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Manuscript title: Strength and stiffness on lab-mixed specimens of stabilized Norwegian clays

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Abstract

In situ ground improvement by deep mixing the soil with stabilizing agents or binders as lime, cement, cement kiln dust or a combination of these is the most common method applied in the Nordic countries. In Norway, soil stabilization with deep mixing has been applied to soft to medium stiff clays with sensitivity ranging from about 5 to over 100. In this paper, a large database of laboratory results from lab-stabilized Norwegian clays is presented. The data show, for example, that shear strength of the stabilized clays decreases when the water to binder ratio (i.e. total clay water to binder content ratio) increases, and that the stiffness development is mainly controlled by time. Some observations from more advanced laboratory testing on samples taken from stabilized columns in the field highlight the anisotropic behaviour of the stabilized clays and the increase in strength with in situ stresses. The data presented is a useful guide on the selection of binder mixes for specific project applications and it should always be supplemented by testing on site-specific lab-mixed specimens.

Keywords: Design methods and aids; Geotechnical engineering; Strength and testing of materials

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Introduction

In situ ground improvement by deep mixing the soil with stabilizing agents or binders as lime (L), cement (C), cement kiln dust (CKD) or a combination of these is the most common method applied in the Nordic countries since mid-1970's (Broms, 1984). In Norway, soil stabilization with deep mixing has been applied to soft to medium stiff clays with sensitivity ranging from about 5 to over 100. The method aims to increase the strength and stiffness of these soils by forming columns, panels or blocks of stabilized clay. Most common applications of the method in Norway are improvement of stability of natural slopes, cuts, and deep supported cuts and excavations (Karlsrud, Eggen, Nerland and Haugen, 2015; Karlsrud, 2012; NGF, 2012; O'Sullivan, Quickfall and Terzaghi, 2009). Other recent applications include the use of deep mixed columns for reducing settlements or vibrations of high-speed railways (Madshus, 2013; Holm, Andréasson, Bengtsson, Bengtsson, Bodare and Eriksson, 2002), under wind turbine foundations (Topolnicki and Soltlys, 2012) and bridge foundations (Juvik, Solås, Koa, Bache and Hove, 2019).

Experience demonstrates that the strength and stiffness of a stabilized clay can show large variations (Karlsrud, Eggen, Nerland and Haugen, 2015) and therefore, there is a need for assessing those stabilization works. Testing of stabilized clays can be performed by various in situ tests, laboratory tests on lab-mixed specimens on representative samples, or laboratory tests on samples taken from stabilized columns. Testing of lab-mixed specimens is most common, and part of the normal design process for selecting type and amount of binder to be used in a project. In Norway, the standard practice is to perform unconfined compression tests (UC) on the lab-mixed specimens, but in recent years drained and undrained triaxial tests (TC) and oedometer tests have also been undertaken, but so far mostly for research purposes. Column Penetration Sounding tests (CPS) or Reverse Column Penetration Sounding (RCPS) are most commonly used for verification of strength of columns in the field (Swedish Deep Stabilization Research Center, 2010). RCPS may be supplemented by cone penetration test (CPTU) and core sampling. For a sensitive Norwegian clay from Klett, Trondheim (Juvik, Solås, Koa, Bache and Hove, 2019; Long, L'Heureux, Bache, et al., 2019), the testing of lime and cement (L/C) stabilized clay showed higher values of shear strength in the field than those obtained when mixing and testing samples in the laboratory.

This paper presents a large database of laboratory results from lab-stabilized Norwegian clays. The database mainly focuses on use of binders based on a combination of lime and cement (L/C) or lime and cement kiln dust (L/CKD), which have been most commonly used in Norwegian practice. The paper tries to identify which soil parameters and binder mixes that mainly influence the strength and the stiffness of the stabilized clay and provide guidance on expected values that can be applied in design. The intention is that this should help in guiding selection of binder mixes for specific project applications, but it should always be supplemented by testing on site-specific lab-mixed specimens

2. Deep mixing with L/C or CKD/C as a stabilization method for sensitive clays in Norway

The marine Norwegian soft and sensitive clays typically have water contents in the range 25-65%. Due to leaching they can be very sensitive or quick. Experiences over about 40 years of applications (Karlsrud, Eggen, Nerland and Haugen, 2015) have shown that the dry mixing method (DDM), as most commonly used in Scandinavia, gives good results in terms of increased stiffness and shear strength, and is also a cost-effective improvement method. Mainly in cases where the water content of the clay is low, and the remoulded strength relatively high, the use of DDM will result in poor in-mixing of binders, and desired homogeneity, strength and stiffness of the stabilized clay will not be achieved. The so-called Modified Dry Mixing method (MDDM) have in such cases been successfully used (Karlsrud, Eggen, Nerland and Haugen, 2015). With MDDM water is introduced through a separate pipe to the mixing head in addition to the dry binder powder. This gives some of the same effect as using Deep Wet Mixing with slurry but allows use of the simpler dry mixing rigs.

The type and amount of binder has a large impact on the strength and stiffness achieved. In Norway, it has been common practice to use a 50/50 ratio of L/C, but in recent years CKD has been used instead of L. CKD is a by-product material of the cement manufacturing process, consisting primarily of reactive minerals i.e. calcium carbonate and silicon dioxide. L is mainly composed by calcium oxide (CaO), while C has at least 2/3 of calcium silicates which are richest in CaO. L, C or CKD need the presence of water to react. It is well known that increased cement content tends to give higher strength, but it can affect the achievement of a good in-mixing. When the water in the clay reacts with the cement, hydration of the cement occurs resulting in pozzolanic reactions that gives a product with high strength that increases as it ages. The strength increase in L-stabilization is mainly due to the hydration of carbon oxide (dewatering of the soil) and the precipitation of silica and alumina minerals from the clay, resulting in pozzolanic reactions and an increase of strength. In the case of CKD, it needs the presence of water to react in the same way as cement. Binders that need to act through pozzolanic reactions with the components of the soil particles are more sensitive to the type of soil than binders reacting primarily with the pore water in the soils (Åhnberg, Johansson, Pihl and Carlsson, 2003). Previous studies (Milburn and Parsons, 2004; Parsons, Kneebone and Milburn, 2004; Åhnberg, Johansson, Pihl and

Carlsson, 2003) have addressed the effect of using different binders in ground improvement. These studies report that high-plastic clays stabilized with CKD have approximately the same stiffness as for L-stabilized clays. In other soil types, the tests show a higher stiffness for L-stabilized clays than for CKD-stabilized clays. Outside Norway, the use of other type of binders have also showed good effects. Examples are for instance use of ground-granulated blast furnace (GGBS) activated by carbide slag (CS) (Yi, Gu, Liu and Puppala, 2014; Yi, Li and Liu, 2015), fly ash and lime kiln dust (Kang, Ge, Kang and Mathews, 2015; Kang, Kang, Chang and Ge, 2015; Zentar, Wang, Benzerzour and Chen, 2012).

3. Data collection on stabilized Norwegian clays

NGI has during 2015-2019 compiled a database on lab-stabilized clays from 26 different Norwegian sites. The database includes 424 samples with 325 samples prepared in the laboratory and 99 samples taken from the field. It includes data from UC and TC tests. The present paper will focus on presenting the data from lab-mixed specimens.

As summarized in Figure 1, the database mainly covers clays with water content in the range 20 to 48 %, plasticity index 5-20%, liquid limit 20-50%, and sensitivity 2 to 280. For the samples with sensitivity larger than about 10 to 20, the water content is equal to or larger than the liquid limit. As it will be discussed later, after stabilization the water content reduce and approach the plastic limit of the original clay.

The samples in the database were mixed with different types of binders using different ratios of binder combinations, total binder contents (as expressed by total binder, α), and curing times. Most of the clay samples were L/C stabilized, some CKD/C stabilized, both with 50/50 binder ratio. The most commonly used total binder content (α) in the database is 100 kg/m^3. Figure 2 presents the distribution of the type of binder, binder ratio and binder factor used for the samples in the database.

Figure 3 shows the water content and total unit weight values in situ (i.e. before mixing) against the values for stabilized clays (i.e. after mixing). As expected, the water content of the stabilized clays is about 13% lower than the in-situ values since part of the pore water was needed to complete the binder hydration process. In natural soils, a decrease in water content from around the liquid limit towards the plasticity limit is accompanied by an increase in strength (Åhnberg, Johansson, Pihl and Carlsson, 2003). The binder type showed no significant impact on the water content. The increase in total unit weight because of soil stabilization is relatively small, indicating that the stabilized clay may contain a larger volume of air pockets, also observed by (Åhnberg, Johansson, Pihl and Carlsson, 2003), mainly due to an inhomogeneous preparation of the samples in the laboratory. These air pockets might affect the strength measurements and contribute to its large scatter. The total unit weight of the treated clays seems to mostly vary between ±5% of that of the original clay.

3.1 Some observations from UC tests on laboratory prepared samples

The water content and plasticity index of the original clay have been suggested to influence the shear strength and stiffness of the stabilized clay. Note that the shear strength is defined as c_u=q_u/2, where q_u is the axial stress at failure in the UC test. Figure 4 shows the influence of the water content of the clay before mixing (in-situ) on the shear strength, c_u, of the stabilized clay. The data is for L/C and CKD/C stabilized clays that were cured for 1-6, 7, 14, 21, 28 and 54-60 days. The general trend agrees with past experiences that the strength of a clay stabilized with a specific binder increases with decreasing in-situ water content. In addition, the strength increases with curing time. Higher c_u values are reached for clays stabilized with CKD/C and large curing times, and water contents lower than 30%. The scatter in the data could partly be due to human factors when mixing and testing the specimens rather than the clay type and variability in its mineralogy or other factors not directly reflected in the water content.

Figure 5 presents the variation in c_u with plasticity index, PI before stabilization. The strength decreases when PI increases. Low plastic clays seem to be more sensitive to the type of binder used since higher c_u values are reached with CKD/C treated clays for the longest curing times. Plastic limits for the stabilized clays are not included in the database, however, it is expected an increase in plastic and liquid limit because of the stabilization works, as observed for Swedish clays (Åhnberg, Johansson, Pihl and Carlsson, 2003).

Figure 6 presents the variation in c_u at 28 days with the binder factor α for L/C and CKD/C treated clays. As expected, the strength increases with α and with a reduction in $w_{in\ situ}$. The highest c_u is observed for α = 120 kg/m³ and CKD/C treated clays; however, the tendency of the different binder type is not clear. The most common binder content factor used in the Norwegian practice is α = 100 kg/m³, however, the data shows that it is possible to reduce the binder factor to α = 70 kg/m³ without compromising the required strength. The Norwegian Public Roads Administration requires a minimum strength of 100 kPa for single stabilized columns. When looking at the complete database as presented in Figure 7, there is a large scatter in the values of c_u for α = 100 kg/m³ at different curing times. This larger scatter is observed for both clays stabilized with either L/C or CKD/C. As mentioned before, the scatter in the data could partly be caused by human factors when mixing and testing the specimens. It is to be expected that the type of binder has effects due to differences in geochemical reactions in the soil, and that the reaction times also depends on the binder type (Åhnberg, Johansson, Pihl and

Carlsson, 2003). The same author presented that for Swedish clays with water contents between 78-89% stabilized with L/C, the strength values at 28 days lie in the range 100-125 kPa, which are lower values than the data presented in Figure 6, mainly due to the high-water content of the clays.

From Figure 8 it is difficult to observe a significant impact on the strength c_u of the value of a_w, defined as the ratio between the dry weight of binder to the dry weight of soil before stabilization. There is a large variability for specific curing times (7 and 14 days), probably mainly due to human factors and some variation in the index properties of the original clay. For data coming from a specific site, and for a specific curing time, the strength of stabilized clay increases with a_w, for L/C stabilized clays. The literature (Coastal Development Institute of Technology, 2002) shows that c_u increased almost linearly with increasing a_w irrespective the curing time. This was for a high plastic clay with water content close to 100%. The peak strength increased with a_w and curing time for another clay with water content close to 120%. This increase of c_u with a_w for a specific site stabilized with L/C agrees well to what has been described before in the case of stabilization with only lime (Terashi, Okumura and Mitsumoto, 1977; Bell, 1996; Liu, Indraratna, Horpibulsuk and Suebsuk, 2012). They state that to achieve the maximum cementation effect (i.e. peak strength), sufficient lime must be added to reach the adequate pH conditions to allow the reaction between the lime and clay minerals. An excess of lime may modify the plasticity indices, water content and reactivity of the clay, which might affect the completion of the hydrolysis of the binder. Consequently, there is generally an increase in strength with lime content until peak strength is achieved, however, with further addition of lime, beyond the optimum content, reductions in strength and stiffness can be observed. In the case of cement, (Terashi, Tanaka, Mitsumoto, Niidome and Honma, 1980) and (Zhang, Zheng and Bian, 2017) presented an almost linear correlation between a_w and $c_{u.}$ However, this last observation cannot be verified with the data presented herein due to its large scatter.

A comparison of the water/binder (w/b) ratio (or total clay water/binder content ratio c_w/a_w) for L/C and CKD/C stabilized clays and its influence in c_u is presented in Figure 9. It can be observed that the CKD/C treated clays can show 400 kPa difference in strength for small variation in w/b ratio from 4 to 5, while L/C treated clays may show just 200 kPa difference in the same w/b ratio interval. The data shows as expected, that larger curing time will then increase c_u for higher values of w/b (i.e. c_w/a_w). These results agree with previously published data by (Lorenzo and Bergado, 2004; 2006) and the relation proposed by (Miura, Horpibulsok and Nagaraj, 2001).

Figure 10 shows the influence of curing time on the strength, c_u, and stiffness, defined as the secant elastic modulus, E_{50}, of clays stabilized with L/C or CKD/C. The data confirm the trend of increasing c_u with increasing curing time and α. A similar trend for curing time might be observed for E_{50}; however, the scatter in the data is larger and no particular impact of the binder content, α, can be observed. Some of the low values in c_u and E_{50} at large curing times may indicate anomalies in the stabilized samples that seem to become more important with an increase in strength and brittleness. The highest values of c_u and E_{50} are observed for stabilized clays with CKD/C at 28 days of curing and more than 100 kg/m³ of binder. Similar values of c_u are observed for lower binder factor values at larger curing times for CKD/C stabilized clays, however the stiffness decreases. A more detailed study is intended in Figure 11, where the influence of curing time on c_u is presented for three different clays from the database: Norcem quick clay and Sjølyst clay both with water contents about 41-44% and Vik quick clay with water content of 28%. Three different binder factors are also plotted for either L/C or CKD/C treated clays. As expected, a larger c_u is observed for larger curing times for both types of binders. No particular difference in c_u is observed for the type and the amount of binder for high water contents. For low water contents, as in Vik quick clay, no significant change in c_u is observed for the amount of L/C used; however, the amount of CKD/C seems to have a positive effect in increasing c_u. These results may indicate that at high water contents, the type of binder and the binder amount do not have any significant effect on the strength. However, a further and systematic study should be done to confirm this.

The values of E_{50} in the database varies substantially between about 5 and 200 MPa. The results in Figure 10 show that the stiffness development is mainly controlled by time, while the type of binder used also has some impact. Surprisingly, no significant impact of normalized binder content, α, can be observed.

The observations presented above are specific for Norwegian clays with lower water contents and plasticity than clays presented by, for example, (Zhang, Zheng, Bian, 2017; Coastal Development Institute of Technology, 2002; Åhnberg, Johansson, Pihl and Carlsson, 2003). The database shows also a large variability due to the inherent clay variability, different laboratories performing the tests (i.e. human factors) and the origin of the data. Even though, the observations agree basically to what is expected from the literature and it shows that there can be significant variability that is not directly related to only simple index properties of the clays stabilized.

3.2 Some observations from more advanced laboratory testing on samples taken from stabilized columns in the field

In relation to some construction projects in Trondheim and Oslo, Norway, some manually extracted block samples and cored samples have been collected from columns. Such samples have been subjected to both conventional UC tests and more advanced consolidated drained and undrained triaxial compression (CIUC and CIDC) and extension tests (CIUE), direct simple shear tests and constant rate of strain oedometer tests (CRS). Figure 12 shows the results of tests on block samples from Møllenberg (Trondheim) (Hansson, 2012), Klett (Trondheim) and Oslo S (Oslo) (Bache and Lund, 2018; NGI, 2017). The in situ vertical effective stress for the samples is unknown, therefore the undrained shear strength, $c_{u\text{-}adv}$, is plotted against the sample depth, z. All the data are from L/C stabilized clays with binder ratio 50/50. Curing time varies from 65 days for Klett data to 90 days for Oslo S data. Both data come from undrained TC in isotropic conditions (CIUC). Møllenberg data has curing times between 530-583 days and the testing conditions were either undrained (CIUC) or drained (CIDC). Møllenberg tests were performed without use of back-pressure to have the same saturation conditions as in-situ, since stabilized clays are rarely saturated (Ånhberg, 2004). Undrained tests showed a very low pore pressure response without giving much effect in the strength, and therefore, most of the tests were performed drained (Hansson, 2012). The results of these TC show an increasing trend between the sample depth and the strength and stiffness of the stabilized clay. The scatter in the data is due to variations in the confining stress during the consolidation phase.

The high stiffness values for Møllenberg samples in Figure 12 are from CIUC and CIDC test with samples cured for almost 2 years. The E_{50}-values vary between 100-500 MPa which are substantially higher than those used in common design practice. For Swedish clays, an E_{50} =50-100 MPa is normally recommended, and such values have also commonly been adopted in Norway. Swedish research concludes that the properties of stabilized clays are stress dependent (Åhnberg, 2006a; Björkman and Ryding, 1996), and that the strength and stiffness properties of the stabilized clays therefore will increase with depth.

Triaxial and DSS tests results from Klett and Oslo S (Bache and Lund, 2018) are also shown in Figure 12. The results from Klett show an increase in shear strength with depth, while the results from Oslo S show approximately the same shear strength at the two depths. The L/C cement stabilized clay is a heterogeneous material with relatively large variations within the stabilized body. Therefore, the results usually show some scatter. To capture the effect of stress dependency, one should collect field samples from large depths, so that the stress dependency is not masked by the natural variation in the stabilized clay. Multiple tests performed with RCPS at Klett also showed an increase in shear strength with depth. Figure 13 shows the range of variation of these RCPS tests. The columns are L/C columns installed to a depth of 25 m in a deposit of very sensitive clay. The tests were made 7-12 days after installation. The larger increase after 15 m is mainly due to a larger amount of binder (around 3 times more) used during the DDM. The CIUC and CIUD triaxial tests on stabilized Møllenberg clay (Hansson, 2012), were tested for both vertical and horizontal orientation of the test specimens, under different confining pressures. They showed a clear increase in shear strength shear with confining pressure. For the relatively high confining pressures applied there were no significant differences between vertically and horizontally trimmed specimens (Figure 12).

The increase in shear strength of stabilized clays with depth, or confining stress level, has not normally been reflected in Norwegian design practice. This effect could potentially result in significant cost savings. More research is needed to quantify the contribution of confining stress during curing and stress paths followed during testing on the strength of stabilized clays.

The tests presented in Figure 12 were also intended to study if L/C stabilized clays had an anisotropic type of behavior. The undrained triaxial and DSS tests on samples from 3 and 7 m from Klett, show that stabilized clay is anisotropic. When normalized to the CIUC strength the strength ratios for CIUE/CIUC= 0,35, and DSS/CIUC= 0,77. These anisotropic strength ratios are very similar to what is the case for the non-stabilized clay (0,35 and 0,63), see for instance database by (Karlsrud and Hernandez-Martinez, 2013). However, the results from test on stabilized clay from Oslo S showed less anisotropy, with CIUE/CIUC=0,70. The anisotropy of stabilized clays might be explained a difference in horizontal and vertical effective stress at the end of curing, which will be impacted by the geometry of the stabilized body as well as if or to what extent excess materials are allowed to escape to ground level. This will clearly impact the total stresses set up in the ground during the stabilization works. At Klett, the ground consisted of quick clay which was completely remolded during installation of L/C columns, so that excess masses emerged from the column along the drill string. At the Oslo S, the stabilized clay was less sensitive which would result in relatively speaking larger horizontal stresses than at Klett. The Møllenberg tests could suggest that the anisotropy strength ratio decreases over time as the material cements. However, this was also block stabilized clay which one would expect to leave larger horizontal stresses in the ground than for the panel-type stabilization at the two other sites.

More work is needed to clearly define if or to what extent anisotropy should be accounted for in design, and if or to what extent it depends on the configuration of the stabilized body, the ease with which surplus material can escape to ground level, and the dependency of type of stress path followed during loading. The latter was for instance discussed by (Ignat, 2018).

3.3 More detailed stress-strain behaviour and strain at failure

Figure 14 shows a typical example of stress-strain curve of a lab-mixed specimen of Vik quick clay ($w_{in\ situ}$ = 28%) stabilized with L/C and CKD/C with α = 100 kg/m³. The stress-strain curve of the original clay is also plotted. The treated clay curve shows a very high strength and small axial strain at failure, while the original soil has a small strength and large strain at failure.

Figure 15 shows the relationship between the axial strain at failure, ε_f, and the unconfined compressive strength, c_u, of treated soils with L/C or CKD/C. The axial strain at failure decreases with increasing curing time and increasing cu, which is expected since the clay becomes stiffer. The highest c_u values are for CKD/C treated clays at 60 days curing time. Swedish clays stabilized with different types of binder (Åhnberg, 2006b) decreased rapidly with increasing strength at c_u < 100 kPa, they follow a similar pattern, towards the lower bound, to the one presented for Norwegian clays.

The elasticity modulus E_{50} of treated soils is plotted in Figure 16 against c_u. The values of E_{50} for L/C treated clays vary between 50*c_u and 800*c_u depending on the curing time. A best-fit line for 28 days curing is found at E_{50} = 237*c_u with a coefficient of determination r² = 0,80. In the case of CKD/C treated clays, the values of E_{50} between 55*c_u and 660*c_u depending on the curing time. A best-fit line for 28 days curing is found at E_{50} = 290*c_u with a coefficient of determination r² = 0,75.

As discussed before, the $w_{in\ situ}$ has an influence on the values of E_{50} and c_u. Figure 17 presents the data at 28 days curing time for three different ranges of water content. For water contents between 30-50%, no large E_{50} variations are observed. When the water content is lower than 30%, the stiffness increases almost in an exponential form when increasing the strength.

Conclusions

A large database of laboratory results from lab-stabilized Norwegian clays has been presented. The database mainly focuses on the combination of binders: lime and cement (L/C) or lime and cement kiln dust (L/CKD). The data follows some well-known rules about the influence of the index properties for the original clays, like the water content and plasticity of the clay before stabilization, and the binder dosage, on the strength and the stiffness of the stabilized clay. In addition, it confirms earlier findings that UC-tests on lab-mixed specimens will underestimate the strength that can be achieved on DDM columns in-situ. This is most likely due to a combined effect of higher and longer lasting curing temperatures in the field, and the stresses acting on a column during curing.

The data presented is a useful guide on the selection of binder mixes for specific project applications and it should always be supplemented by testing on site-specific lab-mixed specimens. As a general observation the design strength limit of 150 to 200 (300) kPa used in most projects in Norway up until today, is considered conservative, and could be increased by a factor of two to three in future projects provided proper control during and after in-mixing of binder.

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List of notations

a_w	is the binder content ratio or dry weight binder/dry weight soil ratio
CKD/C	is the cement kiln dust/cement binder
CPS	is the Column Penetration Sounding tests
CS	is the carbide slag
c_u	is the shear strength from unconfined compression tests
$c_{u,\,28}$	is the shear strength at 28 days from unconfined compression tests
c_w	is the total clay water
DDM	is the dry mixing method
DSS	is the direct simple shear
E_{50}	is the elasticity modulus from unconfined compression tests
GGBS	is the ground-granulated blast furnace
L/C	is the lime/cement binder
MDDM	is the Modified Dry Mixing method
PI	is the Plasticity Index
q_u	is the axial stress at failure in the unconfined compression test
r^2	is the coefficient of determination
RCPS	is the Reverse Column Penetration Sounding
TC	is the triaxial compression
UC	is the unconfined compression
w/b	is the water/binder ratio in weight
α	is the binder factor or amount of binder
ε_f	is the axial strain at failure from unconfined compression tests

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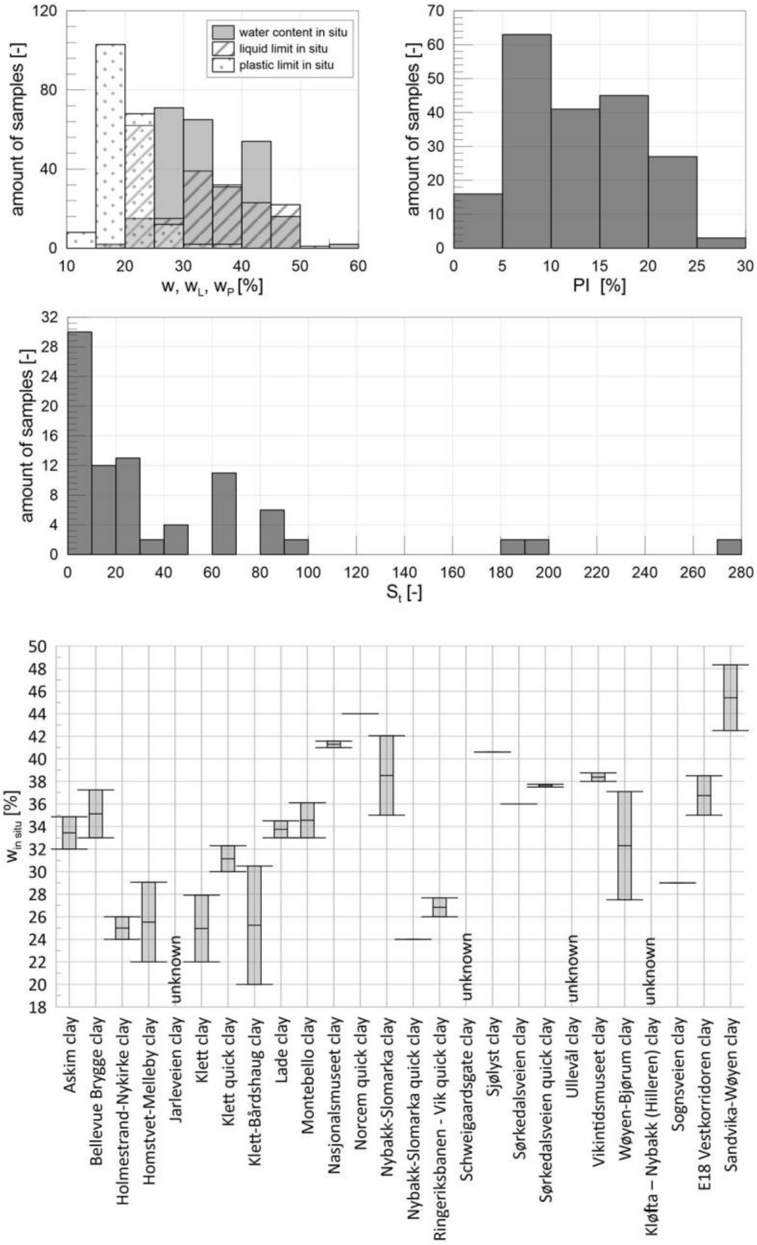

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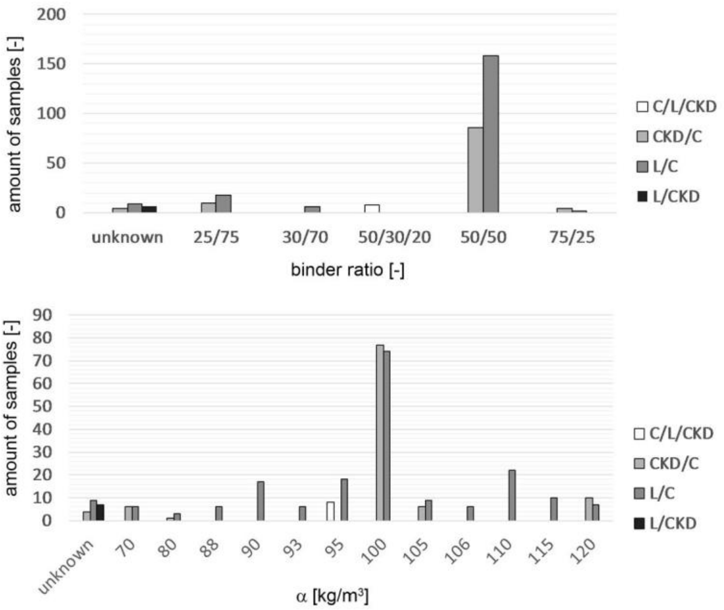

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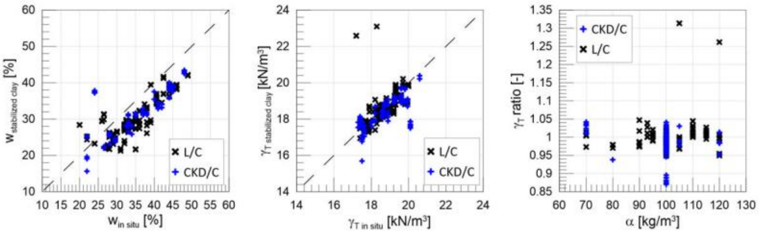

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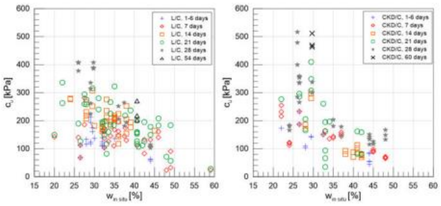

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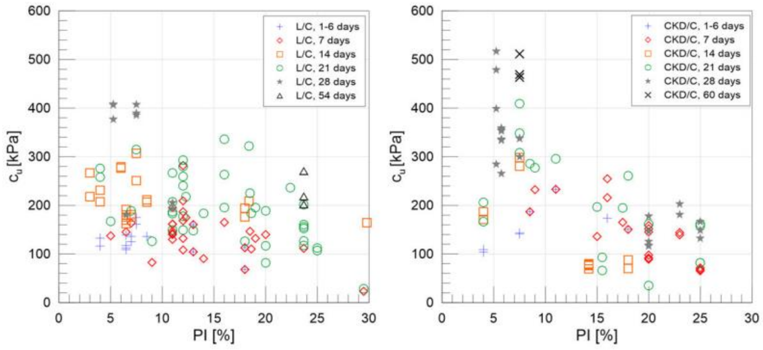

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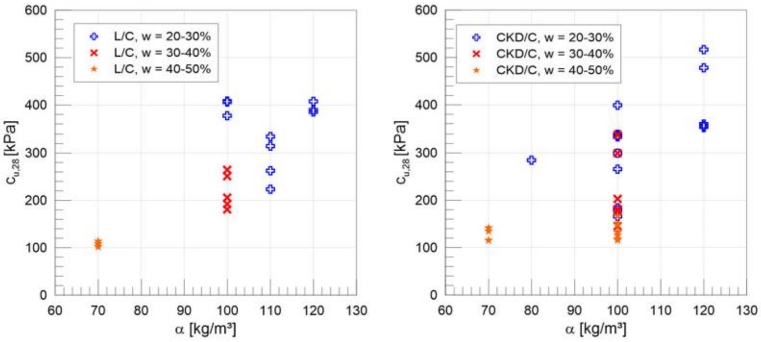

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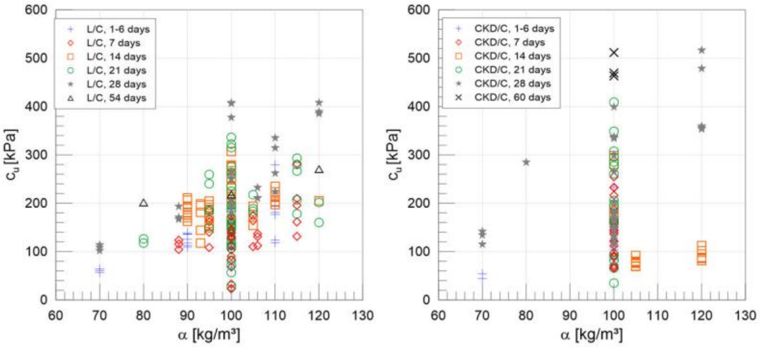

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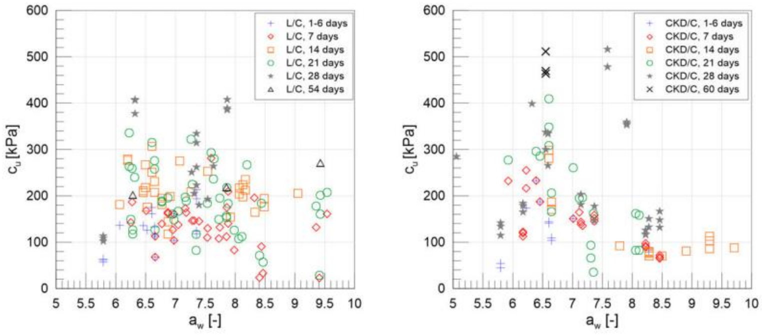

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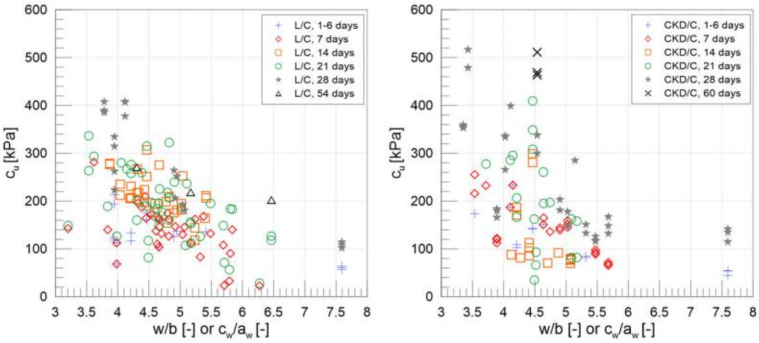

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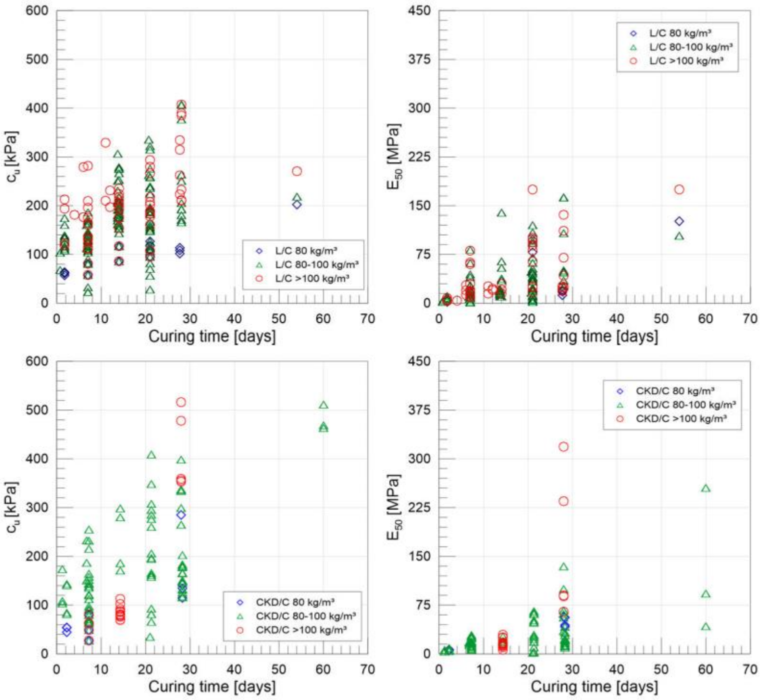

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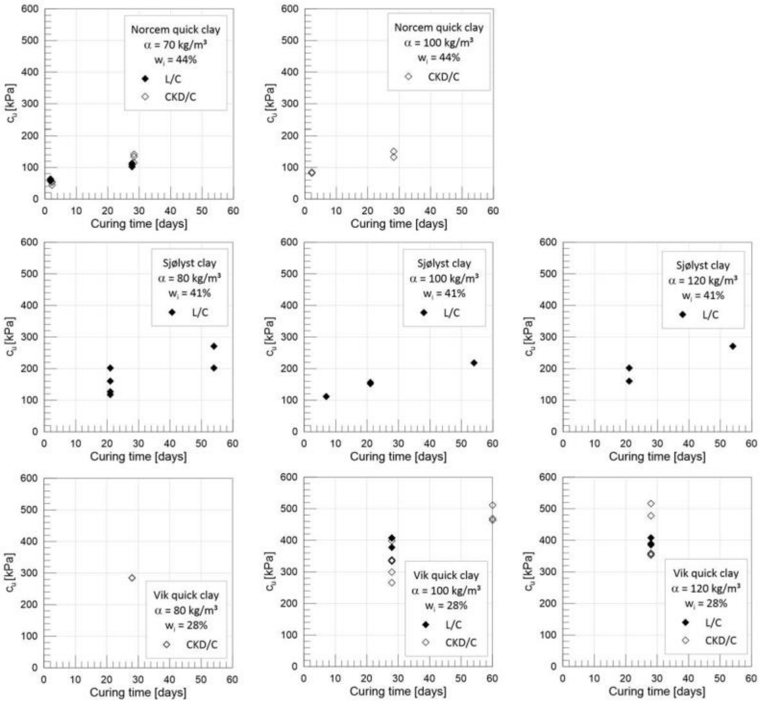

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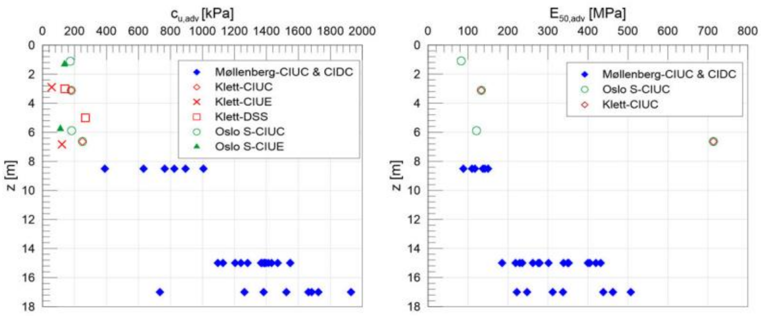

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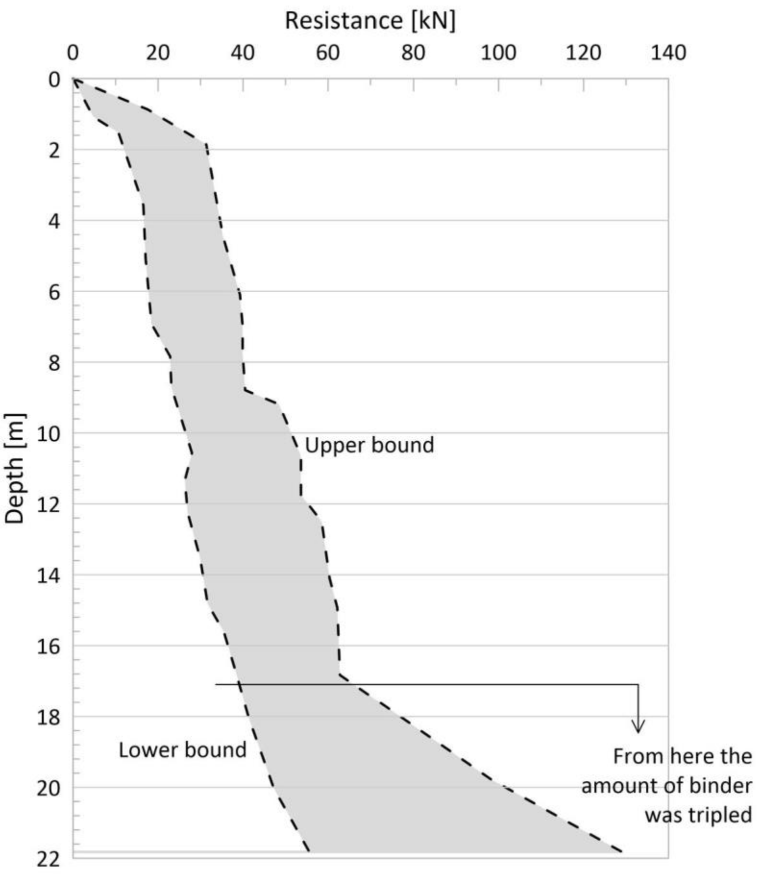

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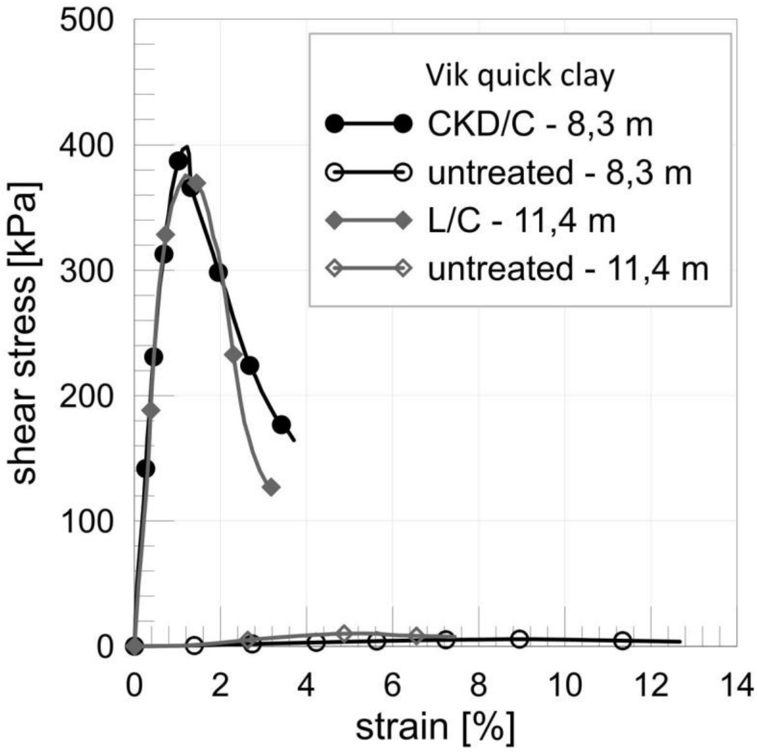

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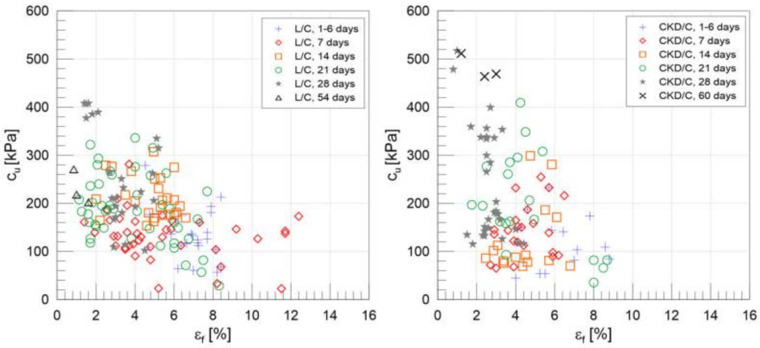

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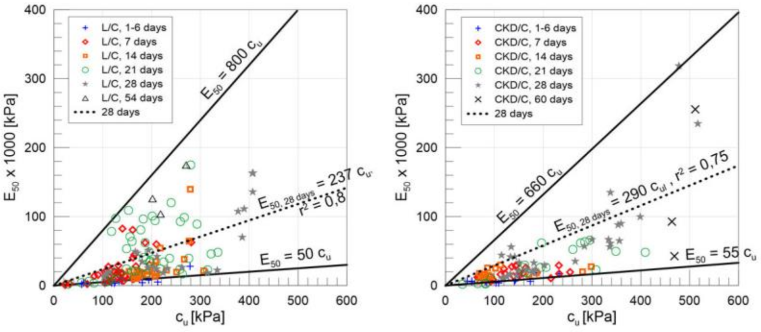

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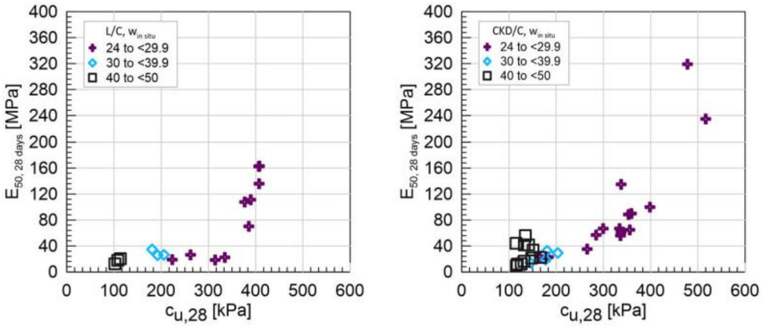

Figure17