Deep soil mixing for stabilising deep excavations

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Abstract. Excavating tunnels in Scandinavia can be difficult and costly due to deep layers of soft clay. To reduce cost and risk, traditional steel struts can be replaced by deep soil mixing (DSM) struts, as is done at West Link project in Gothenburg. DSM struts are made by mixing columns of lime and cement with the clay in the shaft, usually in rows or a grid pattern. Retaining walls can be designed more lightly while reducing deformations that might affect adjacent buildings. Also, earthworks can be performed more easily. However, the safety of the shaft is dominated by the acquired undrained shear strength of the DSM columns. This strength is hard to guarantee compared to steel struts. DSM strength has a reputation of being ‘poorly understood’ or ‘variable’, so authorities apply restrictions to DSM application as bottom struts. This article comprehensively studies the acquired strength of DSM for excavation support. Undrained shear strengths measured in 315 tests were investigated and two full scale test excavations were back calculated. It has been shown that although the achieved shear strength depends on many different input parameters, when these inputs are taken into account, the acquired shear strength can be predicted. Important inputs are curing time and temperature, soil natural shear strength, confining stress, binder quantity and binder composition. Particularly the curing time has a great impact and improvements are suggested for dealing with measurements made with incomplete curing. It is proven that the acquired DSM shear strength can be much higher than current restrictions of 100 kPa and a strength prediction formula is proposed that takes these inputs into account.

1. Problem
Due to the high variability of achieved shear strength, it is common to require laboratory testing and a field test site (with additional testing) to design DSM struts used for excavation support (figure 2). These procedures take up a lot of time in the design phase and give a lot of uncertainty. The goal of this work was to improve the speed of DSM design and to increase the reliability of DSM strength predictions for shaft support. This includes identifying differences between the laboratory results and actual field strength in excavations.

2. Methodology

2.1. Data collection
Traditional design uses mainly undrained compression strength (UCS) tests from the laboratory performed on DSM material on a project-by-project basis. Unfortunately, each project has different soil conditions and different design inputs. Also measurements are made under varying conditions such as different curing times, curing temperatures, test confining stress and test back pressure.
To account for this, data was acquired from a range of different sites and included soil conditions and design inputs for each test. Collected data included soil unit weight, water content, liquid limit, sensitivity, depth below surface and natural undrained shear strength. Project specifics included binder quantity, type and lime-cement ratio. Test specific data included curing time, curing temperature, confining stress (both during hardening and during test) and back pressure. Using a multi-variable analysis, each input factor was tested individually and/or in combination to find a predictor with the smallest possible residual error. To reduce soil variability, the scope has been limited to soft silts and clays in Scandinavia.

**Figure 1.** DSM struts to stabilise excavated shafts.

**Figure 2.** Potential improvement of design cycle with a good DSM shear strength prediction.

| Site                          | Tests          | Full scale test site | UCS | Triaxial | PMT | FOPS | Total | Source                  |
|------------------------------|----------------|----------------------|-----|----------|-----|------|-------|-------------------------|
| Enköping, Sweden             |                | 2                    | 5   | 16       |     |      | 23    | KTH Test Site [6], [7]   |
| Gothenburg, Sweden           |                | 1                    | 43  | 9        | 39  |      | 92    | West Link E02/03/E05[11]|
| Linköping, Sweden            |                | 14                   | 14  | 39       |     |      | 53    | [1], [2], [3]           |
| Löftabro, Sweden             |                | 6                    | 6   | 13       |     |      | 19    | [1], [2], [3]           |
| Trondheim, Norway            |                | 6                    | 6   | 30       |     |      | 36    | Mollenberg [5]           |
| Oslo, Norway                 |                | 10                   | 10  | 28       |     |      | 38    | Schweigaardsgate [9]     |
| SGI Report 17                |                | 12                   | 12  |          |     |      | 12    | [10]                    |
| Oslo, Norway                 |                | 10                   | 10  |          |     |      | 10    | Skøyen [9]              |
| Gothenburg, Sweden           |                | 39                   | 39  |          |     |      | 39    | [8]                      |
| **Total**                    |                |                      |     |          |     |      |       | **322**                 |

**2.2. Data comparison**

The acquired in-situ strength of DSM in field excavations is not accurately represented by laboratory tests. To distinguish differences between the strength measured in laboratory tests and the strength acquired in the field, additional tests were added that relate to field conditions the following was done:

- Laboratory UCS tests on cores taken from field-mixed-and-cured columns.
- In-situ FOPS tests, also known as pull-out tests, performed in-situ on field-mixed-columns.
• In-situ Self-boring Pressiometer Tests (PMT) on field-mixed-columns.
• Back calculation of the KTH full scale field test site in Enköping, which was loaded to failure.

The West Link test site [11] included triaxial tests in horizontal and vertical directions, and previous testing [8] on samples oriented horizontally and vertically showed marginal differences in acquired shear strength, so that strength anisotropy was excluded from this study. Passive loading conditions are still taken into account in the sense that the confining stress levels are considered, which are normally very low at the bottom of an excavation.

2.3. Correction for curing level

The most important contributor to the undrained shear strength of DSM material is how much the sample has cured, which has been previously described by the Maturity Index (M). For this study, all test results were first normalised for a Maturity Index equal to 28 days of hardening at 20°C using [4]:

$$M(T_c,t) = (20 + 0.5(T_c - 20))^{2} t^{\beta}$$  \hspace{1cm} (1)

Where:
- $M$: Maturity index
- $T_c$: Curing temperature in °C
- $t$: Curing time in days
- $\beta$: Hardening factor, found by Åhnberg and Holm to be 0.50

Without correction for maturity using equation (1), measured data would be completely incomparable. When low predicted shear strengths were encountered, this was usually the result of a very low maturity during the test. This was caused in turn by short waiting periods of 7 days or less, but also due to a low laboratory curing temperature of 7°C. This ‘immature testing’ problem has been worsened by Swedish guidelines that prescribe this temperature as the background temperature for curing in laboratory conditions, reasoning that that is the approximate groundwater temperature.

However, in large diameter columns deep in the soil, temperatures rise due to the heat released by quick-lime or cement hydration. This heat cannot dissipate easily as it does in a small laboratory sample. Temperature measurements from the test site at West Link project thus showed hardening temperatures in-situ were 20 to 25°C (figure 4), and one test site in Japan even measured temperatures over 50°C. It is concluded that hardening at such a low laboratory temperature is unrealistic. Importantly, it also causes unnecessary delays to projects awaiting the results; to acquire the same strength as an in-situ sample curing for 28 days at 20°C, a laboratory sample cured at only 7°C would have to cure for 140 days (figure 3). The authors suggest that this practice should be changed to a preferred hardening temperature of 20°C for laboratory testing.

2.4. Relating maturity to acquired strength level

Unfortunately, equation (1) has no clear relation to the acquired long term strength, which is of interest to designers. For example, the acquired strength predicted after 90 days could be a factor 2.5 times higher for a 20°C cured material than for a 7°C sample. The authors believe that the curing temperature should influence the rate of strength gain, but not the acquired long term strength. That is why the following alternative is introduced, also shown in figure 6 (orange line):

$$M(T_c,t) := 1 - e^{-\beta_0 t}$$  \hspace{1cm} (2)

Where:
- $M$: Maturity index
- $t$: Curing time in days
- $\beta_0$: Hardening factor, which may depend on curing temperature ($T_c$) and binder composition.
The level of maturity now has a peak value which can be related to the acquired long term strength. This equation has the added advantage that after fitting the formula, it underpredicts long term strength rather than overpredicting it. When applying equation 1 to this dataset (figure 5, red line), based on the short term tests only, it consistently overpredicted the long term strength, as can be seen in the example in the bottom right curves of figure 5.

Currently the hardening curve is not consistent among all data. For example, it is known that mixes containing mostly cement harden faster than those with quicklime as a primary binder. To prevent excessive extrapolation error, it was decided to remove all samples cured less than 11 days at 20°C or less than 60 days at 7°C from the dataset. This reduced the sample size from 331 to only 168. For the sites where long term data was available, 90% of strength was acquired after 56 to 204 days.

3. Natural undrained shear strength

Another good indicator of the acquired undrained shear strength is the natural undrained shear strength of the soil before mixing (see figure 6). In fact, when looking at mature samples, the ultimate strength that can be achieved is clearly limited for very low natural strength samples. It is plausible that high strength bonds between cement particles do not help if shear failure occurs across other unhardened particles. Norwegian test sites with low water content around 30% and high silt fractions show very high achievable strengths, which is attributed to the base silicate material being stronger for silts.

Based on this knowledge, the following relationship dependant on soil strength is proposed in combination with the maturity index presented earlier.

\[
c_u(t; c_{us,soil}) := c_{us,soil} \left(1 + \beta_0 \left(1 - e^{-\beta_1 \frac{t}{\text{day}}}ight)\right)
\]

Where
- \(c_{us,soil}\) soil natural undrained shear strength [kPa]
- \(c_u\) predicted strength of DSM material [kPa]
- \(t\) curing time [days]
- \(\beta_0\) strength gain factor, determining the long term acquired strength gain of the DSM material as a ratio of it’s initial natural undrained shear strength
- \(\beta_1\) Hardening factor, depending for example on curing temperature (Tc) and binder composition.

Figure 3. Samples cured at different temperatures show equal maturity levels after different hardening periods. Low curing temperatures of 7°C lead to lower maturity than in-situ.

Figure 4. Temperature measurements at West Link test site showed curing temperatures around 20°C. (Wood, 2018).
4. Binder content
The total amount of cement and lime blended into the mix per volume of soil is defined as the binder content, and is an input parameter set by the designer. Commonly, part of the cementitious binder is
substituted by lime, up to 50%. Curve-fitting of UCS tests for the West Link project that were performed with different mixes showed that the substituted part of lime was only 75% as effective as cement in improving undrained shear strength. Thus, it was proposed to introduce an effective binder content defined as:

\[ b_{\text{eff}} = (c_{\text{cem}} + 0.75 \left( 1 - c_{\text{cem}} \right)) b \]  

Where:
- \( b_{\text{eff}} \) effective binder content [kg/m³]
- \( c_{\text{cem}} \) ratio by weight of cement additive
- \( b \) total binder content [kg/m³]

It is noted here that different soils respond differently to lime and cement, so that a general correlation for all soils is not yet achievable. After normalisation for the maturing effect, it is shown that there is also a relationship to the acquired undrained shear strength given by the amount of binder added. Fitting of an S-shaped curve worked best: apparently a small minimum binder amount was needed before any effect could be measured, and there is a maximum level above which the strength improvement levels off (figure 7), which has been previously recognised [2]. Thus, a Sigmoid curve is proposed to predict the undrained shear strength:

\[ c_u (b_{\text{eff}}) = \frac{-\beta_0}{1 + e^{-\beta_1 (b_{\text{eff}} - \beta_2)}} \]  

Where:
- \( b_{\text{eff}} \) effective binder content [kg/m³]
- \( c_u \) undrained shear strength, normalised to 28 days of hardening at 20°C
- \( \beta_0 \) fit factor indicating maximum strength improvement possible, similar to that in Eq. 3. [kPa]
- \( \beta_2 \) fit factor indicating the middle of the s-curve [kg/m³]
- \( \beta_3 \) fit factor indicating the steepness of the s-curve [m³/kg]

Because the factors \( \beta_0 \) and \( \beta_1 \) have no physical meaning, the authors suggest the introduction of alternate factors \( \beta_{15} \) and \( \beta_{85} \) to signify the minimum and maximum effective binder content. These are arbitrarily defined as the binder content where 15% of the strength increase has been acquired and where 85% of the strength increase has been acquired. It is not effective to use any binder contents outside this bandwidth.

\[ \beta_{15} = \frac{\ln \left( \frac{100 - 85}{85} \right) + \beta_2 \beta_3}{\beta_3} \quad \beta_{85} = \frac{\ln \left( \frac{85}{100 - 85} \right) + \beta_2 \beta_3}{\beta_3} \]  

Where:
- \( \beta_{15} \) minimum effective binder content, below which strength gain is not distinguishable [kg/m³]
- \( \beta_{85} \) maximum effective binder content, above which additional binder has hardly any effect [kg/m³]

Within our dataset it can be recognised the Norwegian silty clays, i.e. those with low water content <40%, have a stronger structure that bonds easily with cement and where strengthening kicks in at lower binder contents and increases faster: they thus have lower \( \beta_{15} \) and even lower \( \beta_{85} \) values with a steep S-curve. Also, highly sensitive clays had a lower minimum effective binder content, which is attributed to quick clays being easier to mix evenly. For the whole dataset, a best fit was found with a minimum effective binder content (\( \beta_{15} \)) of 9 kg/m³ and a maximum effective binder content (\( \beta_{85} \)) of 79 kg/m³ which corresponds well to practical experience.

### 5. Acquired shear strength predictor

The first predictor was based on UCS tests alone incorporating soil natural strength, maturity and binder content. The fit is shown in figure 8 and described below (R² = 0.82):

\[ t_{\text{eq}} = \left( \frac{20 + 0.5 \ (T_c - 20)}{20 + 0.5 \ (20 - 20)} \right)^\frac{2}{3} \]  

\[ b_{\text{eff}} = (c_{\text{cem}} + 0.75 \left( 1 - c_{\text{cem}} \right)) b \]  

(8), (9)
Where:

- $c_u$: predicted strength of DSM material [kPa]
- $t$: actual curing time [days]
- $t_{eq}$: equivalent curing time at 20°C [days]
- $T_c$: actual hardening temperature (commonly 7°C or 20°C) [°C]
- $b$: total binder content [kg/m$^3$]
- $c_{cem}$: ratio by weight of cement additive (where other parts may be lime or fly-ash)
- $b_{eff}$: effective binder content [kg/m$^3$], assuming an effectivity of lime of 0.75.
- $c_{cem}$: ratio by weight of cement additive
- $c_u$: predicted undrained shear strength [kPa]
- $c_{u,soil}$: soil natural undrained shear strength [kPa]
- $\beta_0$: strength gain factor, determining the long term acquired strength gain of the DSM material as a ratio of its initial natural undrained shear strength
- $\beta_1$: Hardening factor, depending for example on the curing temperature ($T_c$) and binder composition. For this dataset the factor was kept constant by using an equivalent curing time.
- $\beta_2$: fit factor indicating the middle of the s-curve [kg/m$^3$]
- $\beta_3$: fit factor indicating the steepness of the s-curve [m$^3$/kg]

Although the fit can certainly be improved, it should be remembered that it is performed for a great diversity of locations, curing conditions and binder mix. It is also noted that the fit factors that are used now have some physical meaning and they can be related to a final achievable strength in situ.

### 6. Confining stress

A last important factor affecting the calculated undrained shear strength is the confining stress, either in-situ or within triaxial tests. Figure 10 shows how the undrained shear strength increases dramatically when the confining stress in triaxial tests is increased, and also when compared to the UCS test results at zero confinement. It may also be seen that all the best-fit lines - drawn through the triaxial tests - intersect the vertical axis above the associated UCS test results.

Undrained triaxial tests from Ånhberg showed different behaviour and were omitted from this plot. This is because in those tests a back pressure of 400 kPa was applied, giving very poor undrained shear strength results. During undrained triaxial testing for West Link project, it was observed that applied back pressure – which is applied to the inside of the sample through the plate at the top – was not measured in the pore pressure transducer at the bottom of the sample even after 9 hours. It was concluded that the sample was impermeable, and no back pressure should be applied because this effectively loads the sample (see figure 12). This would be a good explanation why undrained shear strength triaxial tests previously yielded very low values in Ånhberg’s test results, where it may have gone unnoticed because the pore pressure in the sample was not measured. For this study, it is taken for granted that the sample was not fully saturated: in excavations drained behaviour is usually decisive.

For DSM in excavations, confinement dependency is of significant importance: the slope of the more reliable test results range between 0.76 and 1.62 (Figure 9), suggesting high friction angles between 37 and 58 degrees - much higher than the 31 to 33 degrees previously assumed in Swedish practice. Unlike embankments, excavations tend to have very low confining stresses – that is: at the base of the excavation the major principal stress direction is horizontal, and the minor principal stress is vertical, but it is almost zero due to the soil removal.
It is not known how field performance is compared to laboratory behaviour. To understand in-situ behaviour compared to laboratory test results, the KTH full scale test site in Enköping was back-calculated. Full details on the site, which was loaded to failure, can be found in [7]. The plan view is shown in figure 10, and the Plaxis model geometry used in figure 11. Research performed on the West Link test site [11] showed that two-dimensional models provide similar results to three-dimensional models when closely spaced DSM panels are used, thus a Plaxis 2D model was applied. For clarity, a simple elastoplastic soil model was used with a Mohr-Coulomb failure criterion and depth-dependant undrained shear strength, although the Hardening Soil model better simulates deformation.

The result of back-calculating two tests to failure, each with different DSM coverage, showed failure at an in-situ strength of 140 kPa and 185 kPa. To achieve a good fit, correction for confining stress was applied as shown in equation 11, using a high friction angle of 48°. The proposed predictor gives an excellent fit for the test site of 143 and 194 kPa respectively (See table 2). The predictor does not work well with all tests though, as shown in figure 13. Based on triaxial tests alone, the strength would have been overpredicted by a factor 2.6.

\[ c_u (\sigma'_{eq}, c_{u\text{soil}}, b_{eff}, \sigma'_{3}) = c_{u\text{soil}} + (c_{u\text{soil}} \beta_0 + \tan(\phi) \sigma'_{3}) \left(1 - \frac{c_{u\text{soil}}}{1 + \frac{c_{u\text{soil}}}{b_{\text{soil}}}}\right) \]

(11)

Where:
- \(\sigma'_{3}\) minor principal effective stress [kPa], equal to the confinement stress in triaxial tests, horizontal earth pressure in FOPS tests, actual \(\sigma'_{3}\) in back-calculation, and zero in PMT’s.
- \(\phi'\) friction angle of DSM material, averaging 48° for this dataset.
- Other parameters as in equations 8 to 10.
Table 2. Back-calculation of full-scale tests, and predicted strength based on fit formula.

| DSM coverage | Curing time before test | Undrained shear strength [kPa] |
|--------------|-------------------------|--------------------------------|
|              | Curing | Clay | Composite DSM panels + clay | Back-calculated DSM | Predicted DSM equation (11) |
|              | time    | $c_{u,soil}$ | $c_{u,comp}$ | $c_u$ | $c_u$ |
| Test 1 | 18% | 36 | 14 | 37 | 140 | 142 |
| Test 2 | 36 | 51 | 14 | 76 | 185 | 188 |

When viewing figure 14 the laboratory triaxial tests (red) appear to the right of the target line, indicating the triaxial laboratory tests alone would give a consistent overestimation (factor 2.6). UCS tests (blue) are less optimistic but still overestimate (factor 1.6). PMT’s (brown) on the other hand underestimate (factor 0.75). It is suggested that tensile loading dominates the failure mode, explaining why PMT’s fail very quickly: tensile ring stresses are introduced during cavity expansion. Many triaxial tests that gave very high strength were recovered from the field and may be more representative than laboratory UCS tests.

8. Conclusions
The acquired strength of DSM material has been assumed to be so variable that it cannot be accurately predicted and test sites are always needed. It is shown that this is not true: the acquired shear strength is dependant on many factors: curing time, curing temperature, confinement stress, natural soil strength and binder content. This can lead to a lot of confusion, but when these factors are taken into account, reliable predictors can be developed for the final strength using equation (11).

Up till now many tests have been done on poorly matured samples, too early and/or at low curing temperatures. This has led to considerable underestimations of achievable strength and poor predictions. Samples should be tested after 28 or 90 days and be cured at 20°C so that the high achieved strength can be used in practice.

It is crucial to compensate for maturity when comparing shear strength tests. The current maturity curve proposed by Åhnberg fits well but is very impractical because the theoretical strength keeps
increasing for eternity. An alternative maturity curve is proposed that approaches a final achievable strength.

At the same time, many shear strength relationships have been based on triaxial tests under high confining stresses. Based on the test data, a very strong strength gain is created by introducing confining stresses, equivalent to friction angles around 48°, higher than previously assumed. Unfortunately, in excavations the principle stress directions are rotated and the confining stress at the bottom of the shaft is very low, meaning that for excavations the achievable shear strength is much lower than for embankments.

When using DSM as excavation support it is thus essential to have some confining stress, i.e. some weight on the DSM material, implying a DSM strut should always be very thick to prevent near-zero or tensile stresses. It is modelled most reliably as a drained material with low cohesion, high friction angle and positive dilatancy.

The achievable undrained shear strength is also dependant on the initial soil undrained shear strength, which has now been included in the proposed undrained shear strength predictor (equation 11) presented here. The proposed undrained shear strength predictor matches the two back-calculated full-scale tests.

When designing DSM struts, undrained shear strengths for excavations can be based on UCS tests and in-situ small diameter (47 mm) PMT’s, although deep columns cannot be tested by PMT due to deviation from the borehole. FOPS tests are not considered useful because the tension cable fails if they are tested in-situ on DSM material anywhere near full maturity. Triaxial testing can also be used but should always be done at realistic low confining stresses with a stress-dependant strength envelope (frictional). It is believed that the use of back-pressures to achieve saturation causes unintended loading of the sample and this is not recommended.
Furthermore, it is suggested that future projects test their specimens after 28 and 90 days (or longer) and cure at 20°C to improve database comparability. Samples cured shorter than 14 days or below 7°C are so immature they are effectively useless.

9. Future research
In the current proposal, the curing rate is independent of the binder content and mix assumes cured at a constant temperature, none of which is realistic. Further research should focus on improving the maturity curves so that the achievable end strength can be better predicted. It is suggested to take into account the heat generated as indicator of the progression of strengthening, for example by thermally insulating laboratory mixed samples and measuring temperature over time using the apparatus used for the thermal resistivity test.

The authors do not believe strength anisotropy has any practical impact on design, and that this research should be ceased. What can be improved is the strength envelope in shear at low confinement stresses and in tension.

Additionally questions have arisen over site deformations caused by DSM installation effects, which need to be better controlled.

10. References
[1] Åhnberg, H., & Johansson, S. E. (2005). Increase in strength with time in soils stabilised with different types of binder in relation to the type and amount of reaction products. Deep Mixing, 195–202
[2] Åhnberg, Helen. (2006). Strength of stabilised soils – A laboratory study on clays and organic soils stabilised with different types of binder. Construction.
[3] Åhnberg, Helen. (2007). On yield stresses and the influence of curing stresses on stress paths and strength measured in triaxial testing of stabilized soils, 54–66. https://doi.org/10.1139/T06-096
[4] Åhnberg, Helen, Johansson, S. E., Retelius, A., Ljungkrantz, C., Holmgvist, L., & Holm, G. (1995). Cement and lime for deep stabilization of soil [Cement och kalk för djupstabilisering av jord]. Linköping.
[5] Hanson, S. (2012). Lime/cement stabilization at E6 Trondheim-Størdal, Trondheim dayzone west. Norwegian University of Science and Technology, Trondheim.
[6] Ignat, R. (2015). Field and laboratory tests of laterally loaded rows of lime-cement columns.
[7] Ignat, R. (2018). Ground Improvement by Dry Deep Mixing Lime-Cement Column Panels as Excavation Support. KTH, Royal Institute of Technology.
[8] Jonsson, C. (2017). Deep stabilization with cement columns - A laboratory study. Luleå University of Technology.
[9] Karlsrud, K., Eggen, A., Nerland, Ø., & Haugen, T. (2013). Some Norwegian experiences related to use of dry-mixing methods to improve stability of excavations and natural slopes in soft clays.
[10] Trafikverket (2011). Trafikverkets tekniska krav för geokonstruktioner - TK Geo 13. https://doi.org/ISBN: 978-91-7467-114-8
[11] Wood, T. (2018) Fullskaleförsök DDM (Dry Deep Mixing) i passivzon, delprojekt E02 Centralen, Västlänken, Trafikverket ärendenummer TRV 2015/74805

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