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Drained shear strength of Søvind Marl - an overconsolidated Paleogene clay of very high plasticity

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ABSTRACT: This paper examines the drained shear strength of Søvind Marl - an overconsolidated, very high plasticity, marine Paleogene clay, which is fissured with slickensides. The strength is derived from triaxial tests, which are carried out as both drained and undrained compression tests. The failure envelope and its curvature is identified, the nature of effective cohesion and the inadequacies of the Mohr-Coulomb model is discussed. Furthermore, the dependency of the shear strength on plasticity index is examined. The results show that the shear strength decreases when the plasticity increases and only the most plastic Søvind Marl appears to have effective cohesion. The fully softened shear strength and the matrix strength is identified as well. It turns out that both are significantly greater than for many other Danish Paleogene clay types, and can be estimated from a cautious lower and upper bound correlation with plasticity respectively. Finally, the results from the undrained triaxial tests is compared with the results from the drained triaxial tests.

KEYWORDS: Drained shear strength, very high plasticity, fissured, Paleogene clay.

1 INTRODUCTION

For decades, Geo (former Danish Geotechnical Institute) has carried out triaxial tests in order to determine the shear strength of Danish soil types. In the recent years, there has been focus on very high plasticity, overconsolidated clays from the Paleogene period, which all are fissured and with slickensides. In Aarhus (Denmark), interest has been concentrated around Søvind Marl due to construction of high-rise buildings. All tests have been conducted to solve urgent technical problems, but in this article, the collected data is used in a more fundamental study of the effective strength properties of Søvind Marl.

Like London Clay, Søvind Marl show a distinct strain softening behavior, which can be related to a breakdown of interparticle bonding and a dilatant behavior (Burland 1990). Consequently, the strength on a shear surface drop to a reasonable steady value shortly after it has peaked. This value is termed the post rupture strength and is relevant for many stability problems such as first time slides in the weathered top zone and in excavated slopes (Burland 1990).

Tests with London Clay gives strength on the fissures close to the post rupture strength of initially intact samples (Burland 1990). Furthermore, experience shows that the fully softened strength also is close to the Critical State (or constant volume) shear strength, which can be determined by measuring the strength of remoulded, normally consolidated clay (Skempton 1964). Regardless of difference in definitions all these strength addresses the same part of the stress-strain curve and it is therefore not surprising that there is good agreement between the respective strength envelopes.

2 MOBILIZED SHEAR STRENGTH

According to Skempton (1977) field evidence show that the shear strength in first-time slides in London Clay will normally not be less than the fully softened shear strength, φ'_{fs}. However, with further shearing overconsolidated clays experience an additional reduction in strength due to alignment of the clay particles on the failure plane. In extreme cases, the effective residual shear strength becomes the controlling friction angle for ultimate stability.

2.1 *Fissures and slickensides*

In existing fissures with slickensides caused by landslides or by tectonics, e.g. glacial tectonics, the shear strength of the clays is significant reduced.

Coastal landslide areas are particularly vulnerable to extreme strength reduction. The risk of coastal erosion keeping landslides active for so long that landslide movements become so significant that the strength in the fissures drops below the strength at first time failure and possibly decrease to near the residual strength.

At Danish latitudes, isostatic uplift due to removal of large glacial ice sheets and erosion is an important source for the formation of failure lines or zones. Failure during uplift may happen if the soil undergoes passive earth pressure failure because of large reduction of the vertical pressures and large horizontal pressures. Tectonic fissure systems exist in contrast to near-surface desiccation fissures to any great depth, and in practice it is very difficult to identify more than a few of them in a geotechnical investigation and it seems impossible to get a complete picture of the fissure system.

The magnitude of the strength reduction depends on the degree of polishing and thus on the magnitude of the shear strain. The effect depend of the spacing, extent and orientation of the fissures, of the direction of shearing, and of whether potential shear surfaces can seek out the lower-strength fissure. This also applies when the strength is measured in the laboratory. Consequently, fissures introduce major natural variability, and any natural preferred orientation of the fissures means that fissures are more likely to be influential for certain directions of loading (Hight 2007). Therefore, the fissures will give rise to a scatter in laboratory test results. This also applies to differences in test types and test procedures as well as natural variations in soil properties such as plasticity.

In the worst case, the fissure strength risks decreasing to the effective residual strength, φ_r, which for many of the Paleogene clays in Denmark typically are in the order of $\varphi'_r \approx 8^o$ and $c'_r \approx 0$ (Sorensen 2014). This mechanism thus introduces an element of Russian roulette when the shear strength must be derived for design purposes within active or previous landslide areas. This is because the overall stability may be controlled by the residual strength of the periodically active failure planes.

2.2 *Weathering*

Another cause to degradation of strength is physical and chemical weathering caused by precipitation. Under Danish climatic conditions the processes are worsened in the near-surface layers by frost and thawing processes and not least by seasonal drying with the formation of desiccation fissure systems and subsequent softening of the clay when the fissure system are filled with water during rainy periods. The softening accelerates in sloping terrain, due to periodic groundwater flow in the water-filled fissure systems.

According to Mielenz (1959) clay slopes progressively flatten in the course of time until they approached the slopes found in natural hillsides in clay strata. This is in good agreement with Danish experiences. It is well known that the stability of natural slopes in high plasticity clay under Danish climatic conditions must be assessed with effective cohesion c' = 0 and hydrostatic pressure in vertical sections with groundwater at ground level. Correspondingly, one finds for long slopes the following relationship between the slope angle β, the fully softened friction angle φ'_{fs} and the unit weight of soil γ and water γ_w (Skempton 1957):

$$tan\beta \approx (1 - \frac{\gamma_w}{\gamma}) \cdot tan\varphi'_{fs} \quad (1)$$

Chemical weathering result from leaching of calcite and other minerals. The leaching slowly degrades the strength and stiffness of the unweathered clay. The weathering processes are enhanced by the presence of desiccation fissures, which in dry periods open the clay up for acid rain. Softening and leaching are thus companions in natural slopes, but two vastly different weathering mechanisms.

3 THE PALEOGENE STRATIFICATION

Søvind Marl is, as the name suggests, predominantly very calcareous. However, the calcite content varies significant through the formation, and thus calcareous free or slightly calcareous zones frequently alternate with more calcareous ones. Søvind Marl is deposited in a deep ocean and the calcite content originates from coccoliths and planktonic foraminifera (Larsen 2017). Typical results of classification tests are shown in Table 1.

Table 1. Classification parameters of Søvind Marl.

Parameter	Range
Natural water content [%]	29-58
Bulk unit weight [kN/m^3]	17-19
Plastic limit [%]	28-50
Plasticity index, I_P [%]	50-250
Calcite content, Ca [%]	1-65

The natural water content is close to the plastic limit cf. Table 1. This corresponds to a consistency between firm and very firm. Søvind Marl is highly overconsolidated with a geological preconsolidation pressure greater than two MPa and is fissured with slickensides. The overconsolidation is due to removal of younger Neogene layers by erosion and the weight of numerous glaciers during the Quaternary period.

The observed variations in classification properties indicate that also strength and deformation properties vary a lot down through the formations. Variability in CPTu cone resistance confirm that this in fact is the case. When decomposing the cone resistance into a trend component and a fluctuating component, the CPTu's shows that, the fluctuating component is characterized by frequent and significant fluctuations around the trend line (cf. Okkels 2024).

4 TEST PROCEDURES

The triaxial tests in Geo's database have in common that they all exclusively are performed for commercial purposes. Consequently, they are tailored to the respective purposes, the overall quality level and the commercial terms. For example, the primary target could be to determine the strength during undrained conditions or to determine the reloading stiffness under drained conditions. Furthermore, the tests has been carried out over a long period of time (1964-2025) and they therefore reflect changing practice.

All tests in this study are compression tests. The recent triaxial test have generally been performed according to Danish tradition (Jacobsen 1970) on nominally undisturbed specimens with a diameter D of 70 mm and a height to diameter ratio H/D of 1. Lubricated (smooth) end plates have been used in combination with end and side drain. The pore pressure is measured at the bottom cap. The test setup use a fixed top cap system. All samples have been extracted from Shelby-tubes with an inner diameter D of 70 mm.

The test set up including the samples are saturated using backpressure and typically consolidated to a cautious estimated preconsolidation stress level to reduce the effect of the inevitable sample disturbances (Jacobsen 1970). The preconsolidation are in most cases performed isotropic. After the preconsolidation the specimen is unloaded to the in situ stress level (or swell pressure level if observed during initial saturation) and then sheared in either drained or undrained compression to failure. A few multiple tests have been carried out. After the first shear phase, the specimen is reloaded to the preconsolidation pressure and then unloaded to a new however higher stress level than the previous shear phase and sheared again (this last step may be repeated). However only the result of the first shear phase is included in this study, as the subsequent shearing are not surprisingly clearly affected by strain softening.

The latest tests are carried out according to the guidelines in DS/EN ISO 17892-9:2018. The rate of strain during shearing is determined accordingly to ensure sufficiently slow rates to achieve full equalization of pore water pressures in the specimen. For high plasticity clay, this means that the applied strain rate is down to around 0.02 %/hr.

In all cases the plasticity index (I_P) and the calcite content (Ca) has been determined on the specimen material itself and/or the trimming material.

The shear strength is derived as the maximum value within the additional strain during shearing interval 0-10%.

Older triaxial tests have in contrast typically been performed with a height to diameter ratio of 2:1, a diameter of 35.7 mm and rough end platens. Samples were usually extracted using a 42 mm inner diameter sampler. Preloading was in some cases carried out under isotropic conditions and in some cases omitted. Saturation was in most cases carried out without the application of backpressure and compression was performed with pore water pressure kept equal to zero. Hence, undrained compression was achieved by adjusting the cell pressure during testing to obtain constant volume.

The applied strain rate during the compression stage was generally higher than recommended today to ensure full equalization of pore water pressures within the specimen. Hence, the actual effective stresses at the failure state are somewhat uncertain because the pore water pressure is unknown.

A total of 46 different triaxial tests are carried out with intact samples of Søvind Marl. An overview of the test types is presented in Table 2 and the results appear in Figure 3.

Table 2. Triaxial tests with Søvind Marl from Geo's database.

Test status	Test type	D (mm)	H/D	n
New tests (<30 yrs. old)	CID	70	1	20
New tests (<30 yrs. old)	MCID	70	1	1
New tests (<30 yrs. old)	CIU	70	1	11
New tests (<30 yrs. old)	CAU	70	1	3
New tests (<30 yrs. old)	MCAU	70	1	3
New tests (<30 yrs. old)	MCAU	70	1	2
Old tests (>30 yrs. old)	MCID	35.7	2	2
Old tests (>30 yrs. old)	$MCIU_{u=0}$	35.7	2	4

In addition to the test in the table, Russell Geotechnical Innovations (RGI) has carried out four CIU tests with remoulded and consolidated samples according to English tradition, i.e. with height to diameter ratio of 2:1 and a diameter of 100 mm. Unfortunately, both the plasticity I_P and the procedure for preparing the specimens is not known.

5 EFFECTIVE SHEAR STRENGTH PARAMETERS

The derived peak shear strengths for all tests performed with intact samples are presented in Figure 1. As can be seen, there is a large scatter around a best fit line corresponding to φ' ≈ 32.1º and c' = 28.3 kPa. Note that the undrained tests with remoulded Søvind Marl (crosses) plot around the lower value envelope, corresponding to the fact that this line probably represents the fully softened secant friction angle.

For fissured clay that has not been subject to previous shearing, there are two bounds to the effective strength parameters: an upper bound given by the parameters defining the peak failure envelope for the intact clay that is without fissures and a lower bound given by the parameters defining the strength in a fissure (Hight et al. 2007).

Most results in Figure 1 are more or less affected by the fissure strength. This means that the mean value line in Figure 1 does not represent the strength of the intact Søvind Marl, defined as the matrix strength, φ'_{matrix}, as the measured strengths depend on the orientation of the fissures embedded in the specimen and consequently on coincidence and choices of test type, sample size etc.

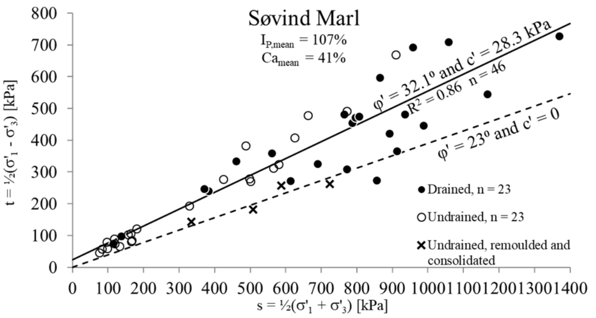

Figure 1. Results of 46 undrained and drained triaxial tests with intact samples of Søvind Marl and 4 triaxial tests with remoulded and consolidated samples. I_P varies between 42 and 192% and the calcite content varies between 8 and 64% for samples where such classification tests were performed.

Figure 1 indicates a preliminary lower value envelope corresponding to φ' ≈ 23º and c' = 0 because this envelope, with a single exception, delimits all test results with intact samples and intersects the results of the four CIU tests performed with remoulded and consolidated Søvind Marl. The results of the last mentioned tests represent for the London Clay approximately the fully softened strength (Burland 1990), which is also assumed to apply to Søvind Marl; however, a single value implies even lower strength parameters.

The fully softened strength must not be confused with the residual strength, which is much lower and only reached after much larger shear strains (>>10%).

Based on the above, the preliminary lower bound value line in Figure 1 is assessed to correspond to the fully softened strength envelope relevant for first-time slides.

6 EFFECTIVE SHEAR STRENGTH AND PLASTICITY

For stiff clays containing plate-shaped clay minerals a correlation between the fully softened friction angle φ'_{fs} and plasticity index I_P should exist (Mesri and Shahien 2003).

In Figure 2, the dependency of the shear strength on plasticity index I_P is examined. The Figure show results from both drained and undrained tests. Figure 2a, 2b and 2c show t-s diagrams for specimens of Søvind Marl with respectively I_P less than 80%, I_P between 80 and 120% and I_P greater than 120%.

Each figure also shows the corresponding effective strength parameters found by linear regression and the preliminary lower bound envelope from Figure 1. According to the regression lines in Figure 2a-2c the shear strength decrease when the plasticity increase and only the most plastic Søvind Marl appears to have effective cohesion.

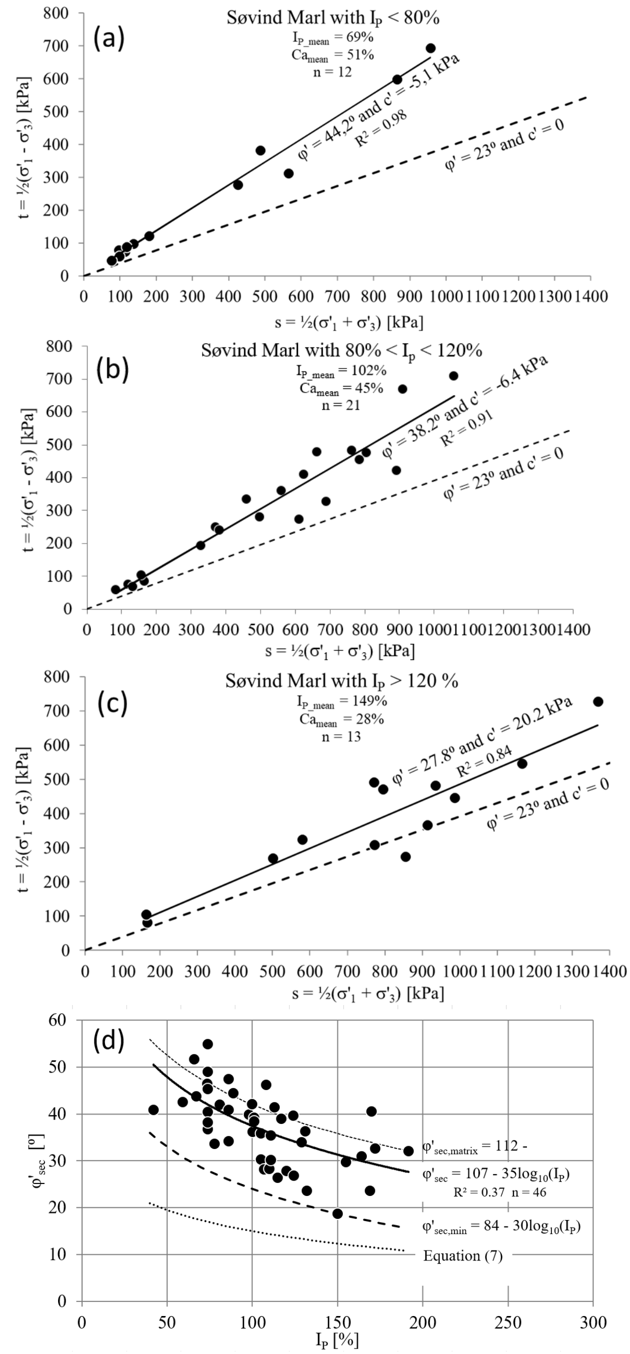

Figure 2. The effective shear strength (a-c) and secant friction angle of Søvind Marl versus plasticity (d).

According to Figure 2 the frequency of measurements affected by fissures increases with plasticity. Thus, the influence is hardly to detect as long as the plasticity of the Søvind Marl is less than 80%. When I_P is between 80 and 120% the influence is detectable, and it is clear when I_P is greater than 120%. This is in accordance with results of the engineering geological soil description (Galsgaard, 2025) of the entire sample material extracted from the boreholes, where almost no slickensides are observed in the least plastic zones of the Søvind Marl formation.

The secant friction angle φ'_{sec} derived from the same test is plotted against I_P of the specimen in Figure 2d. A new lower bound envelope is added with support in the results of the tests with remoulded samples ($\varphi'_{sec,min} \approx 23°$ for $I_P \approx I_{P,mean}$) cf. equation (6) and for comparison the fully softened friction angle for Paleogene clays according to the later mentioned equation (7). The lower bound is set with support in the secant angle to which the strength drops at the end of the tests (not shown).

The large scatter in Figure 2d indicate that the effective secant friction angle depend strongly on the fissures. A weak but nonetheless genuine dependency of plasticity is detected as well:

$$\varphi'_{sec} = 107^o - 35^o \cdot log_{10}(I_P(\%)) \quad (2)$$

According to the regression lines in Figure 2a-2c only the most plastic Søvind Marl appears to have effective cohesion. This means that the regression line in Figure 2a almost represent a common φ'_{sec} for Søvind Marl with I_P less than 80%. If the regression lines in Figure 2b and 2c are forced through origin a common φ'_{sec} can be derived from these two figures as well cf. Equation 3-5.

For I_P < 80%:

$$\varphi' = 44.2^o \text{ and } c' = -5.1 \text{ kPa or } \varphi'_{sec} = 43.8^o \quad (3)$$

For 80% < I_P < 120%:

$$\varphi' = 38.2^o \text{ and } c' = -6.4 \text{ kPa or } \varphi'_{sec} = 37.7^o \quad (4)$$

For I_P > 120%:

$$\varphi' = 27.8^o \text{ and } c' = 20.2 \text{ kPa or } \varphi'_{sec} = 29.1^o \quad (5)$$

Consequently Figure 2a-2c confirm the φ'_{sec} dependency of I_P barely detected in Figure 2d ($R^2 = 0.37$) due to the dominant dependence on structure.

The most plastic samples typically appear more fissured and slickensided than the less plastic samples indicating that fissure intensity depends on the plasticity of the marl. Figure 2 confirm this.

The new lower bound envelope for φ'_{sec} in Figure 2d clearly depend on the plasticity index. The results indicate a logarithmic function like the one shown in the figure.

The new lower bound envelope in Figure 2d is assessed to correspond to the fully softened friction angle φ'_{fs} relevant for first time slides. Since $c'_{fs} = 0$, the fully softened friction strength of the clay is thus also a function of I_P:

$$\varphi'_{fs} \approx \varphi'_{sec,min} = 84^o - 30^o \cdot log_{10}(I_P(\%)) \text{ and } c'_{fs} = 0 \quad (6)$$

Consequently, the preliminary lower bound envelope in Figure 1 only represents φ'_{fs} of Søvind Marl with a plasticity index that corresponds to the mean value of the plasticity index of all samples ($I_{P,mean}$ = 107 %).

We recommend to perform additional tests with remoulded and consolidated Søvind Marl to verify the deduced fully softened shear strength and to document how this strength depends on plasticity.

7 THE EFFECTIVE COHESION

According to the Mohr-Coulomb failure criteria, the effective shear strength is a function of the effective cohesion and the effective friction angle. However, it is generally recognized that the shear strength envelope is curved and that it passes through origin without an effective cohesion intercept (de Beer 1970 and Chandler 1984). Consequently, the Mohr-Coulomb failure criteria can only be considered as a practical approximation to the effective strength envelope (Kulhawy and Mayne 1990).

In Figure 3 below, a power function has been used as a regression line. As expected, a somewhat better description of the measurement results at small stress levels is achieved, supporting that also the strength envelope for Søvind Marl is described more precisely with a curved function passing through origin, i.e. without an effective cohesion intercept. The results of the four old tests (u=0) plot within the same range as the new tests at small stress levels.

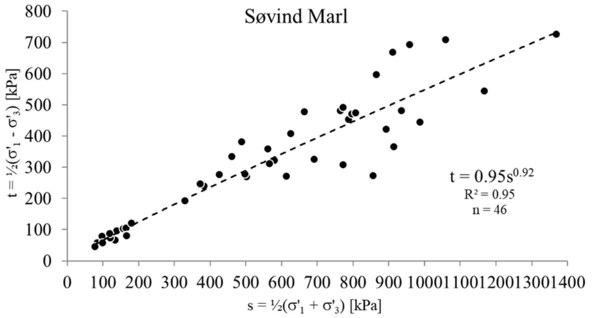

Figure 3. Mean effective shear strength for Søvind Marl using a power function.

8 DRAINED VERSUS UNDRAINED TESTS

Drained triaxial tests are compared with undrained tests in Figure 4. The trend lines indicate that the strength from undrained tests is almost identical with the strength from drained tests when s < 500 kPa. The largest deviation is found for test with Søvind Marl at large mean stress levels. This may be because the dilation causes the pore pressure in the slip surface to be overestimated when the effective strengths are calculated in the undrained tests. However, it could also simply be due a random deviation because there are relatively fewer results of undrained tests performed at high mean stress levels.

Taking into account the variations in test methods and the large random variations of the natural strength due to structure, the deviations between the results of drained and undrained tests are assessed to be without significance for many practical purposes.

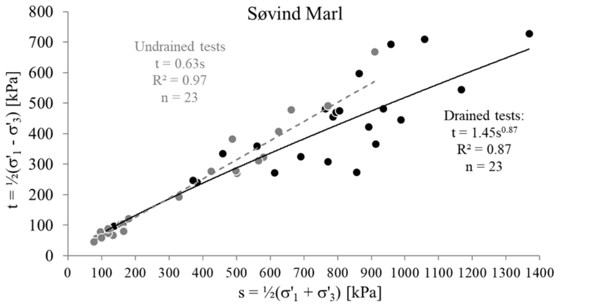

Figure 4. Drained triaxial test results versus undrained triaxial test results (grey dots and dashed regression line = undrained; black dots = drained).

9 PREDICTIONS OF FULLY SOFTENED STRENGTH

Based on the calculation of precisely stable/unstable slopes and with the support of triaxial test results and English experience, the following lower bound envelope for the fully softened effective strength parameters for Danish Paleogene clays is derived (Knudsen 1981 and 1991):

$$\varphi'_{fs} = 45^o - 15^o \cdot log_{10}(I_P(\%)) \text{ and } c'_{fs} = 0 \quad (7)$$

In 2013, Geo performed a review of this equation based on Geo's database at the time, a statistical processing of the data and foreign experience. This work resulted in a correction of the equation (Sorensen and Okkels, 2013).

For $I_P < 50\%$:

$$\varphi'_{fs} = 44^o - 14^o \cdot log_{10}(I_P(\%)) \text{ and } c'_{fs} = 0 \quad (8)$$

For $50\% < I_P < 150\%$:

$$\varphi'_{fs} = 30^o - 6^o \cdot log_{10}(I_P(\%)) \text{ and } c'_{fs} = 0 \quad (9)$$

Based on a literature study, a cautious lower bound envelope for the effective strength parameters of normally consolidated clays was derived as well (Sorensen and Okkels 2013):

$$\varphi'_{NC} = 39^o - 11^o \cdot log_{10}(I_P(\%)) \text{ and } c'_{NC} = 0 \quad (10)$$

The strength of normal consolidated clays is interesting in this context, because for London Clay, among others, it has been shown that the strength of reconstituted samples roughly corresponds to the fissure strength of overconsolidated high plasticity clays (Burland 1990).

In Figure 5, the fully softened strength is calculated from plasticity indexes (I_P) using Equation (6). Moreover, the same strength for the London Clay is added to the figure. The fully softened strength parameters was determined to correspond to $\varphi'_{fs} = 20^o$ and $c'_{fs} = 0$ for London Clay with $I_P \approx 52\%$ (Skempton et al. 1969).

For both London Clay and Søvind Marl it apply that the fully softened strength is derived with support in tests with remoulded and consolidated samples.

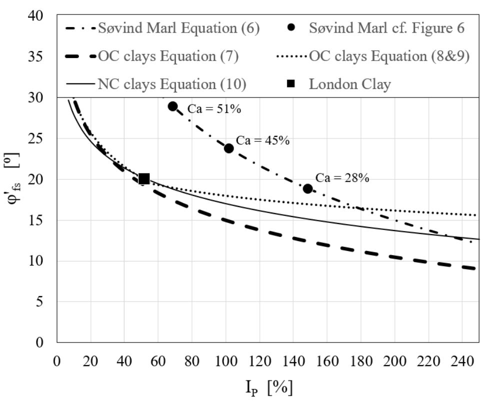

Figure 5. Fully softened effective secant friction angle versus plasticity index.

As can be seen from the figure, the above-mentioned lower bound envelopes for the fully softened effective strength parameters cf. Equation (7) and Equation (8&9) as well as the lower bound for normal consolidated clays cf. equation (10) predict a fissure strength that agrees well with the derived fissure strength for London Clay. It also appear that the fully softened strength of Søvind Marl are significantly larger than any of the lower bound equations predicts and that it converges towards Equation (7) as the calcite content decreases.

This emphasizes that it is very important to distinguish between clay types when examining the relationship between effective strength and plasticity. Accordingly, the lower bound defined by Equation (8&9) is problably on the uncertain side for many Paleogene clays, as it is partly based on results of tests with the more competent Søvind Marl. Sørensen (2023) suggest that the rather competent geotechnical properties of Søvind Marl is a result of the presence of silt-sized coccoliths in the clay matrix.

Figure 5 also shows the lower bound envelope for the effective strength of normally consolidated clays. As expected, it is rather close to the two lower bound envelopes for the fully softened effective friction angle for overconsolidated clays.

10 PREDICTIONS OF THE MATRIX STRENGTH

The matrix strength of the Søvind Marl is tricky to derive, as many of the test results are more or less affected by fissures.

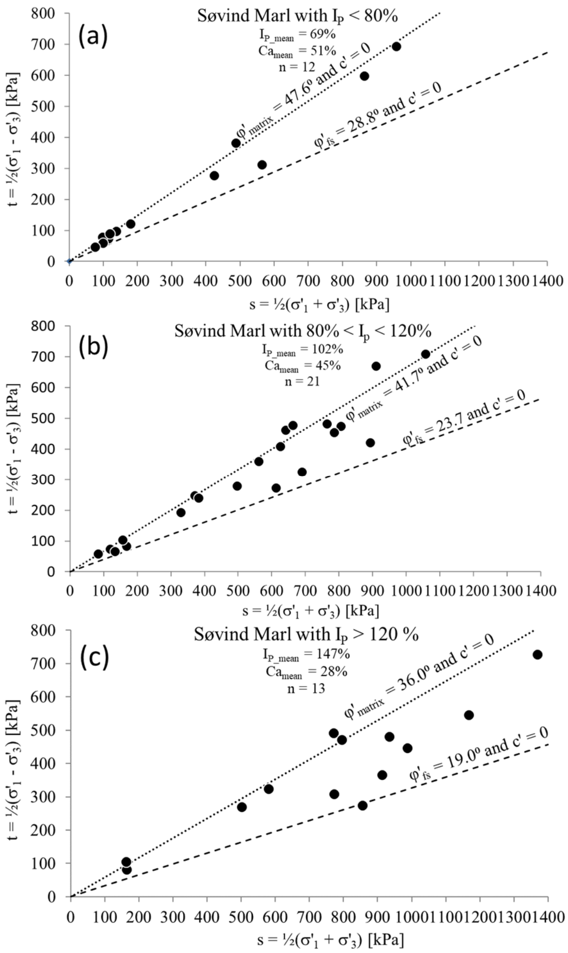

Figure 6. The matrix strength φ'_{matrix} and the fully softened strength φ'_{fs} of Søvind Marl is calculated using Equation (6) and (11) respectively and the mean plasticity, $I_{P,mean}$.

The matrix strength apparently depends on plasticity in the same way as the mean strength does cf. Figure 2d and Equation (2). Assessed from Figure 2a-c, the matrix strength φ'_{sec_matrix} is approx. five degrees larger than the mean strength described with Equation (2):

$$\varphi'_{sec\_matrix} \approx 112^o - 35^o \cdot log_{10}(I_P(\%)) \quad (11)$$

Consequently, the matrix strength of Søvind Marl is between 1.6 and 2.3 times the fully softened strength increasing with plasticity ($50 < I_P < 250\%$).

11 CONCLUSIONS

To get a complete picture of the Søvind Marls effective strength properties, the following four strengths must be addressed:

- The effective shear matrix strength cf. Equation (11), which is the strength without any influence of fissures.
- The mean effective shear strength cf. Equation (2), which include influence of fissures.
- The fully softened effective shear strength cf. Equation (6), which roughly corresponds to the fissure strength at first time failure.
- The effective residual strength, which is the minimum mobilized shear strength after very large straining.

According to the derived equations, the first three strengths are inversely proportional to plasticity and with no effective cohesion intercept.

This study illustrate that it is a very uncertain procedure to try to determine the fully softened shear strength solely from tests with intact samples, regardless of test procedure and sample size.

A safe lower value for the fully softened effective shear strength parameters relevant to first-time slides can for Danish Paleogene clays be estimated from Equation (7): $\varphi'_{fs} \approx 45^o - 15^o \cdot \log_{10}(I_P(\%))$ and $c'_{fs} = 0$. However, for Søvind Marl, this equation is too conservative for practical purposes. This study confirms the suspicion that a content of coccoliths improves the effective shear strengths. Consequently, it is important to distinguish between clay types and calcite content when examining the relationship between effective strength and plasticity.

As expected, the mean strength envelope for intact samples of Søvind Marl is best described with a power function passing through origin, i.e. without an effective cohesion intercept. This confirms, that the effective cohesion is thus to be regarded as a mathematical parameter that is introduced in order to get the best possible match between the Mohr-Coulomb failure criteria and the real strength envelope.

This study shows that not only does calcite content and plasticity fluctuate strongly, so does the effective strength. The effect that plasticity has on the fully softened strength is of the same order of magnitude as the effect that strain softening has on the effective matrix strength

Despite the fact that the triaxial tests included in this study were performed in different ways (drained/undrained) and with different sizes of samples, the results produce reasonably uniform mean trend lines when the strength is plotted in a t-s diagram. This indicates that the significance of the above-mentioned differences is subordinate to natural strength variations of the clay types due to structure, plasticity and calcite content. Accordingly, it is thus not decisive for most commercial purposes how the tests are carried out within the given framework. However, the results point to the importance of knowing the pore pressure in the slip surface when the effective strength is to be derived from undrained tests.

Since the effective shear strength is greatly influenced by both plasticity and the calcite content (or maybe rather the coccolith content), it is highly recommended to consistently measure these parameters. In this regard, it is crucial to carry out the classification tests on the sample material itself and/or the trimming material.

12 ACKNOWLEDGEMENT

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