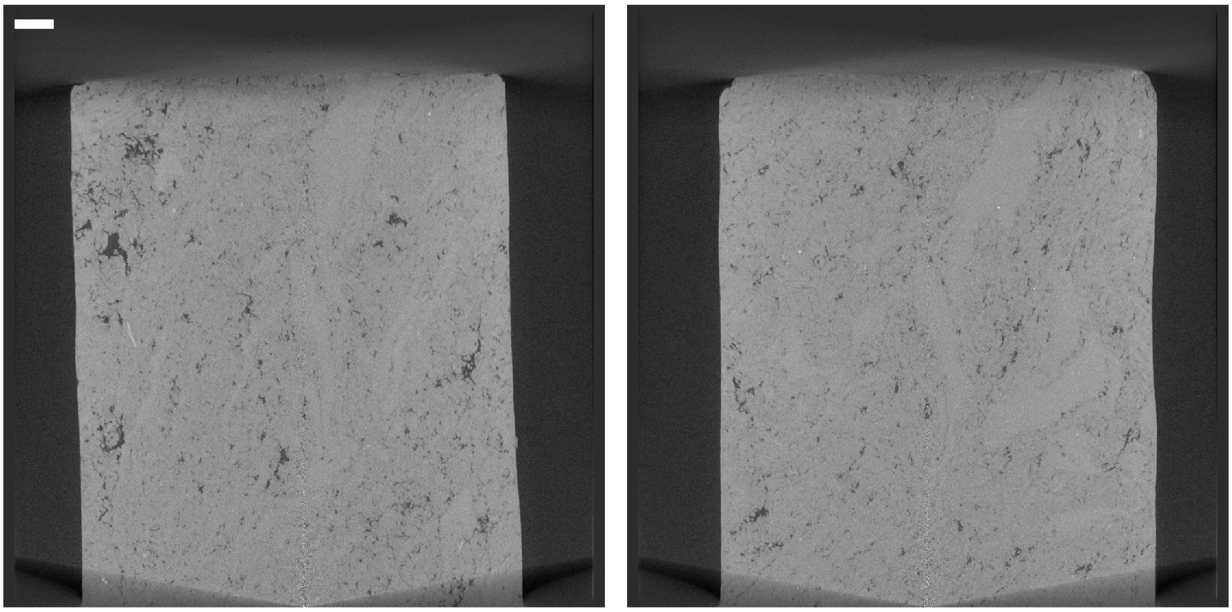


xy-plane (depth=1.2cm)

xy-plane (depth=3.5cm)

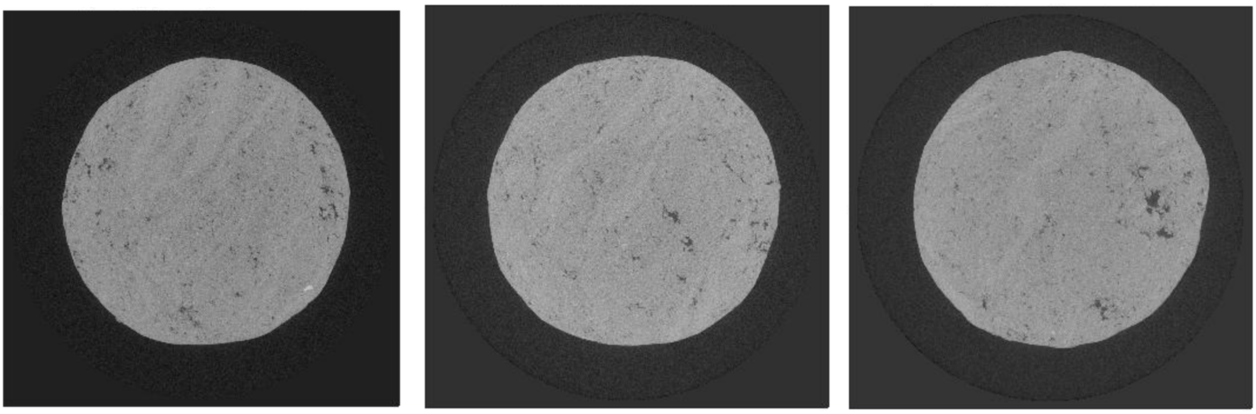
xy-plane (depth=5.2cm)

Figure A.28: CT-images field type III-a (80 kg/m^3 binder, sample F35 III-a)



xz-plane centre

yz-plane centre



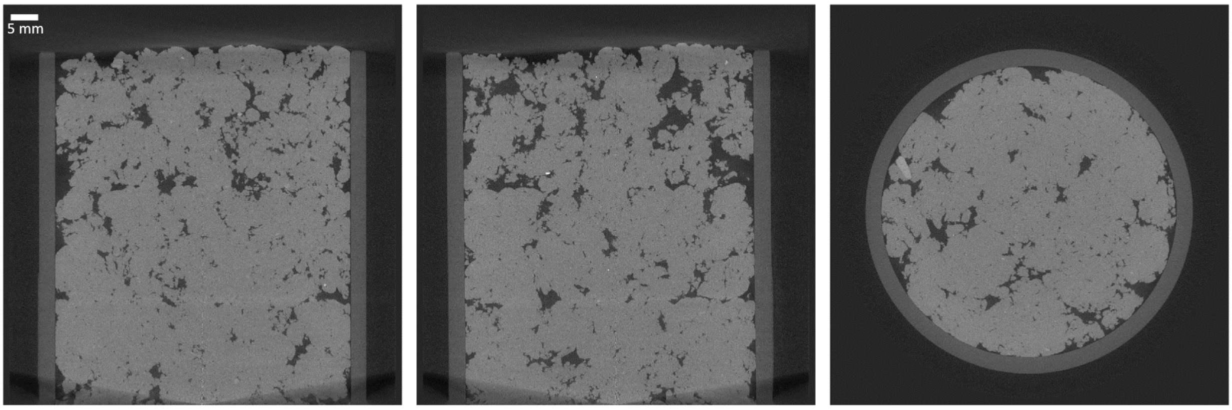
xy-plane (depth=1.2cm)

xy-plane (depth=3.5cm)

xy-plane (depth=5.2cm)

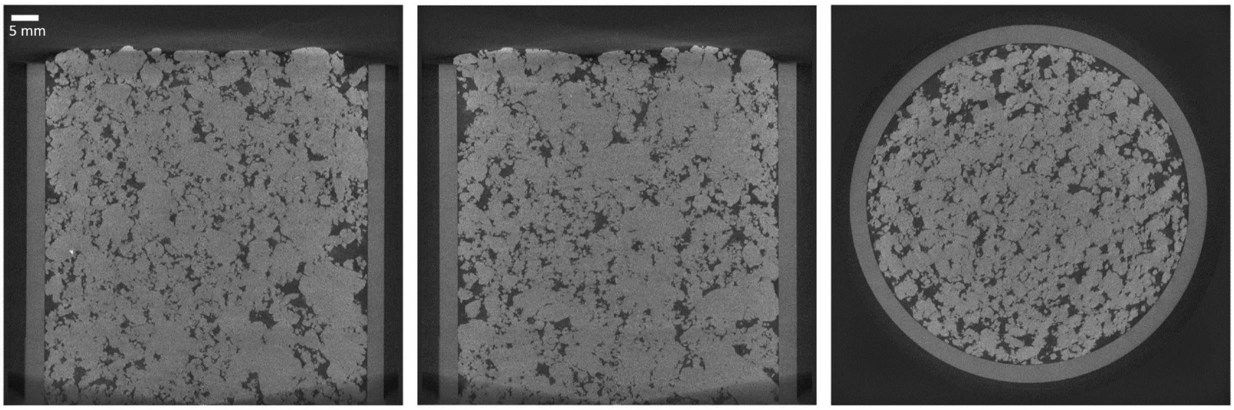
Figure A.29: CT-images field type III-b (80 kg/m^3 binder, sample F45 III-b)

A.3.2 CT-images - Laboratory samples



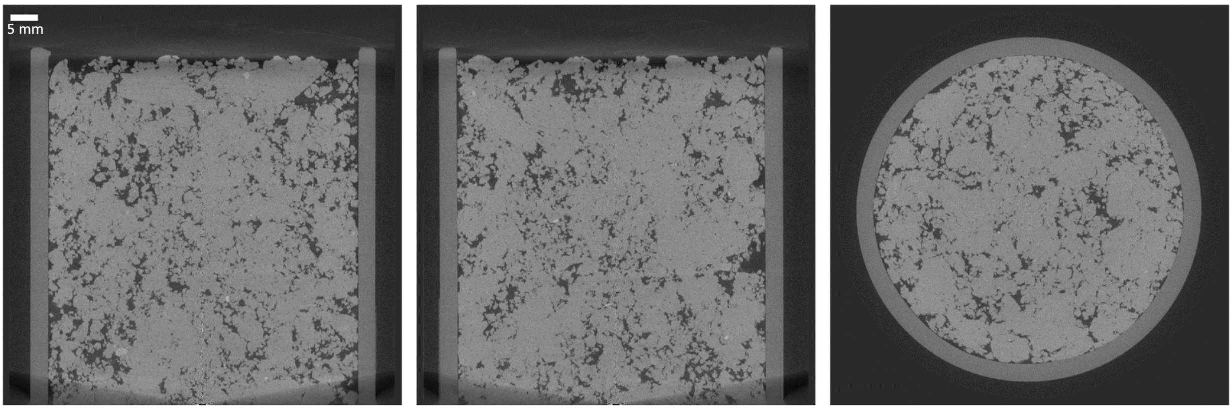
xz-plane yz-plane xy-plane (depth 0.0cm)

Figure A.30: CT-images L09 I (SVV-method, 30 kg/m^3)



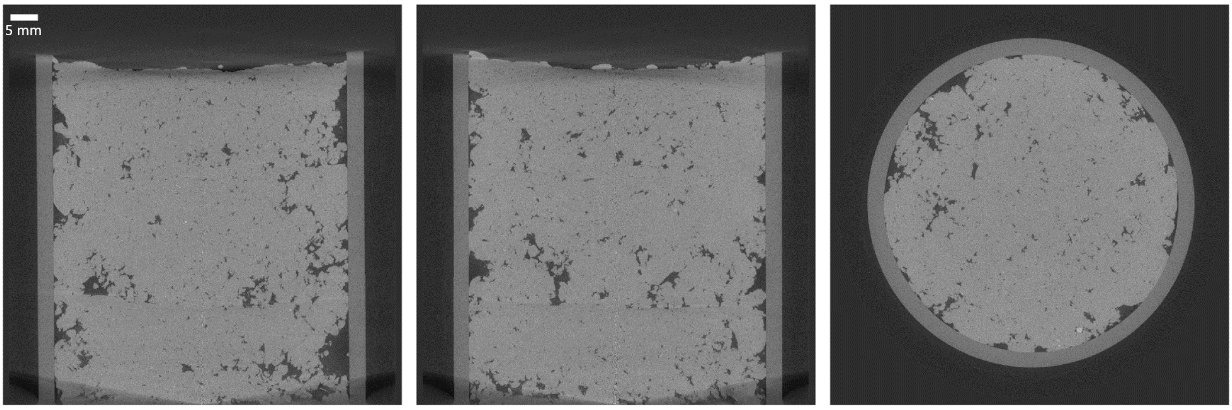
xz-plane yz-plane xy-plane (depth 0.0cm)

Figure A.31: CT-images L19 II (SVV-method, 50 kg/m^3)



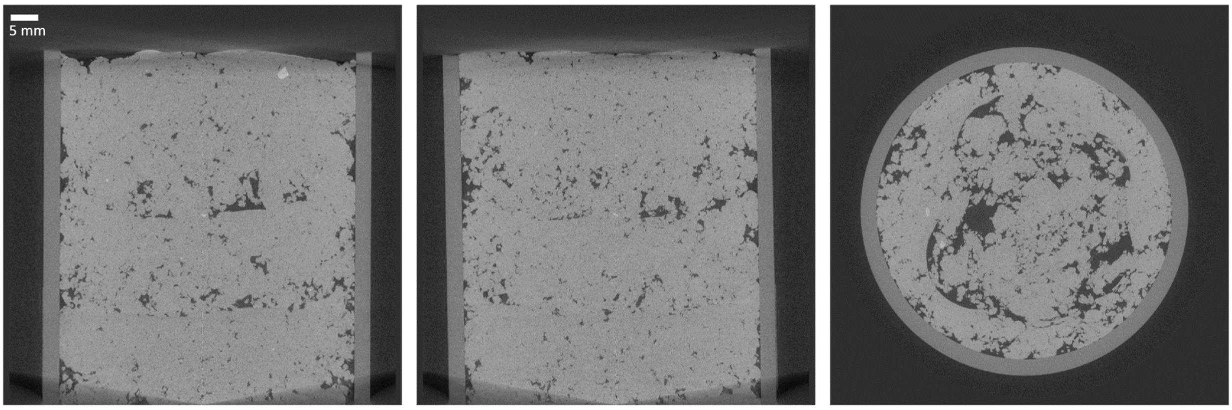
xz-plane yz-plane xy-plane (depth 0.0cm)

Figure A.32: CT-images L29 III (SVV-method, 80 kg/m^3)



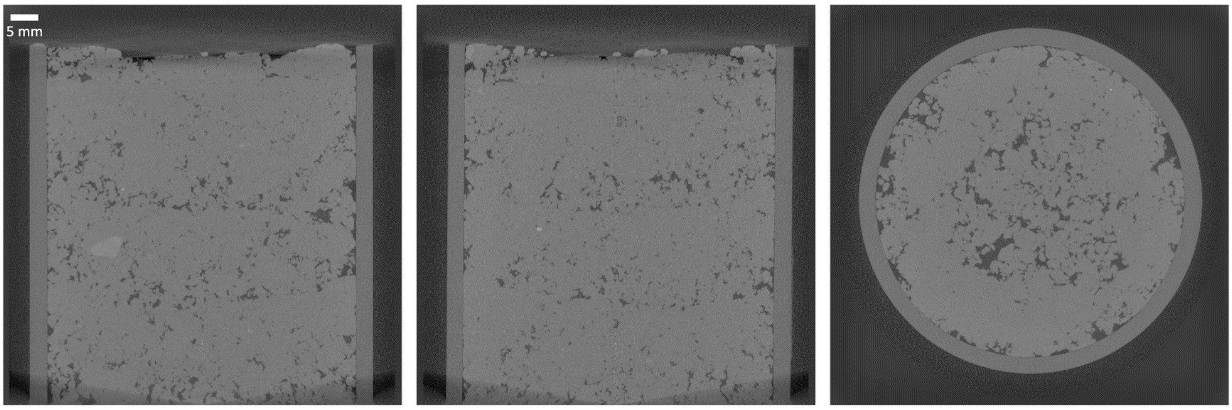
xz-plane yz-plane xy-plane (depth 0.0cm)

Figure A.33: CT-images L10 I (NGF-method, 30 kg/m^3)



xz-plane yz-plane xy-plane (depth 0.0cm)

Figure A.34: CT-images L20 II (NGF-method, 50 kg/m^3)



xz-plane yz-plane xy-plane (depth 0.0cm)

Figure A.35: CT-images L30 III (NGF-method, 80 kg/m^3)

B | Laboratory procedure

B.1 Initial testing (Non stabilised material)

Field clay is tested with standard procedures for:

- Density (Cylinder density and/or ring density)
- Water content
- Falling cone test; undisturbed and remoulded
- Unconfined compression test
- Percussion liquid limit (Casagrande's method)
- Plasticity limit test

Material used for cone penetration test and unconfined compression test can be reused for stabilised sample preparation. A total amount of at least three samples should be prepared for each test and depth range to create an accurate soil profile. As recommended in the "Guideline for lime-cement pile stabilisation" by the Norwegian Geotechnical Association (Norsk Geoteknisk Forening, 2012), it is recommended to initial test each sample tubes/block used as reference material.

B.2 Field sample retrieval and preparation of stabilised material

Sample retrieval is preferably done with piston sampling, but because of the high strength of the lime-cement stabilised columns this method often seems to exceed the capacity of the equipment. An alternative may be to use excavators and hand tools to break of chunks of the stabilised clay.

If the sample is done by piston sampling, the samples is handled regularly as normal soil. If the sample comes in blocks/chunks the soil needs to be packed in watertight plastic bags containing a moist paper cloth to regulate the water content in the material. By arrival at the laboratory facility, 2 water content samples are to be retrieved from the blocks/chunks before storing the samples in the correct temperature, still wrapped in plastic.

The samples need to be prepared by trimming to the correct sample dimensions (d=54mm, h=100mm) before the desired curing time is reached. If the testing is not to happen immediately after trimming, the sample again needs to be wrapped in plastic and stored at correct temperature. Trimming of the samples can be done with mechanical tools as pistons or core rotation bore but are often most successful trimmed with knife by hand due to brittle behaviour in stabilised material.

B.3 Laboratory preparation of stabilised clay samples

Before preparation, a detailed sample plan is made containing at least the number and ID of the samples that is to be prepared, the type of binder (with standardised code, producer etc.) and mixing ratio, binder quantity, preparation method, curing stress, curing time and curing temperature. A minimum number of three samples for each variation per test method should be made. I.e., three samples for unconfined compression test, three samples for triaxial compression test etc. Since some of the methods require a substantial amount of time and/or is very costly it might be unrealistic to get three samples per test method. Therefore, the number of tests should be prioritised according to the information value of each test method and the sensitivity of sample variations.

Mixing of soil and binder

Clay, previously stored airtight and cold, are measured to the necessary amount (m_{clay}) before mixing based on known density (ρ_{clay}). Basically, the amount of clay should correspond to the volume of the total number (n) of cylinder samples (diameter 5.4cm and height 10cm) plus a margin (m_{margin}).

$$m_{clay} = n \cdot (\frac{D}{2})^2 \cdot h \cdot \rho_{clay} + n \cdot m_{margin} \tag{B.1}$$

For mixing, the procedure from the Norwegian Public Road Administration Manual R210 (Statens Vegvesen, 2016) is used:

1. The clay, with known mass, is placed in a steel bowl of a (Kenwood) kitchen stand mixer with a "K-whisk". It is then mixed for 30 seconds on "min" power setting to break down the clay structure.

2. Pre-measured binder amount (based on the clay weight) is then added to the steel bowl and the clay is mixed additionally 30 seconds at "min" power setting.

3. Material attached to the whisk is scraped back into the bowl and the machine is ran one more time.

4. A small sample is retrieved for water content testing and the lime-cement clay is ready for moulding.

Remark: By experience, the maximum capacity of clay and binder, due to the kitchen machine limitation, is typically equivalent to two or three samples. Still, in some cases, the resistance in the clay mix overruns the kitchen machine capacity and the mixing process has to be cancelled before the desired mixing time.

For the determination of necessary amount of binder for mixing, equation B.2 can be used:

$$m_{b,i} = n \cdot \frac{m_{clay}}{\rho_{clay}} \cdot \beta_i \cdot \frac{m_{b,tot}}{1000} \tag{B.2}$$

Where:

$m_{b,i}$ = Necessary amount, binder i (g)

β_i = Mixing ratio, binder i

$m_{b,tot}$ = total specific binder amount (kg/m^3)

Preparation method: Norwegian Geotechnical Association (NGF-method)

Reference: "Veiledning For Grunnforsterkning Med Kalksement-peler", Norsk Geoteknisk Forening (2012)

The build-in is done into pre-cut plastic (typically PET) pipe moulds with inner dimensions d=54mm and h=100mm with known tare. Remark: The manual recommends h=110mm, but for this trial 100mm is used for correct comparison.

Ready mixed clay is placed with layers of 20mm thickness (total of 5 layers). The compression is executed with a brass cylinder/rod of 1200g with diameter 20mm. The soil is compacted with an energy of approximately 0.2-0.25 Nm about 20 hits per layer. At the geotechnical laboratory at NTNU a 1200g piston with a drop weight of 800g is used (see figure B.1). For calculation of energy and drop height, equation B.3 can be used.

$$W = F \cdot h = m \cdot g \cdot h \tag{B.3}$$

Practically this implies that for a drop weight of 800g the drop height is 2.5 to 3.1 cm. After compression the samples are weighed and the ends are sealed by vapour tight plastic and tape. The samples are marked and cured.

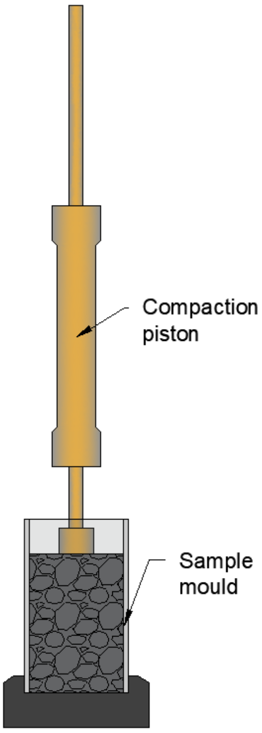


Figure B.1: Principle of the NGF-method

Preparation method: Norwegian Public Road Administration (SVV-method)

Reference: Manual R210, Statens Vegvesen (2016)

The build-in is done into pre-cut plastic (typically PET) pipe moulds with inner dimensions d=54mm and h=100mm with known tare. For the process an air-pressure apparatus with a piston is used (see figure B.2).

Ready mixed clay is placed with layers of approximately 20mm thickness (total of 5 layers). Compression pressure is 200 kPa (2 bar) for 10 seconds per layer.

After compression, the sample is weighed, and the ends are sealed by a vapour tight plastic and tape. The samples are marked and cured.

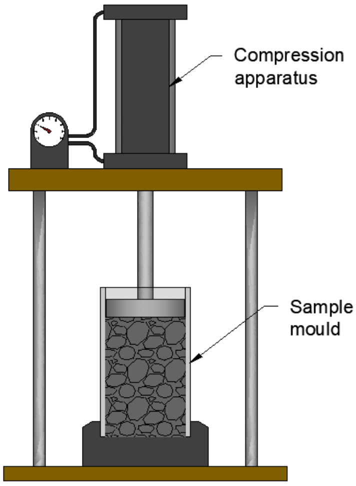


Figure B.2: Principle of the SVV-method

Curing

Curing is done according to the given information. The curing process aims to recreate the field samples and conditions as good as possible. That is: similar curing temperature, curing stress and curing time before testing. The samples are stored in a tight container/bag with a wet paper cloth inside to secure for any potential moisture escape.

B.4 New laboratory sample preparation methods

All samples are moulded in plastic cylinders similar to that for the SVV- and NGF-method. Unless otherwise is mentioned, the mixing and curing procedure for each of the following method is also similar to whats presented in the last section.

A1. SVV-method with increased compression strength

The method is similar to the SVV-method except for an increased compaction stress of 200 kPa. This implies that the samples are moulded in layers of 20mm thickness (5 layers), with compression energy of 400 kPa for 10 seconds per layer.

A2. NGF-method with increased impact energy

The method is similar to the NGF-method except for an increased impact energy of 0.3 Nm. This implies that the samples are moulded in layers of 20mm thickness (5 layers), with impact energy of 0.5-0.55 kPa for 20 hits per layer. For the equipment used in this trial, the drop height of the drop weight is increased with 3.8 cm giving a total of about 6.3 to 6.9 cm.

B1. SVV-method with 10 compaction layers

The method is similar to the SVV-method except for an increased number of compaction layers from 5 to 10. This implies that samples are moulded in layers of about 10mm thickness with compression energy of 200 kPa for 10 seconds per layer.

B2. NGF-method with 10 compaction layers

The method is similar to the NGF-method except for an increased number of compaction layers from 5 to 10. This implies that samples are moulded in layers of about 10mm thickness with compaction energy of 0.20 to 0.25 Nm for 20 hits per layer.

C1 and C2. Dynamic compaction with large Proctor-hammer

A large Proctor-hammer with total weight of 4 480 g and drop weight of 2 660 g is used for sample compaction. The piston diameter is 5.0 cm which will give an even compaction distribution for each layers at the same time that air pockets is allowed to escape from the cylinders. The sample is compacted in 5 layers with 5 hits per layer. The compaction energy is set to C1) 1 Nm and C2) 2 Nm (drop height of respectively 3.8 and 7.6 cm). The mixing and curing process is equal to that of the NGF- and SVV-method.

D1 and D2. Increasing the water and binder amount

The method is separated into D1) 50% water content increase and D2) 50% water content and binder increase (stable w/c-ratio). The water increase is relative to the natural water content.

The procedure is based on a standardised Japanese laboratory preparation for slurry mixing (Kitazume & Terashi, 2013) but with minor adjustments. For mixing a kitchen stand mixer is used. Clay and additional water is placed into the mixing bowl and then homogenised as long as needed (max 5 minutes) at "min" power setting. The binder is then added and the mixing is continued until a smooth consistency is achieved (max 5 minutes). Some small pauses in the process to scrape material by hand from the whisk and edge of the bowl might be needed. Note: as the material dries fast, especially with use of quicklime, it is desirable to use no longer mixing time then needed.

For moulding, the material is built in in 5 layers. For compaction a solid brass cylinder is used to gently compact and squeeze out air pockets. Make sure that the material is sufficiently covering the walls of the cylinder. For the last layer, the top surface is evened by a spatula without adding air to the sample. Typical surface smoothing by "knifing"-movements is most successful. The samples are then cured similar to that of NGF- and SVV-methods.

B.5 Sample testing procedures

The procedure for testing is dependent on the intended test method for each sample. In general, basic information on all samples should be obtained regardless of testing method. That is pictures of samples and density measurement. For samples not significantly influenced by drying, water content measurement for each samples should also be performed.

De-moulding, initial examination etc.

Before testing samples must be prepared. This is mainly done by unwrapping the samples from plastic, and moulds for laboratory prepared samples. De-moulding is done by suitable sample ejectors or similar. The samples are then photographed and weighed. For field samples, which are often not confined by moulds with fixed volume, a dimension control should be performed such that any deviating measurements is taken into consideration when calculating the density. It might also be necessary to adjust the height and surface by a sharp knife to make the sample fit into the test apparatuses.

Unconfined compression test

1. The sample is placed into the apparatus. The top an bottom surfaces are controlled to be relatively flat such that the sample is straight and good contact between the load cell and the sample can be obtained.

2. The load cell is manually adjusted such that contact with sample is reached. The deformation/strain is then set to 0.

3. Test is performed with deformation rate set to 1.5 mm/min (1.5 %/min).

4. When the strength peak is clearly reached the test can be terminated, and the raw data can be retrieved and analysed.

5. With the analyse, shear strength ($c_u = q_u/2$), stiffness (E_{50}), and the failure strain (ε_v) could be determined.

Water content

After testing (typically unconfined compression test) a representative piece (20-30g) of the sample is retrieved and placed into a drying container and weighed. The sample is later dried for at least 2 days such that the water content could be found.

CT-analysis

The samples are placed in the µCT apparatus and scans are performed by qualified personnel. For this trial the settings presented below were used. After the scan the images is then reconstructed and analysed digitally.

Settings	Field samples	Laboratory samples
Tube voltage:	215 kV	225 kV
Current:	173 µA	140 µA
Voxel size:	45 µm	30-40 µm
Time of exposure:	1.42 s	1.42 s
No. of projections:	3141	3141
Filter:		2.0 mm Cu

Porosity estimations is done by choosing a section for each sample to be analysed, as done in figure B.3. Typically, the sample edges is avoided due to irregular cutting for the field samples, as well as wall effects from the mould in the laboratory prepared samples. From the selected section, all horizontal projections are first filtered and then digitally segmented which separates clay material and pores by a given threshold value (see figure B.4). The threshold value will determine the efficiency of which algorithms separates pores and solid material. Since the CT-images resolution is based on the obtained voxel size, these values will be the limitation of pore detection. For this trial, where voxels sizes is set to 30-45 µm, only macro porosity is estimated.

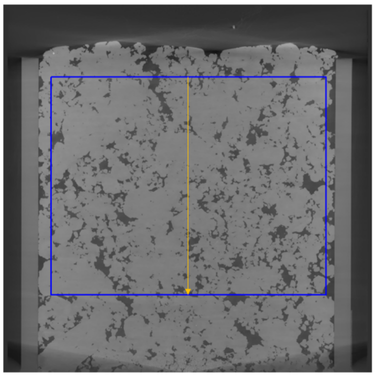
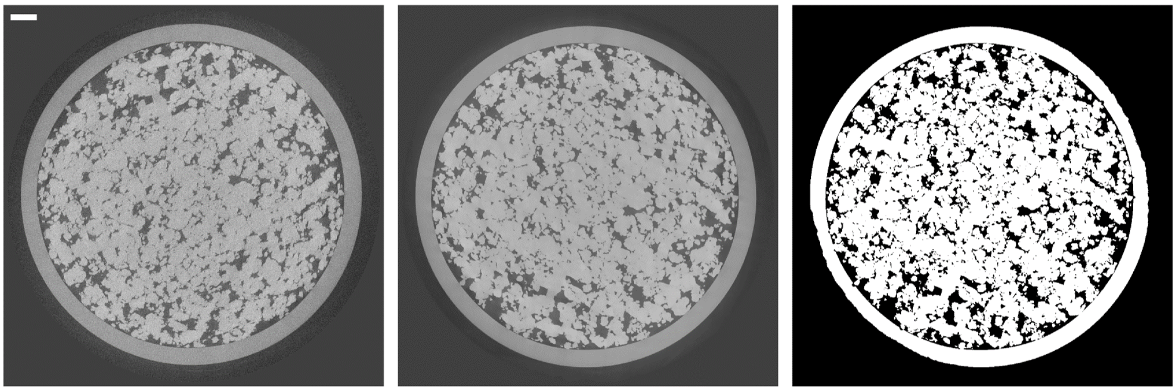


Figure B.3: Principle of CT porosity estimation

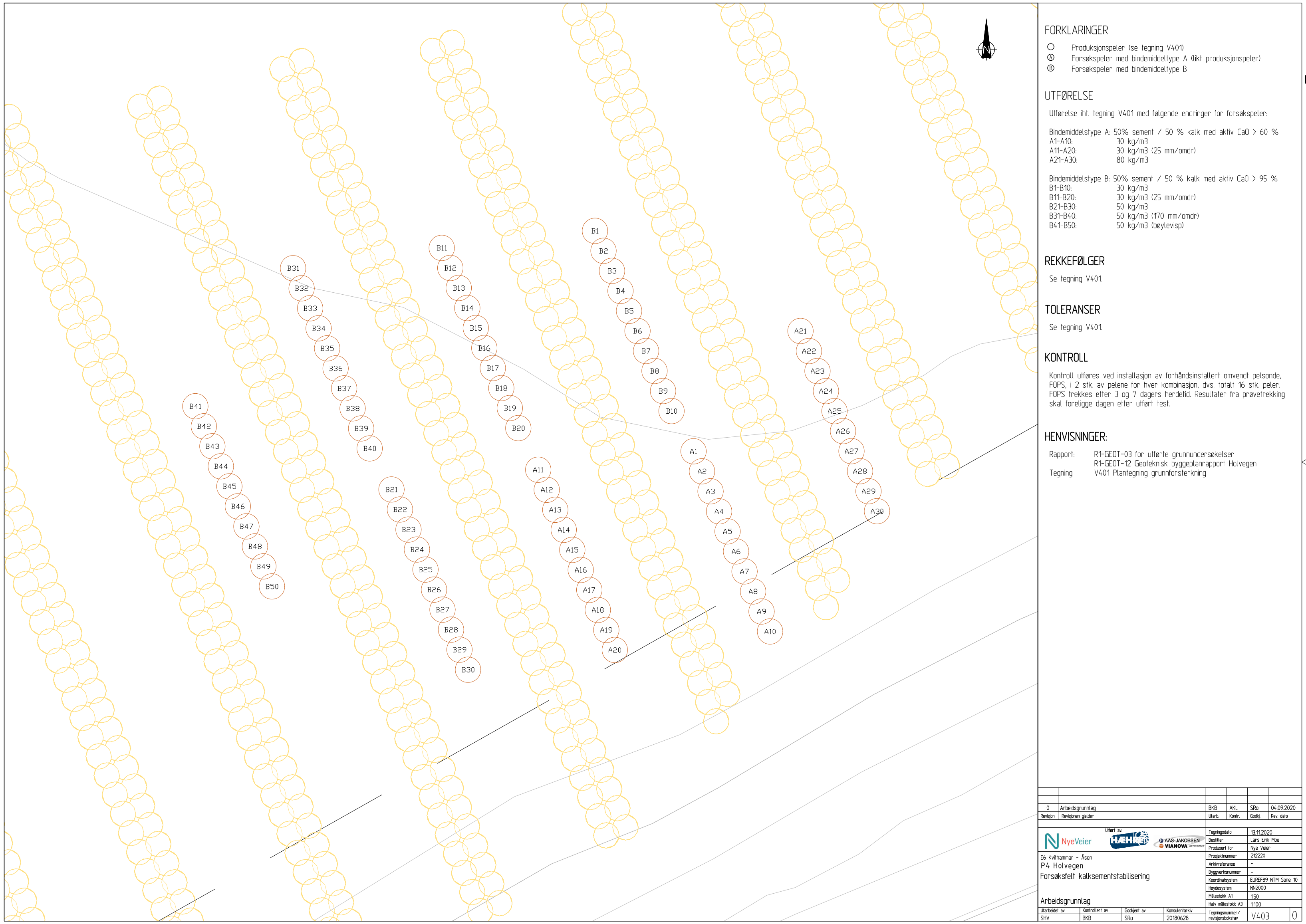


Raw image Filtered image Segmented image

Figure B.4: CT-images segmenting

C | Additional documents

C.1 Pile plan



FORKLARINGER

○ Produksjonspeler (se tegning V401)
Ⓐ Forsøkspeler med bindemiddeltype A (likt produksjonspeler)
Ⓑ Forsøkspeler med bindemiddeltype B

UTFØRELSE

Utførelse iht. tegning V401 med følgende endringer for forsøkspeler:

Bindemiddelstype A: 50% sement / 50 % kalk med aktiv CaO > 60 %

A1-A10:	30 kg/m3
A11-A20:	30 kg/m3 (25 mm/omdr)
A21-A30:	80 kg/m3

Bindemiddelstype B: 50% sement / 50 % kalk med aktiv CaO > 95 %

B1-B10:	30 kg/m3
B11-B20:	30 kg/m3 (25 mm/omdr)
B21-B30:	50 kg/m3
B31-B40:	50 kg/m3 (170 mm/omdr)
B41-B50:	50 kg/m3 (bøylevisp)

REKKEFØLGER

Se tegning V401.

TOLERANSER

Se tegning V401.

KONTROLL

Kontroll utføres ved installasjon av forhåndsinstallert omvendt pelsonde, FOPS, i 2 stk. av pelene for hver kombinasjon, dvs. totalt 16 stk. peler. FOPS trekkes etter 3 og 7 dagers herdetid. Resultater fra prøvetrekking skal foreligge dagen etter utført test.

HENVISNINGER:

Rapport:	R1-GEOT-03 for utførte grunnundersøkelser R1-GEOT-12 Geoteknisk byggeplanrapport Holvegen
Tegning	V401 Plantegning grunnforsterkning

Revisjon	Revisjonen gjelder	Utarb.	Kontr.	Godkj.	Rev. dato
0	Arbeidsgrunnlag	BKB	AKL	SRo	04.09.2020

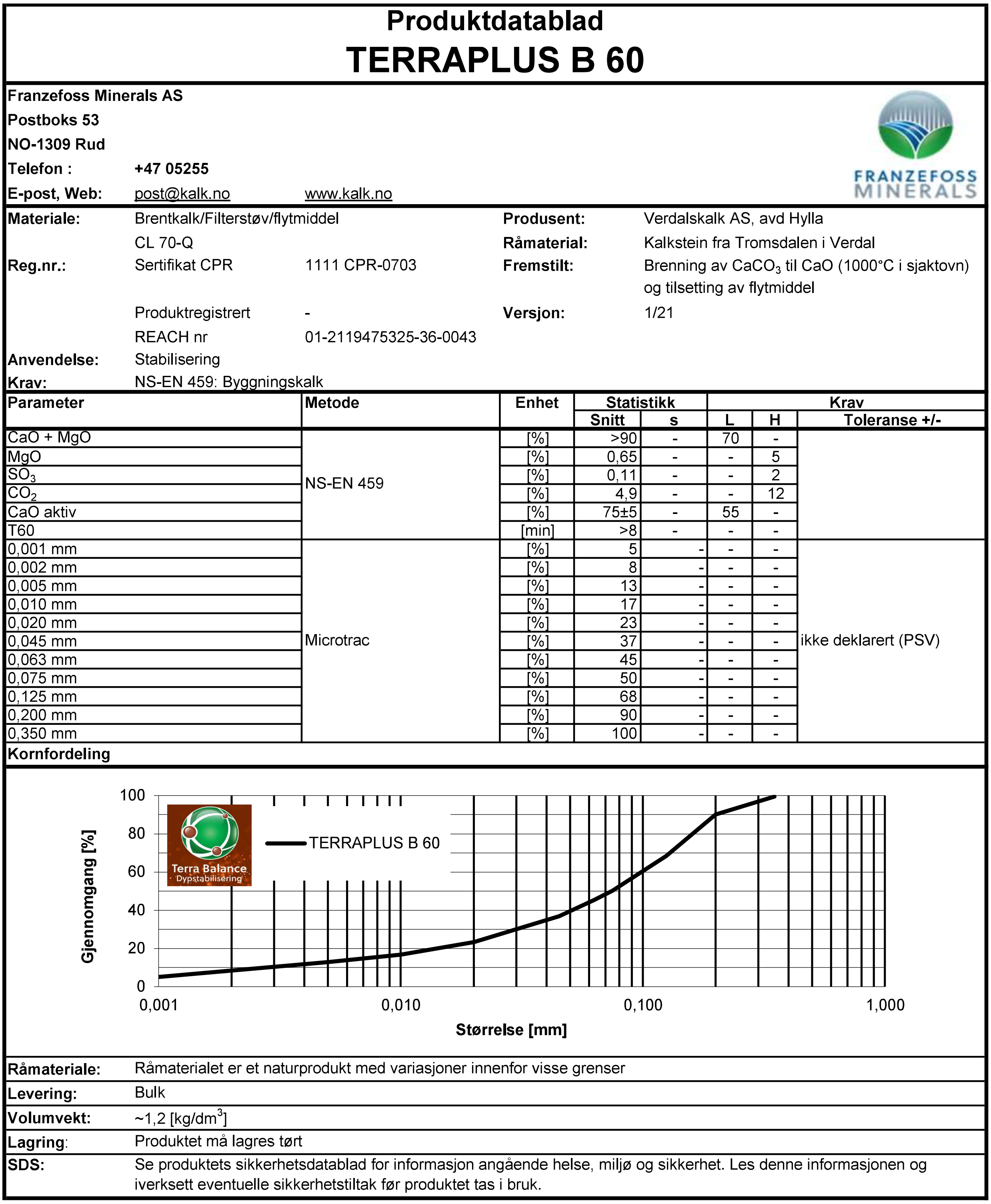
E6 Kvithammar - Åsen
P4 Holvegen
Forsøksfelt kalksementstabilisering

Arbeidsgrunnlag

Tegningsdato	13.11.2020
Bestiller	Lars Erik Moe
Produsert for	Nye Veier
Prosjektnummer	212220
Arkivreferanse	-
Byggverksnummer	-
Koordinatsystem	EUREF89 NTM Sone 10
Høydesystem	NN2000
Målestokk A1	1:50
Halv målestokk A3	1:100
Tegningsnummer/ revisjonsbokstav	V403 0

Utarbeidet av	Kontrollert av	Godkjent av	Konsulentarkiv
SHV	BKB	SRo	20180628

C.2 Product Data Sheet - Terrapluss B60 [In Norwegian]



Produktdatablad

TERRAPLUS B 60

Franzefoss Minerals AS
Postboks 53
NO-1309 Rud
Telefon : +47 05255
E-post, Web: post@kalk.no www.kalk.no

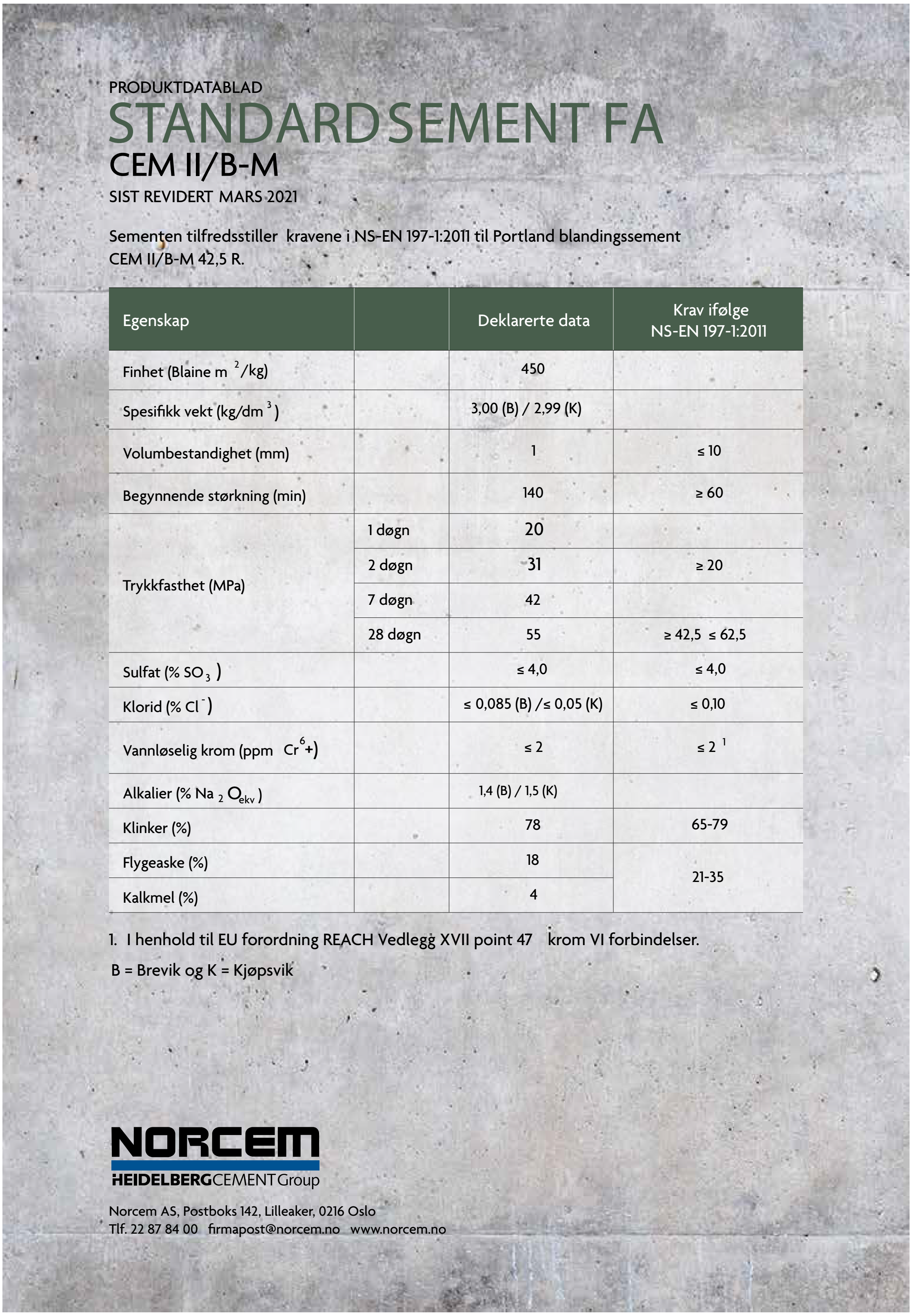
Materiale:	Brentkalk/Filterstøv/flytmiddel		**Produsent:**	Verdalskalk AS, avd Hylla
	CL 70-Q		**Råmaterial:**	Kalkstein fra Tromsdalen i Verdal
Reg.nr.:	Sertifikat CPR	1111 CPR-0703	**Fremstilt:**	Brenning av $CaCO_3$ til CaO (1000°C i sjaktovn) og tilsetting av flytmiddel
	Produktregistrert	-	**Versjon:**	1/21
	REACH nr	01-2119475325-36-0043		
Anvendelse:	Stabilisering			
Krav:	NS-EN 459: Byggningskalk			

Parameter	**Metode**	**Enhet**	**Statistikk**		**Krav**		
			Snitt	**s**	**L**	**H**	**Toleranse +/-**
CaO + MgO	NS-EN 459	[%]	>90	-	70	-	
MgO		[%]	0,65	-	-	5	
SO_3		[%]	0,11	-	-	2	
CO_2		[%]	4,9	-	-	12	
CaO aktiv		[%]	75±5	-	55	-	
T60		[min]	>8	-	-	-	
0,001 mm	Microtrac	[%]	5	-	-	-	ikke deklarert (PSV)
0,002 mm		[%]	8	-	-	-	
0,005 mm		[%]	13	-	-	-	
0,010 mm		[%]	17	-	-	-	
0,020 mm		[%]	23	-	-	-	
0,045 mm		[%]	37	-	-	-	
0,063 mm		[%]	45	-	-	-	
0,075 mm		[%]	50	-	-	-	
0,125 mm		[%]	68	-	-	-	
0,200 mm		[%]	90	-	-	-	
0,350 mm		[%]	100	-	-	-	

Kornfordeling

Råmateriale:	Råmaterialet er et naturprodukt med variasjoner innenfor visse grenser
Levering:	Bulk
Volumvekt:	~1,2 [kg/dm^3]
Lagring:	Produktet må lagres tørt
SDS:	Se produktets sikkerhetsdatablad for informasjon angående helse, miljø og sikkerhet. Les denne informasjonen og iverksett eventuelle sikkerhetstiltak før produktet tas i bruk.

C.3 Product Data Sheet - Cement [In Norwegian]



PRODUKTDATABLAD

STANDARDSEMENT FA

CEM II/B-M

SIST REVIDERT MARS 2021

Sementen tilfredsstiller kravene i NS-EN 197-1:2011 til Portland blandingssement CEM II/B-M 42,5 R.

Egenskap		Deklarerte data	Krav ifølge NS-EN 197-1:2011
Finhet (Blaine m^2/kg)		450	
Spesifikk vekt (kg/dm^3)		3,00 (B) / 2,99 (K)	
Volumbestandighet (mm)		1	≤ 10
Begynnende størkning (min)		140	≥ 60
Trykkfasthet (MPa)	1 døgn	20	
	2 døgn	31	≥ 20
	7 døgn	42	
	28 døgn	55	≥ 42,5 ≤ 62,5
Sulfat (% SO_3)		≤ 4,0	≤ 4,0
Klorid (% Cl^-)		≤ 0,085 (B) /≤ 0,05 (K)	≤ 0,10
Vannløselig krom (ppm Cr^{6+})		≤ 2	≤ 2 [1]
Alkalier (% Na_2O_{ekv})		1,4 (B) / 1,5 (K)	
Klinker (%)		78	65-79
Flygeaske (%)		18	21-35
Kalkmel (%)		4	

1. I henhold til EU forordning REACH Vedlegg XVII point 47 krom VI forbindelser.

B = Brevik og K = Kjøpsvik

NORCEM
HEIDELBERGCEMENT Group

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Fredrik Åsheim Falle

Comparing laboratory and field stabilised clay

NTNU
Norwegian University of
Science and Technology