Correlation between drained shear strength and plasticity index of undisturbed overconsolidated clays

Corrélation entre la résistance au cisaillement des sols drainés et l'indice de plasticité des argiles surconsolidés non perturbées

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ABSTRACT: A number of triaxial compression tests have been performed by The Danish Geotechnical Institute over the past decades on undisturbed overconsolidated Danish clays; ranging from clay till of low plasticity to extremely high plasticity marine Tertiary clays. The test results confirm that the drained peak angle of shearing resistance can be related to the plasticity index, though a large scatter is generally seen. Based on the results and a review of published data a conservative relationship between drained peak angle of shearing resistance and plasticity index for undisturbed overconsolidated clays is proposed. The proposed relationship and the interrelation to the effective cohesion are discussed.

RÉSUMÉ: De nombreux essais de compression triaxiale ont été effectués par l'Institut danois de géotechnique au cours des dernières décennies sur les argiles danoises surconsolidées non perturbées, allant des argiles morainiques ayant une plasticité faible à des argiles marines tertiaires ayant une plasticité extrêmement élevée. Les résultats des essais confirment que l'angle de résistance maximale au cisaillement des sols drainés peut être lié à l'indice de plasticité si une forte dispersion est généralement observée. Sur la base de ces résultats et à partir d'un ensemble de données déjà publiées, une relation conservatrice entre l'angle de résistance maximale au cisaillement des sols drainés et l'indice de plasticité pour les argiles surconsolidées non perturbées est proposée. La relation proposée et les liens à une cohésion effective sont discutés.

KEYWORDS: Plasticity index, Drained shear strength, Laboratory testing, Overconsolidated clay

1 INTRODUCTION

Empirical correlations are widely used in geotechnical engineering practice as a tool to estimate the engineering properties of soils. Useful correlations exist between the index properties obtained from simple routine testing and the strength and deformations properties of cohesive soils among others. For practical purposes the results of routine index tests and correlations can be used as a first approximation of the soil parameters for use in preliminary design of geotechnical structures, and later as a mean to validate the results of laboratory tests. Results from several index tests obtained for a given site can be used to assess the variation in the properties of the soil mass.

This study is aiming to provide a conservative correlation between the effective peak angle of shearing resistance and plasticity index I_P for natural undisturbed overconsolidated Danish clays based on the results from a large database of triaxial compression tests performed by The Danish Geotechnical Institute (GEO) over the past decades.

1.1 *Stress-strain behavior and effective strength of overconsolidated clays*

In contrast to normally consolidated (NC) clays highly overconsolidated (OC) natural clays typically show a distinct strain softening behavior in drained triaxial compression, which can be related to the breakdown of interparticle bonding and the dilatant behavior of the compact clay structure (Burland 1990). With further shearing in the post peak region OC clays (as well as NC clays) may experience an additional reduction in strength due to the alignment of the platy clay particles on the failure plane (residual state). The stress-strain behavior of OC clays compared to NC clays is illustrated in Figure 1a.

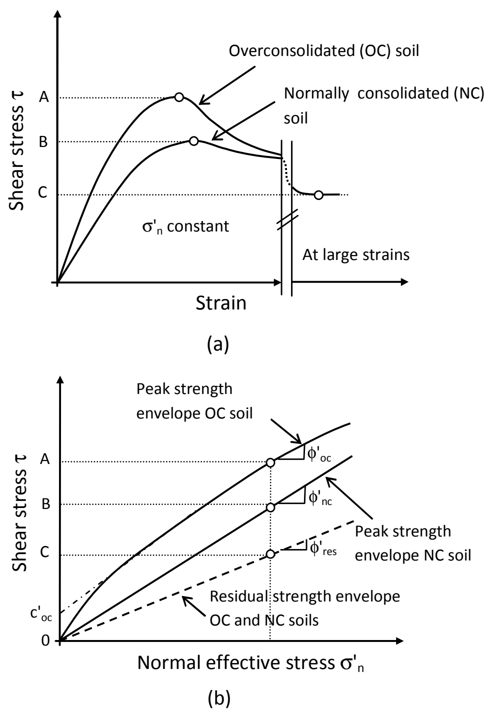

Figure 1. Peak and residual shear strength for normally consolidated and overconsolidated soils (a) Typical stress-strain curves (b) Failure envelopes showing the associated angle of shearing resistance, ϕ'

Stiff fissured overconsolidated clays may fail along preexisting fissures in which case the strength is governed by the fissure strength. Skempton (1957) found the fissure strength to correspond to the fully softened strength i.e. shear strength of the reconstituted normally consolidated soil, which is less than the peak strength of the OC clay, but greater than the residual strength.

It is generally accepted that the effective strength of uncemented saturated clays is frictional and the strength envelope is nonlinear. Hence the strength envelope will pass through origin, and so the true cohesive intercept c'=0kPa (Burland 1990). However, over the typical range of stress levels met in practice (~50-400kPa) the effective peak strength for NC and OC clays can be approximated by a linear relationship between effective normal stress at failure σ'_f and shear strength τ_f using the Mohr-Coulomb strength equation:

$$\tau_f = \sigma'_f \cdot tan\ (\phi') + c' \qquad (1)$$

where ϕ' and c' respectively are the tangent drained angle of shearing resistance and the apparent cohesive intercept, as illustrated in Figure 1b. For OC clays c'_{oc}>0kPa and for NC clays c'_{nc}≈0kPa. While the angles of shearing resistance for OC and NC clays are typically found not to differ much.

Generally the frictional resistance to shearing, as expressed by ϕ', can be expected to decrease as the content of platy clay minerals increase in the soil mass. With increasing content of platy clay particles both the liquid limit w_L and the plasticity index I_P will increase, and hence a correlation between ϕ' and w_L or I_P can be expected.

1.2 *Existing relationships between effective shear strength and plasticity index*

Several studies have been reported in the literature with regards to the correlation between the effective angle of shearing resistance ϕ'_{peak} and plasticity index I_P (Brooker and Ireland 1965, Ladd et al. 1977, Stark and Eid 1997, Terzaghi et al. 1996 among others). These studies are however mainly focused on normally consolidated reconstituted or undisturbed natural clays, while only little has been reported for overconsolidated undisturbed clays.

Figure 2 shows collected data from the literature in a plot of ϕ'_{nc} vs. I_P (single log plot) for primarily NC clays (I_p range 5-240%). ϕ'_{nc} generally represent a peak secant value with the assumption that c'_{nc} is zero. A very significant scattering of the data points is seen, e.g. at I_P=20% the value of ϕ'_{nc} is found to vary between approximately 25deg. and 35deg. However, despite the significant scatter a trend of reducing ϕ'_{nc} with increasing I_P is seen, and the data furthermore suggest the existence of a lower bound value for ϕ'_{nc} at given value of I_P.

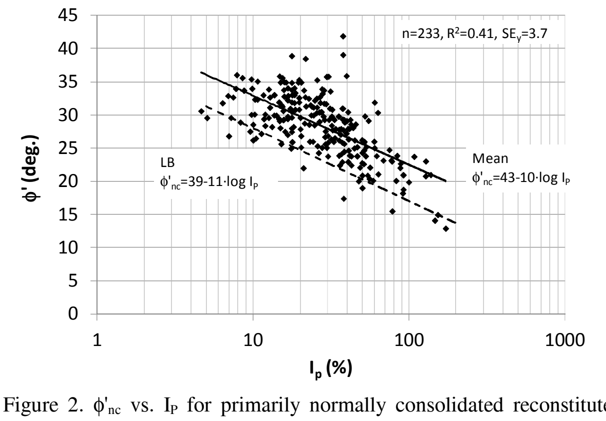

Figure 2. ϕ'_{nc} vs. I_P for primarily normally consolidated reconstituted and undisturbed clays after Ladd et al. 1977 (with data from Kenney 1959 and Bjerrum and Simons 1960), Terzaghi et al. 1996 and Brooker and Ireland 1965.

The shaded area in Figure 2 represents the range of results reported by Stark and Eid 1997 from a large series of ring shear tests on 24 different reconstituted normally consolidated natural soils (I_P=8-244%, Clay-size fraction CF=10-88%, normal effective stress σ'_n=50-400). Based on the data, relationships between ϕ'_{nc} and I_P were proposed which were dependent on clay-size fraction and normal effective stress, as seen in Figure 3. By taking account of clay-size fraction and stress level Stark and Eid showed a significantly reduced scatter around the mean trend lines. A downward shift in the trend lines were observed with increasing stress levels and increasing clay-size fraction. The findings by Stark and Eid suggests that the observed scatter in the reported data found in the literature, as shown in Figure 2, to a large extent can be explained by variations in stress level due to a non-linear strength envelope and additionally clay-size fraction, as both soil mineralogy and clay-size fraction are not accounted for solely by the variation in the index properties.

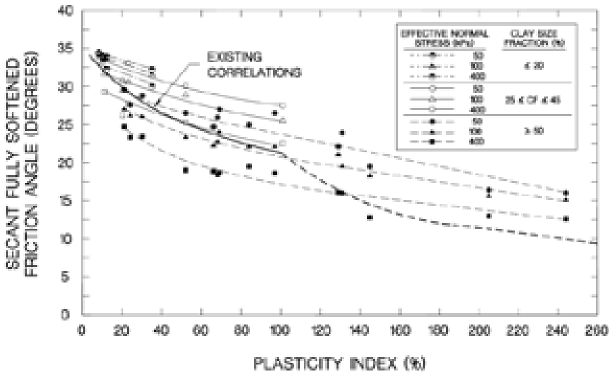

Figure 3. ϕ'_{nc} vs. I_P for reconstituted normally consolidated soils as a function of clay-size fraction and normal effective stress (Stark and Eid 1997)

Based on the literature data a cautious lower bound (LB) estimate of the relationship between ϕ'_{nc} and I_P for NC clays can be derived together with a best estimate from the best-fit regression line through the data, as indicated in Figure 2 and given below.

Cautious LB estimate: $\phi'_{nc} = 39\text{-}11 \cdot log\ I_P$ (deg.) (2)

Best estimate: $\phi'_{nc} = 43\text{-}10 \cdot log\ I_P$ (deg.) (3)

The lower bound estimate, which correspond roughly to the 5% fractile, also approximately match the lower bound of the range of results reported by Stark and Eid for clay-size fractions above 50% and a stress level of 400kPa. Hence for clay-size fractions below 50% and stress levels below 400kPa the effective angle of shearing resistance ϕ'_{nc} can be expected to be significantly greater than estimated from eq. 2 (up to approximately 12deg. for CF<20% and σ'_n=50kPa, as seen from Figure 3).

2 SOIL DESCRIPTION AND TEST PROCEDURES

A number of triaxial compression tests have been performed by GEO on various undisturbed overconsolidated clays over the past decades. Test data have been collected from older tests (> 30 years) and more recent test series as listed in Table 1.

2.1 *Soil description*

The tested soils range from very low plasticity clay tills to extremely high plasticity Eocene clays. The recent tests include a test series in connection to the 1992 Great Belt bridge (GB) ground investigation, which provides a significant amount of test data for very low plasticity clay till. While the newly completed 2011 Fehmarnbelt (Fixed Link) (FB) ground investigation contribute significantly to the understanding of the strength behavior of very high to extremely high plasticity Eocene and Paleocene marine clays from the Røsnes, Ølst and Holmehus clay formations. The majority of the investigated clays from the Fehmarnbelt (Fixed Link) ground investigation have been assessed to be situated within glacial folded strata. A series of recent tests in connection to a ground investigation at Esbjerg Habour (EB) highlights the strength properties of Mica clay – a Miocene marine clay of high plasticity.

The additional old test data covers glacial clay till, a few glacial/late glacial meltwater and late glacial marine clays and furthermore a wide range of Tertiary (Palaeogene) marine clays of late Miocene age to Palaeocen age: Mica clay, Septarian clay, Søvind marl, Lillebælt clay, Røsnes clay, Ølst clay, Tarras clay and Holmehus clay.

The majority of the very high plasticity Palaeogene clays from the listed test series are found to be fissured in nature.

Figure 4 shows the outline of the index properties of the tested clays in Casagrandes classification chart. The classification parameters have generally been determined in accordance with BS 1377:Part 2:1990 using the Casagrande method to determine the liquid limit. The data points for the different soils generally fall close around straight lines and above the A-line corresponding to clays of very low ($4\%<I_P<7\%$) to extremely high plasticity ($I_P>100\%$).

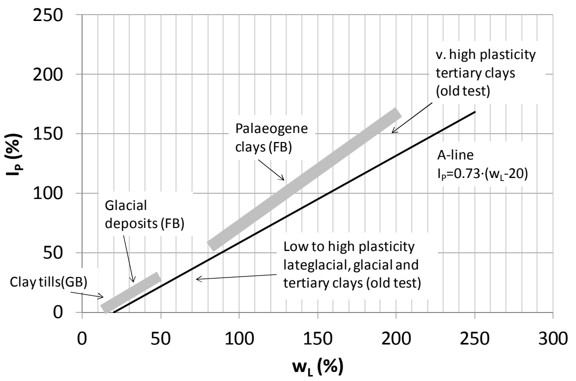

Figure 4. Outline of index properties of the tested clays shown in Casagrandes classification chart I_P vs. w_L

The classification parameters incl. clay-size fraction (CF) and natural water content w_n for the tested clays are also listed in Table 1 alongside test types and number of tests (n). For the consistent test series (GB, FB and EB) the mean values of the classification parameters are listed and the standard deviations are shown in brackets. A more detailed description of the Palaeogene clays incl. mineralogy is given by Fehmarnbelt (Fixed Link) 2011.

2.2 *Test procedures*

The recent triaxial compression tests have generally been performed on nominally undisturbed specimens with a diameter of approximately 70mm and a height to diameter ratio of 1. Smooth end platens have been used. Samples have been extracted by means of a push-in Shelby-tube sampler (called A-tube in Denmark) with an inner diameter of 70mm.

Samples are saturated using backpressure and typically preloaded to reduce the effects of possible sample disturbance and bedding effects. The preloading are in most cases performed under anisotropic K_0 conditions. After the preloading and unloading of the sample, the specimen is sheared in either drained or undrained compression to failure. Multiple tests are carried out, consisting of preloading, unloading to a new and higher stress level followed by a drained or undrained compression test (these steps are repeated 2 or more times).

The applied rate of straining during shearing is chosen according to the soil type to ensure slow enough rates to achieve full equalization of pore water pressures in the specimen. For the high plasticity clays this means that the applied strain rate is around 0.02-0.05%/hr, while a strain rate of approximately 0.1-0.7%/hr has been used for specimens of clay till.

For specimens of very high plasticity clay care is generally taken during all test stages to avoid that the effective stress reduces below the in-situ stresses to prevent swelling, which may lead to destructuration of the micro-structure (Leroueil and Vaughan 1990).

Older triaxial compression tests have in contrast typically been performed with a height to diameter ratio of 2:1 and a diameter of approximately 36 mm. Samples were usually extracted using a 42mm 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 the pore water pressure kept equal to zero. Hence, undrained compression was achieved by adjusting the cell pressure during testing to obtain constant volume displacement.

The applied rate of straining during the compression stage was generally higher than what would be recommended today to ensure full equalization of pore water pressures within the samples of especially high plasticity clays. Hence, the actual effective stresses at the failure state are somewhat uncertain because the pore water pressure is unknown. Nevertheless, the old triaxial tests constitute a very comprehensive database of strength parameters for low to high plasticity clays, which can be compared to the recent and presumably more reliable tests results.

Table 1. Overview of classification parameters and larger series of triaxial compression tests on undisturbed overconsolidated clays performed by GEO.

Project	Soil type	$w_n^{\#}$ [%]	$w_L^{\#}$ [%]	$I_P^{\#}$ [%]	$CF^{\#}$ [%]	Test types	*n*
GB (1992)	Clay till	11 (2)	16 (2)	6 (2)	-	$MCAU_{u=0}$ and CAU	45
FB (2011)	Upper till	9 (-)	19 (1)	7 (1)	26 (9)	CAU and CAD	5
	Chalk till	9 (-)	20 (1)	6 (0)	23 (-)	CAU and CAD	5
	Lower till	13 (5)	28 (4)	16 (3)	24 (3)	CAU and CAD	8
	Røsnes	35 (4)	147 (28)	117 (27)	70 (7)	CAU and CAD	48
	Ølst	45 (11)	140 (25)	106 (23)	51 (15)	CAU and CAD	5
	Holmehus	42 (6)	133 (23)	98 (25)	61 (7)	CAU and CAD	5
	Palaeogene clays	37 (6)	145 (27)	114 (27)	67 (9)	CAU and CAD	58
EB (2012)	Mica clays	28 (3)	58 (9)	36 (8)	-	MCAU, MCAD, MCIU	10
Other recent test	Tertiary clays	-	-	19-85	-	MCAD CAU CU	8
GEO old test (> 30 yrs)	(Late) Glacial & Tertiary clays	-	-	5-151	-	$CAU_{u=0}$ $CAD_{u=0}$ $CU_{u=0}$ $CD_{u=0}$	108

\# mean values with the standard deviation shown in brackets

An overview of the number of tests (n) and test types are given in Table 1. The following abbreviations are used:

- CAU/CAD Anisotropically (K_0) Consolidated. Undrained/Drained compression.
- MCAU/MCAD Anisotropically (K_0) Consolidated. Undrained/Drained compression. Multiple test on the sample.
- CU/CD Isotropically Consolidated Undrained /Drained compression.
- u=0 denominates older test procedures with no backpressure and pore water pressure kept at zero kPa.

The drained strength parameters; angle of shearing resistance ϕ' and effective cohesion c' can be derived from the results of two or more compression tests (either using multiple testing on the same sample or sets of two or more compression tests on samples with similar properties). Alternatively, the strength parameters can be interpreted from the undrained compression effective stress path, since the effective stress path for overconsolidated clays will tend to climb the strength envelope as the soil dilates and the pore water pressures decrease.

Generally, test interpretation may be difficult in cases where the specimen experiences destructuration during testing or if the sample is fissured.

3 DRAINED SHEAR STRENGTH AND PLASTICITY INDEX

3.1 *Drained peak angle of shearing resistance*

Figure 5 shows the relationship between the drained peak angle of shearing resistance ϕ'_{oc} and the plasticity index I_P (single log plot) as derived from triaxial compression tests performed by GEO on the various overconsolidated undisturbed clays shown in Table 1.

ϕ'_{oc} has generally been derived as a tangent value, to minimize the otherwise high influence of stress level resulting from the initially curved failure envelope. Hence, both values of ϕ'_{oc} and c'_{oc} are obtained from the tests. Results from the older tests (> 30 years, open points) are shown separately from the more recent tests (closed points).

The shaded area and dashed lines respectively represents results of the series of triaxial compression tests on palaeogene clays and glacial till deposits (Lower and Upper till) performed in connection to the Fehmarnbelt (Fixed Link) 2011 ground investigation. The spans shown in the I_P and ϕ'_{oc} values represent mean values ± 1 standard deviation.

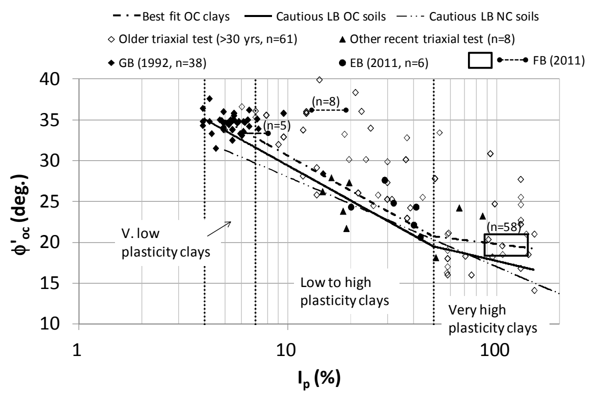

Figure 5. Relationship between peak angle of shearing resistance ϕ'_{oc} and plasticity index I_P for overconsolidated undisturbed clays

A large scatter in the data and a trend of reducing ϕ'_{oc} with increasing I_P is generally seen similarly to what was observed for NC clays in figure 2.

It is observed that the old triaxial test results generally show a much larger spread than the results of the presumably more reliable recent triaxial tests. Furthermore, the new test data tend to be found in the lower half of the range of results from the old tests. This indicates that ϕ'_{oc} is likely to be overestimated by the old tests. The use of high strain rates and the lack of full saturation in the old undrained $CU_{u=0}$ triaxial compression tests are likely to have lead to some uncertainties in the actual effective stress state at failure. This may explain some of the observed differences between the results from the old and more recent triaxial compression tests.

As for NC clays some of the scatter may arise as a result of variations in clay-size fraction, since this is not accounted for solely by the variations in I_P. Variations in stress level on the other hand should have less of an influence, since a tangent value of ϕ'_{oc} is derived from a failure envelope which is approximately linear within the typical stress range (100-600kPa), as seen in Figures 8-10. As mentioned previously, difficulties in test interpretation and influence of fissures and destructuration may have influenced some of the results.

Based on previous experience and much of the old data shown in Figure 5 within the I_P range of 6%-106%, GEO (Knudsen 1981) has since the early 80's suggested using the following correlation to get a cautious lower bound estimate of ϕ'_{oc} on the basis of a mean value of I_P:

$$\phi'_{oc} = 45\text{-}15 \cdot \log I_P \text{ (deg.)} \quad (4)$$

In the light of the more recent test data, which extends the I_P range, especially in the high I_P end, to 4%-151%, it is suggested that the lower bound values of ϕ'_{oc} should be slightly less than previously predicted by eq. 3 for very low I_P clays and somewhat higher for high plasticity clays (I_P>50%). Hence, it is suggested to use the following revised cautious lower bound (LB) estimate of the relationship between ϕ'_{oc} and the plasticity index I_P as given by eq. 5 and 6, and shown in Figure 5 (solid line):

Cautious LB estimate:

$4<I_P<50$	$\phi'_{oc} = 44\text{-}14 \cdot \log I_P$ (deg.)	(5)
$50 \le I_P<150$	$\phi'_{oc} = 30\text{-}6 \cdot \log I_P$ (deg.)	(6)

For an I_P value of 100%, which is typical for e.g. Røsnes clay, this means that the lower bound estimate increases from a peak value of 15 deg. using eq. 4 to 18 deg. using eq. 6.

The best estimate given by the best-fit regression line through the recent test data is shown in Figure 5 (chain dotted line) and is given by:

Best estimate:

$4<I_P<50$	$\phi'_{oc} = 45\text{-}14 \cdot \log I_P$ (deg.)	(7)
$50 \le I_P<150$	$\phi'_{oc} = 26\text{-}3 \cdot \log I_P$ (deg.)	(8)

Eq. 5.-8. are believed to be applicable to most overconsolidated natural clays with clay-size fractions below 80%. For soils with clay-size fractions higher than 80% the above relationships should be used with caution until its validity is confirmed by additional tests.

It should be noted that some cases may dictate a mobilized angle of shearing resistance which is lower than the above estimated peak values, e.g. when progressive failure is considered in connection to slope stability analysis in high plasticity clays (Skempton 1977, Burland 1990).

The lower bound estimate for NC clays is shown in Figure 5 (dash-double-dot line) for comparison. It is observed that the lower bound estimates for NC and OC clays do not deviate much.

3.2 *Effective cohesion*

Figure 6 shows the relationship between the cohesive intercept of the strength envelope c'_{oc} and the plasticity index I_P (single log plot). Data from recent tests and older tests (>30 years) are separated. Two sets of c'_{oc} values have been plotted: the derived values from the tests and estimated values of c'_{oc}. The derived values have been interpreted from the tests results and are paired with the ϕ'_{oc} values shown in Figure 5. While the estimated value of c'_{oc} is found from each failure point (σ'_f, τ_f) by subtracting the stress dependent "frictional" strength contribution $\sigma'_f \cdot \tan(\phi'_{oc})$ from the shear strength τ_f. This can also be expressed in terms of mean effective stress $s'_f=½(\sigma'_1+\sigma'_3)$ and shear stress $t_f=½(\sigma'_1- \sigma'_3)$ from the following equation.

$$c'_{oc,est} = \frac{t_f - s'_f \cdot \sin \phi'_{oc,est}}{\cos \phi'_{oc,est}} \tag{9}$$

where $\phi'_{oc,est}$ is the the lower bound estimate of the angle of shearing resistance, as given by eq. 5. and 6. on the basis of I_P. The estimated c'_{oc} may show negative values in cases where the estimation of ϕ'_{oc} is too high.

The intercept of the strength envelope is very sensitive to the interpretation of the test results, and factors like destructuration or influence of fissures may have great impact on the failure points and hence the value of c'_{oc}. The scatter in the data points will therefore be very significant, much more so than what was observed for ϕ'_{oc} in figures 2 and 5. Despite the scatter a weak trend of reducing lower bound value of c'_{oc} with increasing I_P is seen for I_P greater than 7%. Part of the data from the old tests has been excluded since it is not certain if the meltwater and late glacial marine deposits are heavily overconsolidated (overconsolidation ratio OCR>4).

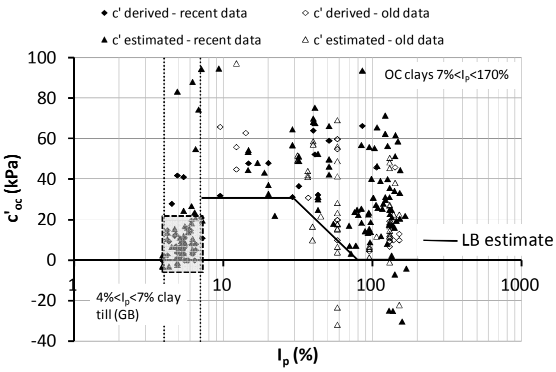

Figure 6. Relationship between effective cohesion c'_{oc} and plasticity index I_P for overconsolidated undisturbed clays

The test results from the Great Belt bridge ground investigation (GB) on very low I_P ($4\%<I_P<7\%$) clay till are seen to deviate significantly from the general trend by showing relatively low c'_{oc} values; typically between 0 and 15kPa. This is true for both the derived and estimated values of c'_{oc}. One or several factors might explain this, among others: a) Some of the tested clays are possibly ablation tills that are deposited in a nearly normally consolidated state. b) The clay till may have been subject to postdepositional geological processes (e.g. subsequent passing of ice sheets, meltwater, heave etc.), which may have disturbed the structure and erased the effects of overconsolidation. c) Generally, the tested clay tills have a high content of silt, sand and gravel, and the soils can therefore be characterized as transitional soils, which in terms of strength may behave more like a frictional soil ($c'\approx 0$kPa) than clay. This is further supported by the fact that the I_P value is likely to be overestimated, as particles larger than 0.425mm have been removed prior to classification in accordance with BS1377:part 2:1990. An estimated 20-40% of the soil particles have been removed. Hence sand till or gravel till may in most cases be a more representative classification for the tested till deposits.

The low c'_{oc} values observed at very high I_P values above 60% are likely to be a result of fissures and/or sample destructuration. Destructuration may result from large straining events during sampling, sample preparation, testing or geological history. The Palaeogene clays from the Fehmarnbelt (Fixed Link) project mostly belong to an upper stratum which is folded by glacier. Hence, the measured strengths are likely to be influenced by varying degrees of destructuration caused by glacial disturbance of the otherwise intact clay layers.

The significant variation in the c'_{oc} values seen in Figures 6 and 7 also seem to indicate this. For heavily overconsolidated clays with $7\%<I_P<80\%$ the test data indicate a cautious lower bound estimate of the relationship between c'_{oc} and the plasticity index, I_P as shown on Figure 6 (solid line) and given by the following equations depending on the value of I_p:

Cautious lower bound estimate:

$7\%<I_P<30\%$	$c'_{oc} = 30$	(kPa)	(10)
$30\%\leq I_P<80\%$	$c'_{oc} = 48-0.6\cdot I_P$	(kPa)	(11)
$I_P>80\%$	$c'_{oc} = 0$	(kPa)	(12)

While the drained angle of shearing resistance ϕ'_{oc} is more naturally linked to soil mineralogy composition, as expressed partly by the I_P value, the apparent effective cohesion is more naturally linked to the soil structure and dilative tendencies. As the I_P value is determined from reconstituted state it does not take account of soil structure. Hence, the above relationship between c'_{oc} and I_P may not be the most appropriate to use.

As suggested in the previous Danish code of practice for foundations (Danish Standard DS 415) it may be expected that the value of c'_{oc} is better related to the undrained shear strength c_u rather than I_P. Both c_u and c'_{oc} are influenced by soil structure and dilation, but as the stress level is likely to have a greater impact on c_u than c'_{oc}, the relationship will not be unique. Based on a comparison of the drained and undrained bearing capacity in connection to plate loading tests on clay till (Jacobsen 1970), the previous Danish code of practice for foundations suggests the following cautious estimate of c'_{oc} on the basis of c_u:

$$c'_{oc} = 0.1\cdot c_u \quad \text{(kPa)} \tag{13}$$

Figure 7 shows the relationship between c'_{oc} and c_u based on data from the performed tests. As before both derived values from the tests and estimated values of c'_{oc} are shown. As expected the observed scatter is very significant, but there is a tendency of increasing values of c'_{oc} with increasing values of c_u. Both recent and older data appear to verify that the relationship between c'_{oc} and c_u given by eq. 13 can be used as a cautious lower bound estimate with an upper limit of c'_{oc} =30kPa for all heavily overconsolidated clays except very low I_P clay till. For very low I_P clay till the effective cohesion c'_{oc} is in the majority of cases found to be lower than given by eq. 13, as shown by the shaded area in Figure 7. Hence, in agreement with observations from Figure 6 it is suggested to use c'_{oc}=0kPa for very low I_P ($4\%<I_P<7\%$) clay till/transitional soils independently of c_u, unless specific triaxial test data is available to suggests otherwise. c'_{oc}=0kPa should also be assumed for fissured high plasticity clays in cases where the overall mobilized strength may be governed by the fissure strength.

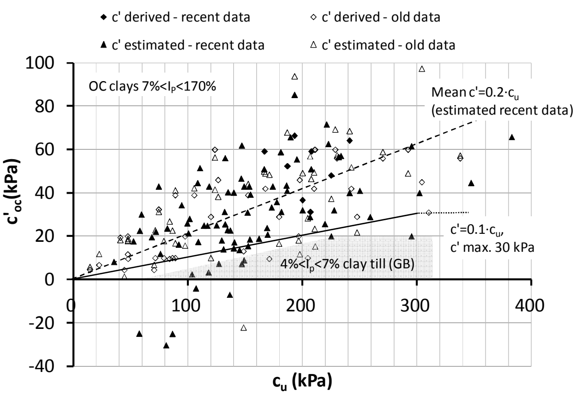

Figure 7. Relationship between effective cohesion c'_{oc} and undrained shear strength c_u for overconsolidated undisturbed clays

Generally it is suggested to estimate c'_{oc} on the basis of eq. 13 for clays with I_P values between 7% and 150%. It should however be noted that if both ϕ'_{oc} and c'_{oc} are estimated cautiously using the above correlations then the estimated combined strength may in some cases be too conservative. On the other hand, the combined strength found from the proposed correlations may overestimate the strength at low effective stresses (<50 kPa) as the failure envelope curves towards origin (see Figure 1b).

4 VALIDATION OF THE PROPOSED RELATIONSHIPS

The test results from the three test series from the Great Belt (GB), Fehmarnbelt (Fixed Link) (FB) and Esbjerg Habour (EB) ground investigations are shown in Figures 8-10. The figures show the failure points for each shear stage for all the tested specimens in a plot of shear stress t against mean effective stress s'. For each test series the best-fit line through the data points is shown, and for comparison the cautiously estimated effective strength envelope is also shown (dashed line). The estimated strength envelope is based on the estimated angle of shearing resistance ϕ'_{oc} found from eq. 5-6 on the basis of the mean value of I_p for the tested clays, and the effective cohesion c'_{oc} determined from eq. 13. c_u has been assumed equal to the mean shear stress t_{fm} at failure, as derived from the best-fit regression line through the failure points.

It is observed that the estimated strength envelope in all cases provides a cautious estimate of the strength of the tested clays within the given stress intervals.

5 CONCLUSION

Simple correlations between the plasticity index and the drained peak strength parameters in terms of ϕ'_{oc} and c'_{oc} have been proposed on the basis of a comprehensive database of triaxial compression tests on undisturbed overconsolidated Danish clays of very low to extremely high plasticity. The proposed correlations gives cautious lower bound values of the drained strength parameters, which can be used as a first approximation for use in preliminary design of geotechnical structures. Furthermore, the correlations can be used to evaluate the results of laboratory effective strength tests, and as a mean to assess how well these results represent the entire soil mass at a given site when viewed in connection to the variations of the index properties in the soil mass.

The authors however believe that the proposed correlations should only be used in cases where time and cost constraints do not allow for actual effective strength tests to be carried out. In most other cases the use of effective strength tests will provide a much more reliable and cost effective estimate of the strength properties of the soil in question.

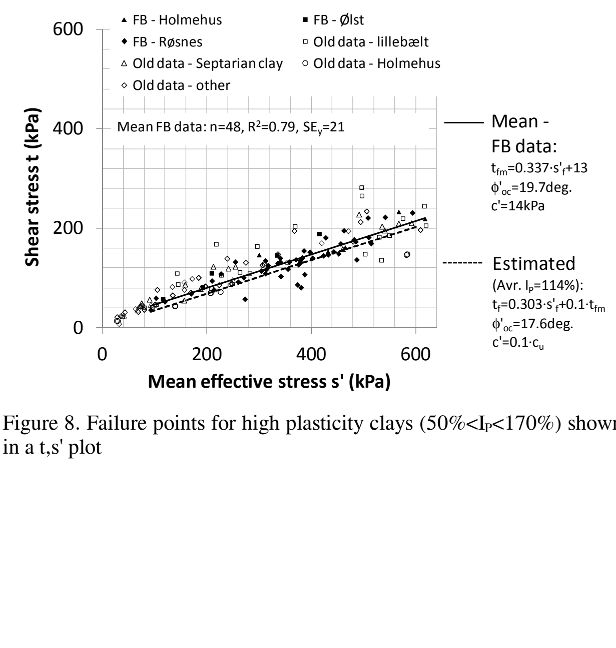

Figure 8. Failure points for high plasticity clays (50%<I_P<170%) shown in a t,s' plot

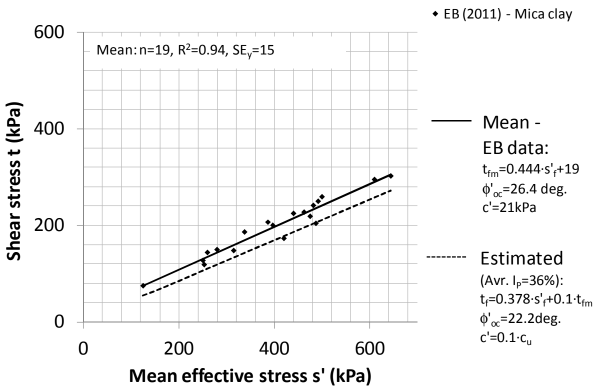

Figure 9. Failure points for mica clay (20<I_P<43%) shown in a t,s' plot

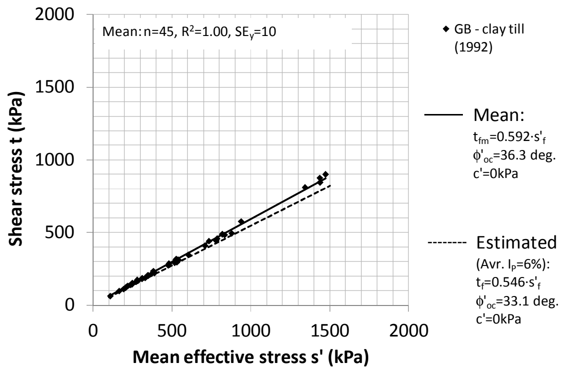

Figure 10. Failure points for clay till (4<I_P<7%) shown in a t,s' plot

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