Triaxial testing of overconsolidated, low plasticity clay till

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ABSTRACT
In northern Europe, clay till (Boulder Clay) is very common. It is characterised by high overconsolidation ratios and low plasticity (7% to 10%). The undrained shear strength vary significantly, mostly between 50 kPa and 1500 kPa. The natural water content and the clay content are below 25% and 20%, respectively. Due to its origin the soil is well-graded and it can contain particles up to boulder size.
In accordance with Danish tradition, the clay till is tested in the triaxial apparatus employing a height equal to the diameter and smooth pressure heads. This is in contrast to standard international practice where a height-diameter ratio of two and rough pressure heads are employed. Supported by test results from a test campaign the paper discusses the impact on the stress-strain behaviour and the strength from the height-diameter ratio and the consolidation procedures, respectively.

Keywords: Triaxial testing, undrained strength parameters, clay till, consolidation procedures in triaxial testing, specimen size, height-diameter ratio.

1 INTRODUCTION

Triaxial testing is considered by far the most reliable tool for assessment of the shear strength and stiffness of clay till in the laboratory. The tests serve to provide a link (calibration) to available in situ testing methods, SPT, CPTU, plate loading tests etc. However, the shape and size of the samples and the stress path show very significant differences inbetween laboratories across the world.

In the crossfires between local traditions, research efforts and commercial interests, different systems have been developed and refined (automated) by the advent of faster and cheaper computer systems.

The Danish clay tills may exhibit in situ undrained shear strength values in excess of 1500 kPa. Thus, it was early on recognised that testing in standard triaxial equipment may produce erroneous results as the defor mations in the apparatus itself were far from negligible for the stiff to very hard clay till samples. As a result, Jacobsen (1970) developed a very rugged type of triaxial apparatus. Apart from reducing the apparatus deformations to insignificant values the new triaxial apparatus introduced a height/diameter ratio H/D = 1, a minimum sample diameter D = 70 mm and smooth pressure heads. This has subsequently formed the basis for both commercial and research triaxial testing in Denmark.

Working on international projects or projects in Denmark with international participation, the Danish triaxial testing tradition is constantly challenged. The main reason for this is that the standard (commercially available) triaxial set-ups use H/D = 2 and rough pressure heads in accordance with international standards (ASTM, AASHTO etc.).

This type of apparatus is available worldwide and often allow for testing on small diameter samples down to 33 mm.

The preparation and execution of tests in the "traditional type apparatus" is simpler, cheaper and less demanding in terms of technician skills. Thus, it is not surprising that only a small percentage of tests world-wide are carried out using what (in Denmark) is considered the superior type of apparatus.

Due to unavailability of the Danish type triaxial set-up by the successful laboratory contractor on a major Danish bridge project, a campaign of triaxial testing using the traditional and the Danish set-ups was initiated.

The purpose of the campaign was to elucidate the impact on the results from different height diameter ratios (*H/D* = 1 or *H/D* = 2), the consolidation stress path before the undrained failure phase and the specimen diameter (*D* = 70 mm and *D* = 100 mm).

The impact on the test results from the above differences is presented and discussed in the paper.

2 LITERATURE REVIEW

During the development of triaxial testing set-ups and procedures some of the main controversies (for undrained testing) are related to:

- Sample size (diameter and height/diameter ratio)
- Sample disturbance from sampling and possible re-creation of stress history in the triaxial cell
- Rough or smooth rigid end platens (a few attempts with flexible)
- Failure criterion as maximum deviator stress or maximum principal stress ratio
- Application of back pressure as a means to achieve acceptable degree of saturation

Extensive experience with testing in soft, homogeneous (marine) clays have been published in relation to some of the controversies above (e.g. Berre 1979 and 1982, Lacasse and Berre 1988, Lunne et al. 2007, Berre et al. 2007). However, it may not necessarily be transferable to testing of very stiff to hard overconsolidated glacial tills and vice versa. For the soft clays (*OCR* up to two) the axial strain to failure is typically of the order of 2-5% (followed by strain softening) whereas it is in excess of 10-15% for the high strength overconsolidated clay tills. However, Lacasse and Berre (1988) report on tests on Drammen clay with laboratory induced *OCR*s of up to 40. They conclude, that higher compressive strengths are observed for *H/D* = 1 (smooth pressure heads), but only at strains higher than 10 %. Furthermore, the initial part of the stress-strain curve is steeper when employing rough end platens.

From a theoretical point of view specimens with *H/D* = 1 and frictionless pressure heads provides a homogeneous stress-strain field in the sample and hence mimic a theoretical element test. This further implies that the principal stress directions are well-defined acting vertically (piston pressure plus cell pressure) and horizontally (cell pressure). The "smooth" end platens are ensured by high vacuum grease located in-between a number of membranes. However, the deformations in the grease and the membranes are non-linear and stress dependent, which is difficult to account for in the interpretation of the tests.

In contrast the *H/D* = 2 specimens with rough pressure heads produce non-uniform stress-strain fields with "dead zones" below the pressure heads and may show pronounced stress-strain peak behaviour and post failure softening. This is the reason for the requirement of use of proximity strain devices on the middle third of the sample to get representative axial and radial strain measurements (mostly applied in research).

However, the *H/D* = 2 samples also promote the development of shear bands (bifurcation) in particular for low-plasticity, fissured or heterogeneous soil samples, which may be the failure mechanism for many real life situations. The creation of a shear band in combination with highly dilative soils may infer that water flows into the shear band from the stiff zones surrounding it. Hence, the strength will theoretically decrease and the shear band acts as a "drainage line".

Re-creation of the stress history (by loading the samples to the insitu preconsolidation pressure followed by unloading to the insitu stress and thereafter take the specimen to failure) and thereby also reduce sample disturbance has been Danish tradition for decades. Jacobsen (1970) states the importance of doing this. However, as noted by Berre (1982) it can lead to significant reduction in the water content. This may result in a too high stiffness and strength. Lately, some of the clay tills at the Fehmarn Belt have been tested without pre-loading samples. An alternative to Danish tradition has been proposed by Ladd and DeGroot (2003).

Many of the above controversies were addressed by Jacobsen (1967, 1968, 1970, 1979) in relation to the development of the new Danish triaxial apparatus and testing of clay till. His conclusions clearly advocated the use of *H/D* = 1, smooth pressure heads as well as pre-loading samples. However, the tremendous efforts to develop the apparatus, the testing technique and addressing most of the controversies simultaneously, somewhat weakens the conclusions in that the test series were not strictly carried out to allow a one-one comparison.

The test series in the new apparatus with direct comparison of height/diameter ratios (*H/D* = 0.5, *H/D* = 1 and *H/D* = 2) were carried out as UU tests (unconfined compression tests with a confining membrane but no confining pressure). Furthermore, the sample diameter was 35mm and a clear definition of failure was not provided.

To back up the theoretical considerations a series of ten CAU triaxial tests were carried on clay till in Malmö for the Citytunneln project using *H/D* = 2 and rough pressure heads. The average water content was 13% and the undrained shear strength from the field vane was 267 kPa thus comparable to the test series presented in this paper and by Jacobsen (1970).

The triaxial tests all showed a distinct barrel shape and in some cases clear bifurcation. Failure was defined as the maximum deviator stress (for the undrained shear strength) at axial strains from 10 to 15%. All tests were carried out to 15- 20% axial strain. The average undrained shear strength in the triaxial tests was 193 kPa and notably the drained triaxial friction angle was 30.5 degrees (c’ = 26 kPa) kPa which is at the lower range expected based on *H/D* =1 tests on similar clay till in Denmark.

3 TEST PROGRAMME

3.1 Clay till

The clay till tested was sampled in relation to the New Storstrømmen Bridge project in Denmark. This new bridge will connect the islands of Zealand and Falster in the South-eastern part of Denmark.

Table 1 Summary of classification tests

	Depth [m]	**w [%]**	**w_L [%]**	**w_P [%]**	**I_P [%]**
B09A	9.0	12.3	21.5	12.4	9.1
B09A	9.9	11.2	20.4	12.6	7.8
B14	6.62	9.6	21.5	11.2	10.4
B34	13.9	10.4	19.7	10.2	9.5
B34	14.7	11.1	18.3	9.9	8.5

High quality core samples with *D* = 102 mm were retrieved with the Geobor S system. In some cases it was necesarry to trim down the specimen to a diameter of approximately 70 mm (laboratory capacity demands).

Due to the heterogeneity of the soil, specimens for comparison were sampled from the same core within approximately one meter distance, see Section 3.3.

The deposits are generally firm to very stiff clay till, slightly sandy to sandy and slightly gravelly to gravelly. The colour is grey to brownish grey to dark brown. Clasts of chalk and flint are found as well as a few cobbles.

Classification test values are summarized in Table 1 and Table 2. The initial water content, *w*, varies between 9.6 and 15 % with the majority around 12.5%. Plasticity index tests show I_p between 7.8% and 10.4%. The classification values are in the range normally observed for Danish clay tills east of Storebælt.

The clay till is highly overconsolidated with overconsolidation ratios *OCR* from 4.9 to 8.3.

3.2 Stress paths

In principle, all tests are carried out as anisotropically consolidated undrained triaxial compression tests with pore water measurements and constant cell pressure during the shear phase to failure.

All specimens have been pre-loaded in order to reduce the effects of sample disturbance and to partly restore the insitu stress history (Steenfelt and Foged, 1992).

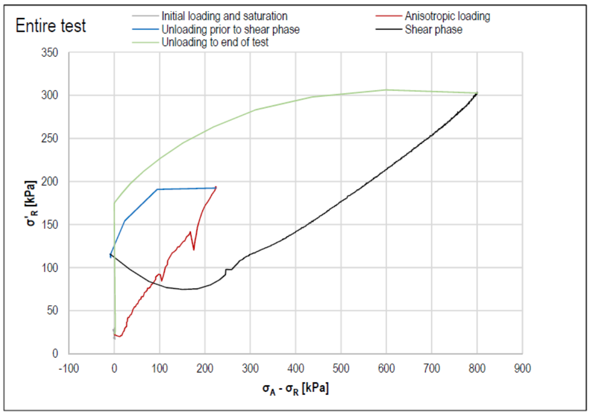

Figure 1 Stress path for a Type 1 test, B 09 (B) cf. Table 2. Red line: Anisotropic loading, blue line: unloading prior to shear phase, black line: Shear phase.

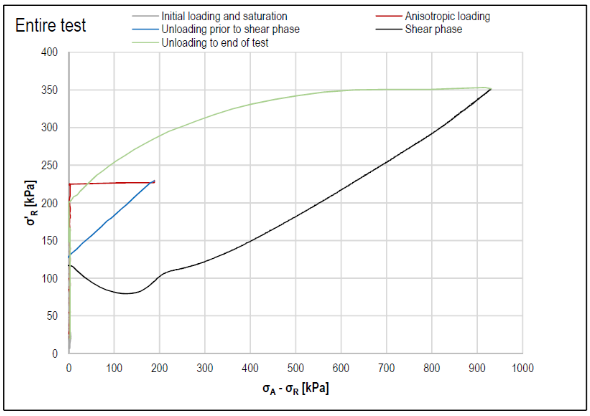

Figure 2 Stress path for a Type 3 test, B 09 (A) cf. Table 2. Grey line: Initial isotropic loading, red line: Anisotropic loading, blue line: unloading prior to shear phase, black line: Shear phase.

Two types of tests - distinguished by differences in the consolidation phases - were carried out, denoted Type 1 and Type 3 in the following.

Type 1 tests have been undertaken according to Danish Practice using *H/D* = 1 and smooth pressure heads. The stress history is “replicated” employing area-constant consolidation (loading and unloading), in which the specimen is constrained in the horizontal direction by adjusting the cell pressure. After (i) saturation, the specimen is (ii) taken to approximately 80% of the vertical pre-consolidation pressure followed by (iii) unloading (also area-constant) to the in situ vertical stress, before (iv) shearing undrained to failure (at a constant strain rate and cell pressure). During the area-constant consolidation the specimen follows it’s "true" K_0-path and hence the horizontal effective stresses do not need to be estimated (in contrast to the Type 3 tests). Figure 1 shows a typical stress path followed in a Type 1 tests. The red and blue lines show the stress path for the area-constant loading and unloading during the consolidation phase. The black line in Figure 1 indicates the effective stress path for the shear phase to failure, which clearly shows that the soil exhibits a dilative behaviour during the shearing.

A simplified consolidation procedure (in terms of test control and time duration) was adopted for the Type 3 tests. However, compared to Type 1 tests, the test specification requires more input from the Designer.

In the test campaign, Type 3 tests have been undertaken for specimens with *H/D* = 1 and smooth pressure heads as well as a *H/D* = 2 and rough pressure heads. The test sequence adopted for the consolidation phase was: (i) saturation, (ii) isotropic loading to 80% of the estimated horizontal effective pre-consolidation pressure. (iii) Anisotropic loading to 80% of the vertical effective pre-consolidation pressure with a constant cell pressure equal to the stress state described under item (ii), (iv) Unloading to a stress state representing in situ conditions. The horizontal effective stress was established based on the in situ overconsolidation ratio, *OCR*, using

$$K_{0,OC} = (1 - sin\varphi) \cdot OCR^{sin\varphi} \qquad (1)$$

where $K_{0,OC}$ is the coefficient of earth pressure at rest corresponding to the insitu conditions. *OCR* has been evaluated based on oedeometer (incremental loading and constant rate of strain tests) and piezocone penetration tests. Again, the last phase of the test involves undrained shearing to failure at a constant strain rate and constant cell pressure. Figure 2 shows a typical stress path followed in a Type 3 tests. The red and blue lines show the consolidation phases (Items (ii) and (iii) described above). The black line indicates the effective stress path for the shear phase to failure, which clearly shows that the soil exhibits a dilative behaviour during the shearing.

The rationale for the pre-consolidation to 80% of the estimated pre-consolidation pressure in both types of tests was to partly restore the stress history without destroying the initial structure of the specimen. Accidentally, the tests on B34 were taken beyond the pre-consolidation stress. However, the impact on the results and comparisons undertaken seem negligible.

3.3 Test details

11 successful tests were carried out. All tests were conducted on high quality specimens with limited sample disturbance.

Three of the tests are of Type 1 and eight are of Type 3. In seven tests H/D = 1 and in four H/D = 2. Specimens with diameters of approximately 70 mm (six tests) and 100 mm (five tests) were tested.

An overview of the test types, specimen dimensions, consolidation stresses and strength parameters are shown in Table 2. $\sigma'_A = \sigma'_1$ and $\sigma'_R = \sigma'_3$ are the axial and radial stresses, respectively, at the end of the different stages of the consolidation phase. The subscripts *loa.* and *unl.* denote loading and unloading, respectively. w_{init} is the natural water content at the start of the test whereas

$$K_0 = \frac{\sigma'_R}{\sigma'_A} = \frac{\sigma'_3}{\sigma'_1} \tag{2}$$

is the coefficient of earth pressure at rest corresponding to the end of the loading and unloading phases. Hence, the stresses reflect the different stress paths for the consolidation parts of the Type 1 and Type 3 tests as described in Section 3.2.

The same values of the coefficient of earth pressure at rest, K_0, and the mean effective stress, p' $(=\sigma'_1 + 2\ \sigma'_3)/3)$, are specified for tests to be compared. This was achieved for all tests except B 14 (C). However, this is accounted for as described below. It should be mentioned that this test was conducted at a very early stage of the test campaign.

Due to the way the Type 1 tests are undertaken (cf. Section 3.2), the K_0-values and the mean effective stresses in the loading and unloading phases differ in some circumstances from the corresponding stresses in the comparable Type 3 tests, in which the K_0 values are a part of the test specifications.

Table 2 Test characteristics and results.

Label	Borehole no.	Depth	Test type	H	D	H/D	w_{init}	$\sigma'_{A,}$	$\sigma'_{R,}$	$\sigma'_{A,loa.}$	$\sigma'_{R,loa.}$	K_0	$\sigma'_{A,unl.}$	$\sigma'_{R,unl.}$	K_0	Strain rate	σ'_A ε_1=10%	σ'_R ε_1=10%	s_u	ε_{50}
	[-]	[m]	[-]	[mm]	[mm]	[-]	[%]	[kPa]	[kPa]	[kPa]	[kPa]	[-]	[kPa]	[kPa]	[-]	[%/hr]	[kPa]	[kPa]	[kPa]	[%]
B09 (A)	B09A-BH	9.20-9.30	3	102.6	96.1	1	13.4	229	229	418	229	0.55	105	118	1.12	0.66	911.1	242.6	334.2	3.06
B09 (B)	B09A-BH	9.90-10.10	1	100.0	100.0	1	12.6	N/A	N/A	416	192	0.46	106	114	1.08	0.66	878.5	232.3	323.1	1.18
B09 (C)	B09A-BH	10.20	3	181.4	99.0	2	15.0	227	227	416	227	0.55	104	118	1.13	0.66	879.0	246.0	316.5	1.61
B14 (A)	B14-BH	6.72-6.79	1	70.0	70.0	1	11.3	N/A	N/A	419	159	0.38	63	69	1.10	0.20	611.8	146.4	232.7	0.77
B14 (B)	B14-BH	6.81-6.88	3	70.0	70.0	1	11.4	182	182	418	182	0.44	62	90	1.45	0.20	635.5	168.1	233.7	1.05
B14 (C)	B14-BH	5.31	3	140.2	69.8	2	10.7	177	177	351	177	0.50	58	79	1.36	0.50	677.5	188.9	244.3	1.11
B14 (D)	B14-BH	5.72	3	204.8	97.4	2	11.3	184	184	416	183	0.44	59.3	89	1.50	0.20	700.8	194.9	253.0	1.75
B14 (E)	B14-BH	7.07-7.17	3	100.0	99.5	1	11.0	183	183	419	183	0.44	58	87	1.50	0.20	683.9	182.8	250.6	1.95
B34 (A)	B34-BH	13.92-13.99	1	70.0	70.0	1	12.5	N/A	N/A	584	255	0.44	121	106	0.88	0.29	646.9	163.5	241.6	0.26
B34 (B)	B34-BH	14.09-14.16	3	70.0	70.0	1	13.4	255	255	582	255	0.44	121	144	1.19	0.29	717.0	169.3	273.8	0.44
B34 (C)	B34-BH	14.70	3	140.0	70.0	2	12.3	257	257	581	257	0.44	122	145	1.19	0.29	799.1	208.8	295.1	0.25

This could ideally have been avoided if the Type 3 tests were conducted after the Type 1 tests. However, this was not possible due to time constraints in the laboratory campaign for the project.

The strengths compared in Section 4 are the undrained shear strengths, s_u, shown in Table 2 corrected for differences in loading and unloading mean stress level and thereby K_0. With the considerations presented in Steenfelt and Foged (1992) as a starting point, s_u is more rigorously corrected due to differences in mean effective stress and laboratory induced overconsolidation ratio, $R = p'_{max}/p'_{min}$. p'_{max} and p'_{min} are the mean effective maximum (= p'_{loa}) and minimum stresses (= p'_{unl}) in the consolidation phase, respectively. Based on Critical State Soil Mechanics with the modified Cam Clay conceptual soil model it appears after some manipulation that

$$\frac{s_{u,ii}}{s_{u,i}} = \left(\frac{p'_{loa,ii}}{p'_{loa,i}}\right)^{\Lambda} \cdot \left(\frac{p'_{unl,ii}}{p'_{unl,i}}\right)^{1-\Lambda} \qquad (3)$$

where the indexes ii and i refer to two different stress conditions. $\Lambda = 0.85$, similar to the power in the SHANSEP relation for clay till, has been adopted, cf. Steenfelt and Foged (1992). The mean effective stress, p', in the triaxial set up is

$$p' = \frac{\sigma'_A + 2\cdot\sigma'_R}{3} = \frac{\sigma'_1 + 2\cdot\sigma'_3}{3} \qquad (4)$$

As indicated in Table 2, the strain rate (=0.5%/h) applied in the shear phase to failure in B14 (C) differs from the strain rate (=0.2%/h) applied for the other B14 tests. Therefore, the undrained shear strength for B14 (C) has been corrected based on the recommendations by Lunne et al. (2006), i.e. on average the undrained shear strength increases by 9.4 % per log cycle of strain rate.

Typically, the stress-strain behaviour did not exhibit a pronounced peak in a deviator stress, q, – axial strain, ε_1, plot; hence the undrained shear strength, $s_{u,qmax}$, corresponds to an axial strain of 20% (the approximate axial strain at which the tests were terminated). In contrast, when plotting the principal stress ratio (σ'_1/σ'_3) versus the axial strains, a peak and thereby an undrained shear strength, $s_{u,max\sigma1/\sigma3}$, can be found. However, making use of this criterion to define failure in case of design, a lower bound of the undrained shear strength is estimated. Hence, in this paper the undrained shear strength, $s_{u,\varepsilon1=10\%}$, is defined according to a certain axial strain, which is chosen to be 10% according to common practice. These values are given in Table 2.

The pre-failure stress-strain behaviour is in a simple manner characterised by ε_{50}, which is the axial strain corresponding to 50% of the deviator stress q at failure. ε_{50} has not been corrected for the stress level and the overconsolidation ratio.

4 RESULTS

The clay tills tested are relative weak (uncorrected undrained shear strengths vary between 230 kPa and 350 kPa, cf. Table 2) compared to other clay tills encountered in Denmark; however comparable to those tested by Jacobsen (1968, 1970). Furthermore, the *OCR*s (vary between 4.9 and 8.3) are not particularly high. Still, in all tests the soil exhibited a highly dilative nature when approaching failure (see for example Figure 6 and Figure 12) and in the none of the tests a clear shear band (bifurcation) were detected, even for $H/D = 2$. Hence, the observations presented in the following may not be applicable for the stiffest and strongest clay tills found in Denmark and Northern Europe.

In the following sections, the effects of sample size ($H/D = 1$ versus $H/D = 2$) and stress path (Type 1 versus Type 3) on the undrained shear strength and the pre-failure stress-strain behaviour are elucidated.

For each label in the legend, the letters in parenthesis refer to Table 2 and they indicate the tests that are compared, e.g. B14 (D-E) indicates that tests B14 (D) and B14 (E) in relation to B14-BH are compared. Furthermore, the first letter in the parenthesis refers to the value of the ordinate and the second to the abscissa. This notation and methodology are employed throughout Section 4 and it implies that each data point in a graph involves two tests. Solid lines bisect the plots, i.e. data points located on these lines indicate a perfect match between the parameters compared.

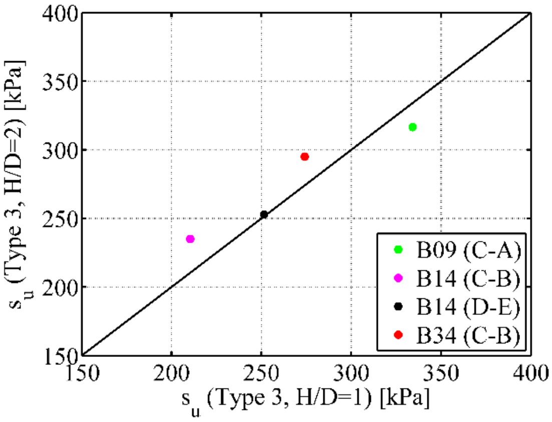

Figure 3 Comparison of undrained shear strengths based on Type 3 tests for H/D = 1 and H/D=2.

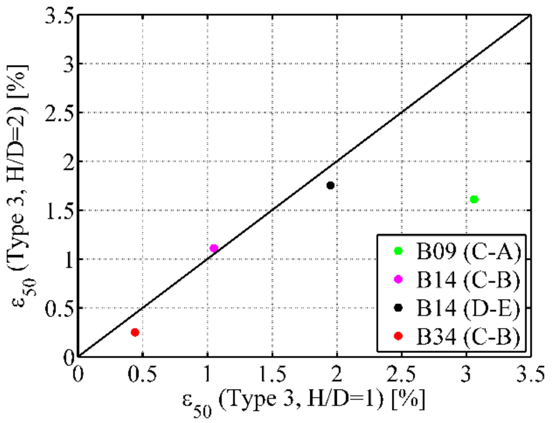

Figure 4 Comparison of uncorrected ε_{50} based on Type 3 tests for H/D = 1 and H/D=2.

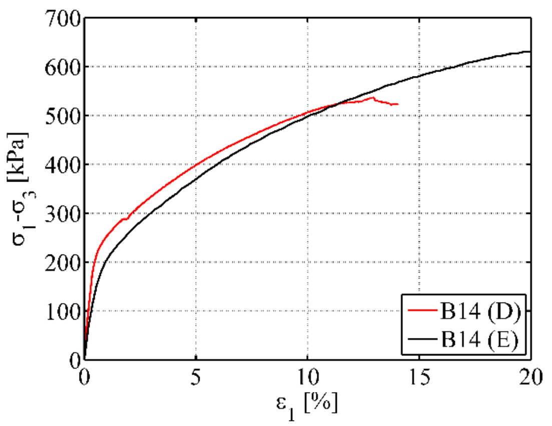

Figure 5 Deviator stress – axial strain for B14 (D) (Type 3, H/D = 2, D=100mm) and B14 (E) (Type 3, H/D=1, D = 100mm).

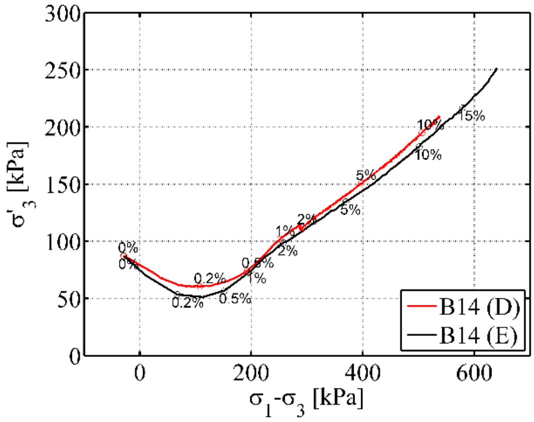

Figure 6 Effective stress paths for the shear phase for B14 (D) (Type 3, H/D = 2, D=100mm) and B14 (E) (Type 3, H/D=1, D = 100mm). 0% to 15% indicate the axial strains.

4.1 *H/D ratio*

The influence of height-diameter ratio, *H/D*, on the undrained shear strength, s_u, and ε_{50} is shown in Figure 3 and Figure 4, respectively. The tests compared have the same diameter and follow similar stress paths in the consolidation phase.

Figure 3 indicates, based on the limited amount of tests, that there is no significant difference in undrained shear strength (ε_1 = 10%), from a design point of view, between conducting *H/D* = 1 and *H/D* = 2. The variation is below ± 10%. There is a tendency that *H/D* = 2 gives rise to the highest strength for relatively low undrained shear strengths (<250 kPa), whereas *H/D* = 1 provides higher undrained strengths for strengths exceeding 250 kPa. Furthermore, when increasing the diameter the undrained shear strength from the *H/D* = 1 tests exceeds the strengths based on the corresponding *H/D* =2 tests.

Compared to the variation in s_u, the variation in ε_{50} is higher, as expected, cf. Figure 4. However, for the tests on B14 the variation is below ± 10%. Despite differences, the stress-strain curve shapes can be very similar as exemplified in Figure 5 for B14 (D) and B14 (E). Generally, the pre-failure stress-strain curve for *H/D* = 2 is stiffer than the corresponding curve for *H/D* = 1. Jacobsen (1968, 1970) also reported this based on unconfined compression tests.

Figure 7 Specimen shape at failure (ε_1 = 10%) for B34 (C), Type 3, H/D=2, D = 70mm.

Figure 8 Specimen shape at failure (ε_1 = 10%) for B34 (B), Type 3, H/D=1, D = 70mm.

Figure 5 indicates that for B14 (D), *H/D* = 2, a potential shear band may have started to develop at the end of the test, which is not the case for B14 (E), *H/D* = 1. However, a shear band cannot be detected from the sample photos. For other *H/D* = 2 tests (not shown here) the deviator stress-strain curves also start to flatten out after 12 – 15% axial strain indicating that a potential shear band develops, which theoretically should be the failure mechanism, see Section 2. This is not the case for the *H/D* = 1 tests. This is also reported by Berre (1982).

Exemplified by B14 (D) and B14 (E), cf. Figure 5 and Figure 6, there is a relatively good match between the effective stress paths and stress-strain curves.

The advantage of testing *H/D* = 1 and employing smooth end platens is that homogenous stress and strain conditions exist in the specimen. If such conditions prevail, the specimen keeps its cylindrical form when approaching failure. Figure 7 and Figure 8 show the shape of B34 (B) and B34 (C), respectively, for ε_1 = 10%. As expected, the *H/D* = 2 test develops this "barrel-shaped" form. But maybe more surprisingly, even though it is not to the same extent, the *H/D* = 1 test also exhibits this "barrel-shaped" form.

Generally, the influence of the definition of failure on the undrained shear strength seems higher compared to the effects of testing samples with height-diameter ratios of either unity or two.

4.2 *Stress path*

The influence of consolidation stress path on the undrained shear strength, s_u, and ε_{50} is shown in Figure 9 and Figure 10. The tests compared have the same height-diameter ratio and diameter.

Figure 9 indicates, based on the limited amount of tests, that there is no significant difference in undrained shear strength (ε_1 = 10%), from a design point of view, between conducting Type 1 and Type 3 tests. The variation is below ± 10%. Still, there is a tendency that Type 1 tests gives rise to the highest strength.

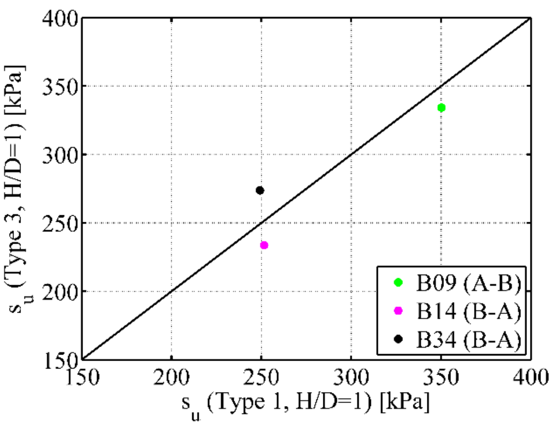

Figure 9 Comparison of undrained shear strengths based on Type 1 and Type 3 tests for H/D = 1.

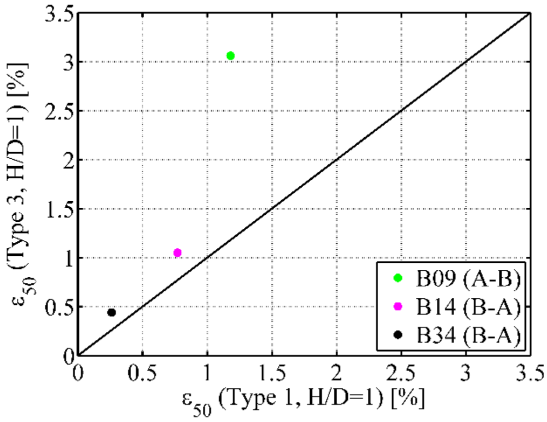

Figure 10 Comparison of uncorrected ε_{50} based on Type 1 and Type 3 tests for H/D = 1.

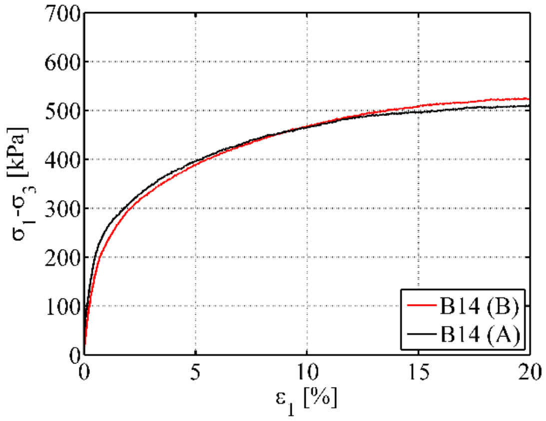

Figure 11 Deviator stress – axial strain for B14 (A) (Type 1, H/D = 1, D = 70 mm) and B14 (B) (Type 3, H/D = 1 and D = 70mm).

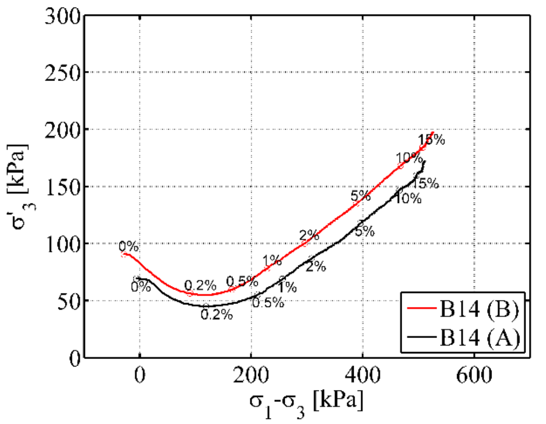

Figure 12 Effective stress path for the shear phase for B14 (B) (Type 3, H/D = 1, D = 70 mm) and B14 (A) (Type 1, H/D = 1, D = 70mm). 0% to 15% indicate the axial strains.

Figure 10 indicates, based on uncorrected ε_{50}s, that the pre-failure stress-strain curves for Type 1 tests is stiffer than the corresponding curves for Type 3 tests. Despite differences, the shape of the stress-strain curves can be very similar as seen in Figure 11 for B14 (B) and B14 (A).

Figure 12 shows the effective stress paths for B14 (A) and B14 (B). There is a relative good match between the effective stress paths. The difference may be due to the fact that the unloading stress state is not completely identical for the two tests.

Steenfelt and Foged (1992) and Jacobsen (1970) state the importance of pre-loading the clay till samples before shearing it to failure. This has been Danish tradition for decades and it is undertaken to replicate the insitu stress history. However, as noted by Berre (1982) it can lead to significant reduction in the water content. This may result in a too high stiffness and strength. Lately, some of clay tills at the Fehmarn Belt have been tested without pre-loading samples. From the authors point of view it is recommended to preload clay till samples, especially if the *OCR* is high, if the test campaign is limited and if prior knowledge about the subject for the tills to be tested is not available.

Results (not presented here) from the New Storstrømmen Bridge indicate that preloading samples yield undrained shear strengths that are higher compared to samples that have not been pre-loaded.

5 CONCLUSIONS

The basis for the Danish tradition of undertaking trixial testing on clay tills has been reviewed (Section 2). A tremendous and outstanding work has been done in the early days by Jacobsen (1967, 1968, 1970, 1979) to update and cope with shortcomings and uncertainties of testing highly overconsolidated and stiff clays. Theoretically, the proposed way of testing clay till seems plausible and correct. But the experimental documentation is weakened since the test series where *H/D* = 1 (smooth end platens) and *H/D* = 2 (rough end platens) are compared were not strictly carried out to allow a one-one comparison.

The results from a limited test campaign have been presented in this paper. The effects of conducting *H/D* = 1 and *H/D* = 2 on the undraind shear strength and prefailure stress-strain characteristics have been investigated. The results indicate, from a design point of view, that no significant differences are observed if the tests are carried out by well renowned and highly experienced companies. The same is the case if comparing results of tests in which highly sophisticated area-constant consolidation stress paths are compared with a much more simplified and faster consolidation stress paths. Furthermore, it is important to test samples with a minimum diameter of 70 mm.

Generally, the influence of the definition of failure (here $\varepsilon_1 = 10\%$) on the undrained shear strength seems higher compared to the effects of testing samples with height-diameter ratios of either unity or two or employing different consolidation stress paths.

The clay tills tested are relative weak (undrained shear strengths between 230 kPa and 350 kPa) compared to other Danish clay tills. Furthermore, the *OCR*s (ranging between 4.9 and 8.3) are not particularly high. Hence, the observations presented may not be applicable for the stiffest and strongest clay tills found in Denmark and Northern Europe.

This paper is intended as an appetizer and more research should be undertaken before decisive conclusions can be drawn.

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