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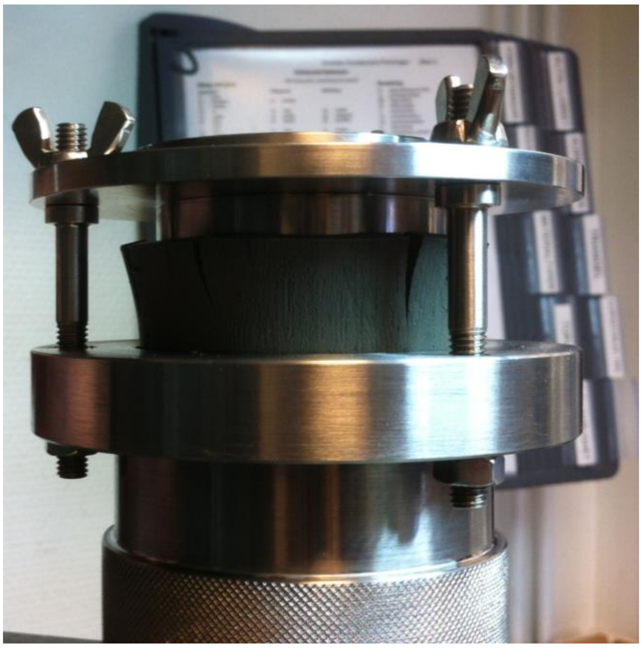

Sample quality and disturbance in soft marine clay

A comparative study of the effects from using two different sized piston samplers

Master of Science Thesis in the Master's Programme Infrastructure and Environmental Engineering

ROBERT LANZKY
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Cover:
Sample being trimmed directly into a CRS cell.

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ABSTRACT

When determining the properties of clay through sampling and laboratory testing the issue of sample disturbance needs to be taken into consideration. Depending on the sampling method and how the sample is handled, the sample will experience different levels of disturbance. The purpose of this study has been to determine if a 60 mm piston sampler could be an option to the standard 50 mm St II sampler for achieving higher quality samples. The hypothesis being that with an increased diameter of the sampler, the most disturbed periphery of the sample can be removed while using the same testing equipment, thus receiving a higher and more accurate shear strength and preconsolidation pressure in laboratory tests.

The basis for the thesis has been CRS oedometer and CAUC triaxial tests performed on soft marine clay from three different locations in the Göteborg region. Based on the results from these tests and observations made throughout the sampling and testing procedures the performance of the new sampler has been evaluated.

A statistical analysis has been performed on the CRS results showing no improvement in preconsolidation pressure and lower elastic and plastic moduli. However more consistent results were obtained with the 60 mm sampler compared to a larger scatter for the 50 mm sampler, although the overall scatter in the results was small. The triaxial tests show similar tendencies with equal to lower shear strength and stiffness for the 60 mm samples. Based on these results it cannot be established that the 60 mm sampler achieves samples of higher quality and a change of standard sampler is not feasible. However, if it is the sampling or the handling and preparation of the samples that is the cause of the lack of improvement cannot be easily identified. Recommendations are put forward for further studies on investigation of sampling.

Key words: clay, disturbance, sample quality, piston sampler, shear strength, preconsolidation pressure

Contents

Preface

This thesis presents a comparative study of two different sized piston soil samplers. The basis for this thesis has been CRS oedometer and CAUC triaxial tests performed on soft marine clay from three different locations in the Göteborg region, Sweden. Sampling and testing have been performed during spring 2013. The project was started by the geotechnical company Bohusgeo and has been carried out at the department of Civil and Environmental Engineering, Geo Engineering, Chalmers University of Technology, Sweden.

We would like to thank our supervisors Tara Wood and Tobias Thorén for their expertise and guidance throughout the project, Peter Hedborg for his work in the Chalmers laboratory and Bohusgeo for their help and for providing us with the project in the first place.

Göteborg, December 2015

Robert Lanzky & David Palmquist

Notations

Roman letters

A	area
c_v	coefficient of variation (statistics)
D_1	internal diameter of sampler cutting shoe
D_2	external diameter of sampler cutting shoe
D_3	internal diameter of sample tubes
g	gravity
G_0	initial shear modulus
G_s	specific gravity
i	cone penetration depth (in mm)
I_L	liquidity index
I_P	plasticity index
k	permeability
k_i	initial permeability
K	parameter depending on the angle of a fall cone
L	length
m	mass
m_s	mass of solid matter
m_w	mass of water
M_0	elastic oedometer modulus
M_L	plastic oedometer modulus
P	force
S_t	sensitivity
t	time
u	pore water pressure
u_b	pore pressure at undrained surface (CRS oedometer test)
V	volume
V_s	shear wave velocity
w	water content
w_L	liquid limit
w_N	natural water content
w_P	plastic limit

Greek letters

ε_{vo}	volumetric strain
μ	mean value (statistics)
ρ	density
σ	standard deviation (statistics)
σ	total normal stress
σ_1	major principal stress
σ_3	minor principal stress
σ'	effective normal stress
σ'_c	preconsolidation pressure
σ'_0	effective overburden pressure
τ_f	maximum shear strength
τ_{fu}	undrained shear strength
τ_r	remolded shear strength

Abbreviations

CAUC	anisotropically consolidated undrained triaxial compression
CRS	constant rate of strain
CPT	cone penetration test
CTH	Chalmers University of Technology
DSS	direct simple shear
ESP	effective stress path
GWL	ground water level
NGI	Norwegian Geotechnical Institute
OCR	overconsolidation ratio
PTFE	polytetraflouroethylene (colloquially referred to as Teflon)
SGF	Swedish Geotechnical Society
SGI	Swedish Geotechnical Institute
SIS	Swedish Standards Institute
TSP	total stress path

1 Introduction

1.1 Background

When determining the properties of clay through sampling and laboratory testing the issue of sample disturbance needs to be taken into consideration. Depending on sampling method and how the sample is handled, e.g. during transport, storage and in the laboratory, the amount of disturbance will differ which in turn will affect the soil parameters. The consequences of this can be greater expenses and, amongst other things, greater risks of structural damage in geotechnical structures and their surroundings.

The standard method of clay sampling in Sweden today is to use the St II clay sampler. It is a piston sampler with an inner diameter of 50 mm. Most laboratory test methods are designed to use specimens of the same diameter. It has been shown that piston sampling often disturbs the sample to some extent depending on the clay properties and how well the sampling is executed (Simons, Menzies & Matthews, 2002).

Previous studies show that the quality may increase with increased sample diameters (Lunne, Berre & Strandvik, 1999). The geotechnical consulting firm Bohusgeo has acquired a piston sampler with a diameter of 60 mm in addition to the standard 50 mm sampler. The hypothesis is that the outer layer of the sample is the most disturbed part. Therefore, the quality of the sample may improve if the outer layer is removed. When increasing the diameter to 60 mm, a five millimetre thick layer of the outer part of the sample can be removed before laboratory testing. The 60 mm sampler was built from existing blueprints and chosen because it is easy to use, compatible with existing sampling equipment and cost effective compared to larger samplers.

1.2 Purpose

The purpose of this study is to investigate the quality of clay samples taken with the 60 mm sampler compared to those taken with the standard St II 50 mm. Factors affecting the disturbance of the sample will try to be identified and the magnitude of disturbances will be assessed. The aim of the project is to conclude if a change of clay sampler is an attractive and feasible option with regard to sample quality and practical use.

1.3 Question formulation

The following questions have formed the basis of the investigation.

- Which factors affect sample disturbance?
- How can the quality of the clay samples be measured?
- Is the quality of the samples improved when the sampling diameter is increased from 50 mm to 60 mm?
- Does the change in quality differ when the tests are performed on clays with different properties?
- Is the 60 mm sampler a feasible option to the standard 50 mm sampler?

1.4 Limitations

This study is primarily based on CRS (constant rate of strain) oedometer tests and CAUC (anisotropically consolidated undrained triaxial compression) tests for the determination of the quality of the clay samples. Index tests have also been performed. Existing data from the same locations has been used for comparison. At Regionens Hus, only 60 mm samples were taken as part of this study and the results are compared to existing results from 50 mm samples. At Nödinge, 50 mm samples taken as a part of this study were only used for CRS oedometer tests. The results of 50 mm CAUC triaxial tests were taken from results of an ongoing research at CTH. In terms of soil sample disturbance, focus is on the sampling and the handling in the laboratories while disturbance during transportation and storage is only mentioned briefly.

1.5 Method

The project contains two main parts, a literature study and a laboratory study. The focus of the literature study has been on test methods, clay behaviour and more specifically; disturbance factors and stress/strain situations in clay samples. The outcome of the literature study is presented in the theory chapters below and the knowledge gained formed foundation for the analysis of the laboratory test results. Results from the laboratory testing are presented, analysed and discussed in chapters 8-10.

In order for a more qualitative study and fair comparison, samples from different locations have been studied. Initially, the sampling was done at two different locations; 'Regionens Hus' in central Göteborg and 'Lerum Centrum', the centre of the town Lerum. At a later stage, Nödinge was added in order to expand the study. The different test sites are described in chapter 6.

Field samples have been taken both with the standard diameter of 50 mm and with the new 60 mm equipment. The samples have been taken from the same depth and at the same location in order to allow for a fair comparison and the results will also be compared to earlier test results from the same sites. The 60 mm samples have been trimmed to 50 mm and tested with the same equipment as the standard samples. CRS oedometer and CAUC triaxial tests have been performed, though the main focus has been on the first method. The triaxial test cell is equipped with a bender system and this has been used in some cases.

Throughout the sampling and testing of the clay specimens, a thorough documentation has been made in order to make it easier to identify critical stages of the process and secure the quality of the investigation.

2 Theory

This chapter will briefly describe the theory behind the methods for determining the properties of clay.

2.1 Composition of clay

A soil consists of the three phases; solid, liquid and gas. The solids are mineral and clay particles that form the structure of the soil and there may also be organic material. The liquid and gas fills the pores in the form of water and for example air, carbon dioxide and methane, which may be in gas or liquid form, depending on stress conditions (Craig, 2004).

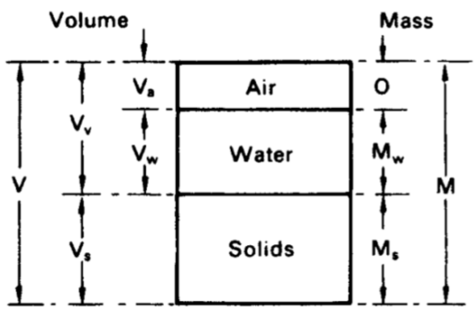

Figure 1 Phase diagram (Craig, 2004).

Clay consists of plate like particles smaller than 0.002 mm that are formed from chemically weathered rock or reworked sediments. In the weathering process water, oxygen and carbon dioxide affects the rock and changes its mineral structure. The clay particles basic structural elements are silicon-oxygen tetrahedrons and aluminum-hydroxyl octahedrons that form silica sheets and gibbsite sheets respectively. One gibbsite sheet together with either one or two silica sheets then form layers that joined together creates the clay particle. Depending on the clay mineral the layers are joined in different ways. Illite for example, which is the most common clay mineral in Sweden (Larsson, 2008), consists of one gibbsite and two silica sheets that are often bound together with potassium ions.

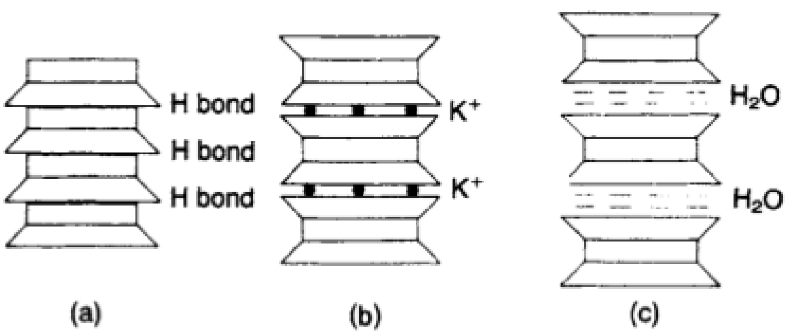

Figure 2 Different types of clay minerals: (a) kaolinite, (b) illite and (c) montmorillonite (Craig, 2004)

Due to negative charges in the clay particles they attract cations and water that can be bound to the surface. However since the forces are weak the cations can be replaced by other cations in the pore water. The aluminum in the octahedrons might also be replaced by magnesium and is then called Brucite instead. The different clay particles are attracted to each other by weak van der Waals forces.

The composition of the pore water depends on the environment in which the clay was formed and all subsequent chemical processes and water flows. If no such processes or flows have occurred the pore water will have the same composition as when the clay was formed. Leaching in a clay formed in a marine environment will reduce the salt content and infiltration might add new substances. Weathering might free previously bound ions/cations and induce other changes to the chemical structure. These processes can significantly affect the geotechnical properties of the clay for example in terms of strength and sensitivity (Rankka et. al, 2004).

In the voids in a soil gases might be present, both in pockets filling the voids or dissolved in the pore water. Above the groundwater level (GWL) this gas will consist mainly of air. Below the GWL carbon dioxide, methane and hydrogen sulphides might also be present.

2.2 Clay structure

The structure of clay varies depending on the factors; clay content, type of clay mineral, sedimentation environment and load history. In Figure 3 different types of clay structures can be seen.

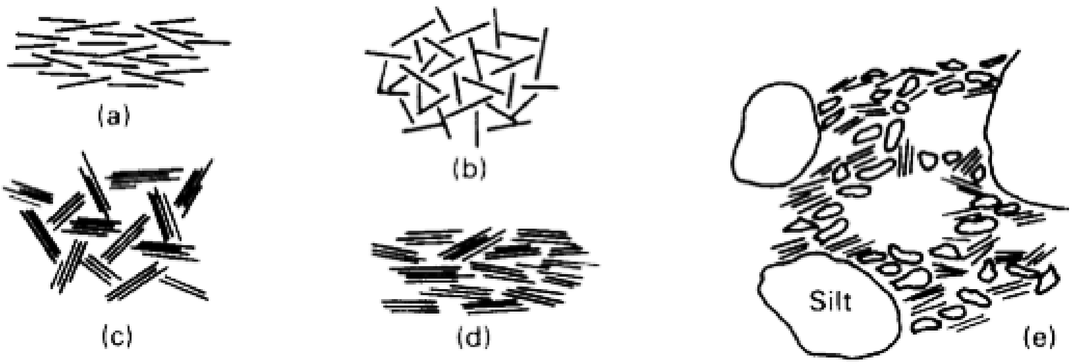

Figure 3 Clay structures: (a) dispersed, (b) flocculaded, (c) bookhouse, (d) turbostratic; (e) example of a natural clay (Craig, 2004)

The dispersed structure, with its face-to-face orientation, form when the net charge in the clay particles is repulsive. Similarly the edge-to-face flocculated structure is formed when there is a net attraction. These structures with interaction between single clay particles are very rare in natural clays (Craig, 2004). Usually aggregates of particles are formed which then come together to form larger structures. The type of structure formed depends on the sedimentation environment. Two such structures are the bookhouse and turbostratic structures, illustrated in Figure 3(c) and (d). In natural clays larger grains are usually present, interacting with the aggregates of clay particles, forming more complex structures, see Figure 3(e).

In the Swedish Illitic clays the clay particles have flocculated to form larger aggregates and/or they have stuck to larger grains of coarser soils (Larsson, 2008). At a high clay content and rapid sedimentation in a marine environment, a very open structure is formed. This is the case in the Göteborg region where large aggregates formed as the fresh melt water carrying clay sediments met the salt water. The large aggregates then sedimented more rapidly together with larger silt particles. The open structure of marine clays is more compressible and will change with an increased load.

The structure of clays with a lower clay content is more similar to coarse soils where larger grains are covered with clay particles and some grains are replaced by aggregates of clay particles. In coarser soils with only a small content of clay the grains are also covered with clay particles and aggregates of clay particles can be found at the contact points between the grains. The coarser grains will be completely separated by clay when the clay contents reach 15-25% of the solids (Larsson, 2008).

A high organic content in a soil will lead to a more open structure, which in turn will lead to a greater compressibility.

2.3 Soil mechanics

2.3.1 Effective stress

Terzaghi was the first to recognize the relationship between effective stress and pore water pressure in 1923. The relationship was based on experimental data and is only applicable to fully saturated soils. It looks as follows:

$$\sigma = \sigma' + u \tag{1}$$

where σ is the total normal stress, σ’ is the effective normal stress and u is the pore water pressure. An increase in total stress will therefore be divided into effective stress and pore water pressure. Initially the increase in total stress will induce an equally large increase in pore water pressure. However, the excess pore water pressure will immediately start to decrease and the soil skeleton will be compressed, thus increasing the effective stress. The clay particles are regarded incompressible and the deformation of the soil is a result of the clay particles being rearranged closer together as the pore water is pushed out and drained at the soil boundary (Craig, 2004).

2.3.2 Yield surface

Compared to most construction materials the stress-strain relationship in soil is very complex. For example steel has relatively simple elastic-plastic deformation behavior and a yield surface according to von Mises or Tresca as can be seen in figure 4. However, due to the complexity of soil behaviour, simplifications have to be made. The failure surface is usually defined by Mohr-Coulombs failure criterion and in combination with knowledge of the vertical and horizontal yield pressures the state boundary surface can be estimated (see Figure 4).

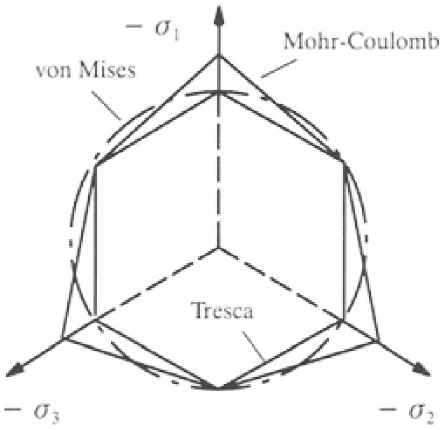

Figure 4 Failure surfaces according to von Mises, Tresca and Mohr-Coulomb (Yu, 2000).

According to Mohr the relationship between the major and minor principal stresses acting on the soil at failure will form Mohr-circles, which in turn is used to create a failure envelope (see Figure 5). According to Coulomb the failure envelope can be described by the following function (Ou, 2006):

$$\tau_f = c + \sigma \tan \phi \tag{2}$$

where τ_f is the shear stress at failure, c is the cohesion, σ the normal stress on the failed surface and ϕ the internal friction angle. Since water cannot carry shear stress only the soil skeleton is resisting the shearing, thus the equation can instead be written with parameters of effective stress:

$$\tau_f = c' + \sigma' \tan \phi' \tag{3}$$

Here c' and ϕ' are only constants for the determination of the linear relationship between shear strength and effective normal stress. In reality the failure surface is not a straight line, which is why it is important to test the soil at relevant stress states (Ou, 2006).

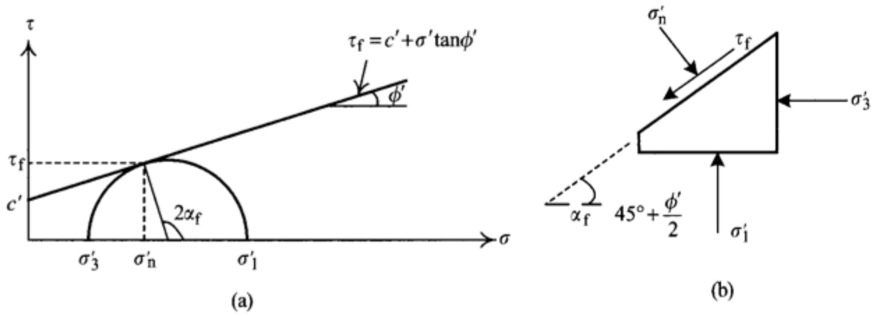

Figure 5 *(a) The failure envelope and a Mohr circle. (b) Stresses acting on the failure surface. (Ou, 2006)*

According to the Mohr-Coulomb failure criterion the deformation behaviour of the soil is elastic – plastic. So if a point is inside the failure envelope it has an elastic behavior and on the failure surface it is perfectly plastic. The theory also states that it is impossible for a point to lie above the failure envelope. This deformation behaviour is however a simplification. In reality the stress strain relationship is non-linear in the elastic as well as the plastic range (see Figure 6).

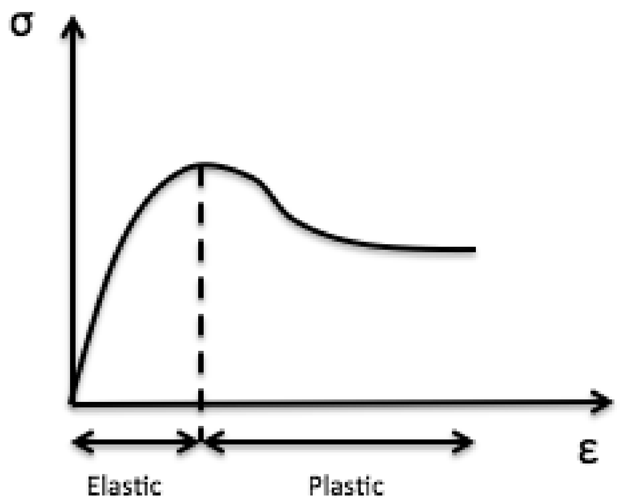

Figure 6 *Non-linear stress-strain relationship in clay.*

2.3.3 Stress path

The different stress conditions in a soil sample during triaxial testing can be represented by Mohr-circles. However, for many successive stress states a simpler way is to display connected points instead, where the points represent the top of the Mohr-circles (Kompetenscentrum Infrastruktur). These connected points are called a stress path, see Figure 7. The stress paths for soils are particularly important since they are affected by the stress state. That means that the deformation parameters are not unique constants, but may change depending on the stress path.

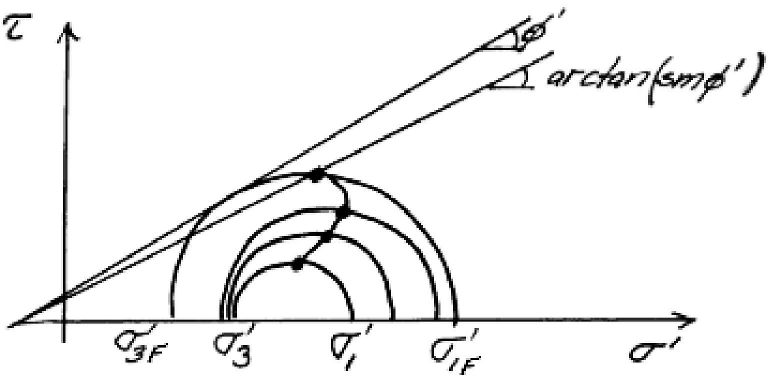

Figure 7 Stress path created from points on the Mohr-circles (Kompetenscentrum Infrastruktur).

Figure 8 below is called an s-t plot, where s is the average principal stress and t is the maximum shear stress. It shows the stress paths with respect to only two dimensions, the major and minor stress direction. The maximum shear stress can be expressed as t or t' since they will assume the same values. The stress path can be expressed in either effective (s') or total (s) stress, where the horizontal difference is the pore water pressure.

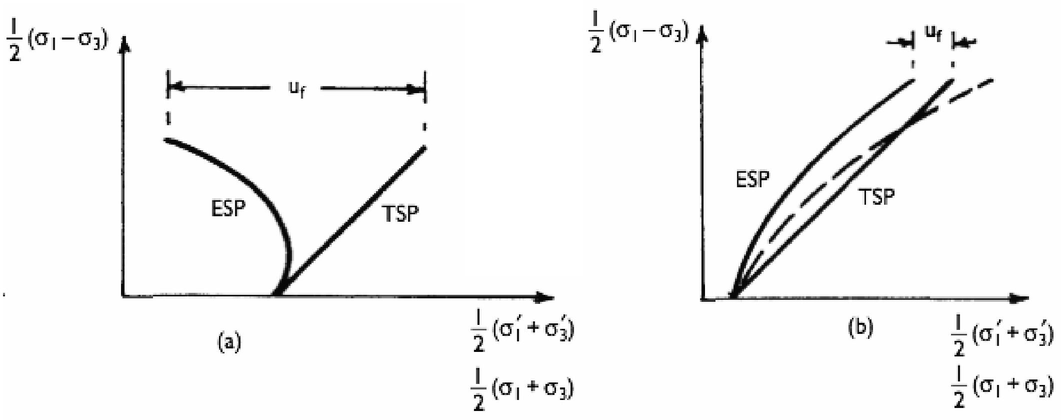

Figure 8 The total (TSP) and effective (ESP) stress paths for (a) normally consolidated and (b) over consolidated clay in undrained conditions.

There are also other ways to display the stress path; a common one being the p'-q' plot where p' is the average effective principal stress and q' is the deviator stress, see Equations 4 & 5.

$$p' = \frac{1}{3}(\sigma'_1 + 2\sigma'_3) \tag{4}$$

$$q' = (\sigma'_1 - \sigma'_3) \tag{5}$$

Since the intermediate principal stress (σ'_2) is equal to the minor principal stress (σ'_3) in a triaxial test, p' is expressed with the later one only (Craig, 2004).

To determine the shear strength and preconsolidation pressure from a triaxial test the stress path needs to be examined. The maximum shear strength (τ_f) is equal to the radius of the Mohr circle at failure (Kompetenscentrum Infrastruktur), which is the peak of the stress path. The preconsolidation pressure (σ'_c) is evaluated by drawing a 45° tangent from the stress path to the mean effective stress axis, see Figure 9.

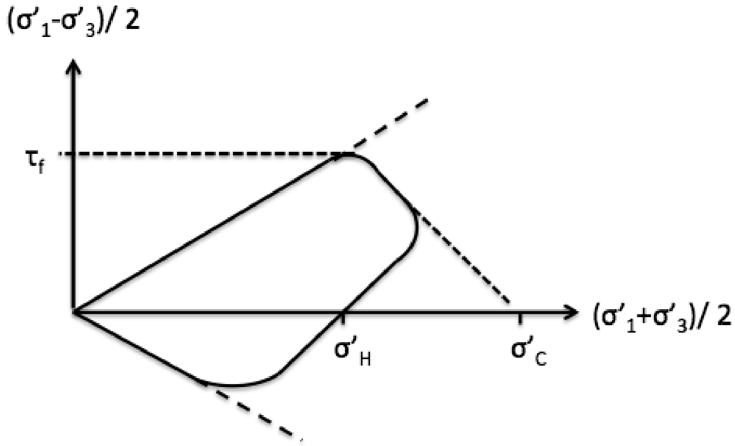

Figure 9 Evaluation of maximum shear stress and preconsolidation pressure from the failure surface.

2.4 Consistency properties and limits

One technique for classifying fine-grained soils is to determine the consistency of remolded samples (Karlsson & SGF, 1981). This approach was studied and introduced by the Swedish chemist Albert Atterberg. He studied how the consistency of the soil changed during changes in water content. The soil can, depending on its water content, exist in four different states: solid, semi-solid, plastic or liquid (see Figure 10). The water content when the soil is in the boundary between two states is called a consistency limit, e.g. plastic limit and liquid limit. According to Craig (2004), most fine-grained soils will in nature appear in the plastic state.

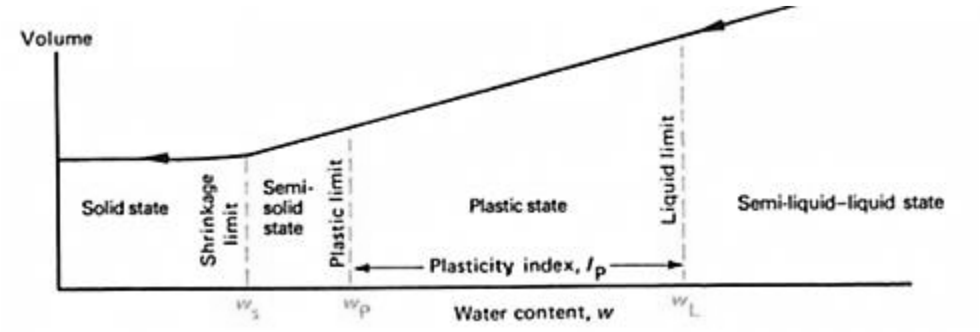

Figure 10 Consistency states and limits (from Karlsson & SGF, 1981).

In the solid state, the soil has a brittle behaviour, while being mouldable in the plastic state. In the liquid state, the soil sample will behave like a fluid and deform by its own weight. The plasticity of a soil can be measured by the plasticity index, I_P, as seen in Figure 10, and is defined as:

$$I_P = w_L - w_P \tag{6}$$

where w_L is the liquid limit and w_P is the plastic limit. Changes in water content below the shrinkage limit will not affect the volume of the sample.

The remolded consistency of the soil at its natural water content is described by the liquidity index, I_L:

$$I_L = \frac{w_N - w_P}{I_P} \tag{7}$$

where w_N is the natural water content. The liquidity index is a measure of how close the natural water content is to the liquid limit and works as an indicator of how sensitive the soil is to mechanical disturbance. For a quick clay, the value of the liquidity index is always above 1 (Larsson, 2008).

Another parameter for the classification of a soil is the sensitivity, S_t, which is defined as the ratio between the undisturbed and the remoulded shear strength, using fall cone test:

$$S_t = \frac{\tau_{fu}}{\tau_r} \tag{8}$$

As can been seen in Table 1, clays can, in regard to sensitivity, be classified as low-sensitive, medium sensitive or high-sensitive, where quick clays are a sub-group in the high-sensitive class.

Table 1 Sensitivity classification

Classification	**Sensitivity**
Low-sensitive	<10
Medium-sensitive	10-30
High-sensitive	>30

If the sensitivity of a clay exceeds 50 and the remoulded shear strength is below 0.4 kPa, it is defined as a quick clay.

2.5 Preconsolidation pressure

The strength of a soil specimen is often dependent on the load history the soil has been subjected to. The maximum vertical stress at a certain point in the sample is called the preconsolidation pressure (σ'_c) and it increases with depth (Craig, 2004). It is usually determined through oedometer tests. When loading a soil its properties change drastically at loads beyond the preconsolidation pressure, resulting in large deformations, see Figure 11. The initial elastic modulus (M_0) is much larger than the plastic modulus (M_L) after the preconsolidation pressure. According to Swedish practice the pre-consolidation pressure is determined from CRS oedometer tests according to the Sällfors method. The preconsolidation pressure is evaluated by first drawing two tangents on the oedometer curve before and after the increase in deformation. By drawing a third tangent which together with the previous two creates an isosceles triangle, the preconsolidation pressure is evaluated by drawing a vertical line from the triangles upper left corner (Larsson, 2008). The evaluation is only valid for diagram scales where the effective stress of 1 kPa equals 0.1% strain. The moduli are evaluated from the gradients of the curve, see Figure 11.

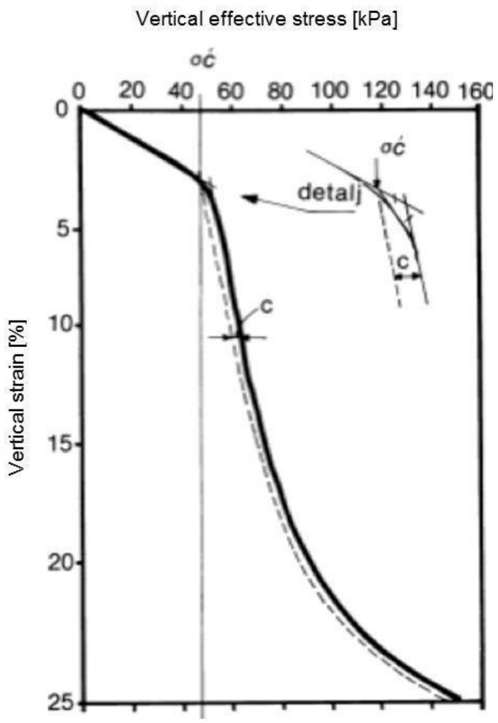

Figure 11 Evaluation of the pre-consolidation pressure from an oedometer test curve according to the Sällfors method (SGI, 2009).

The clay may also consolidate due to creep deformations, known as secondary consolidation. Creep deformation occurs during constant effective stress and is believed to be caused by rearrangement of the clay particles in the aggregates in the clay structure. According to Bjerrum (1967) the schematic picture of secondary consolidation looks as follows:

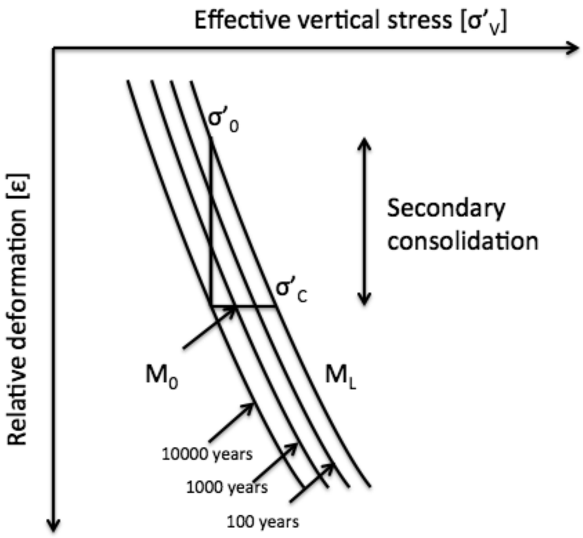

Figure 12 Schematic picture of secondary consolidation (adapted from Bjerrum, 1967).

The ratio between the in situ vertical effective stress (σ'_0) and the preconsolidation pressure is called the over consolidation ratio and is calculated as follows:

$$OCR = \frac{\sigma'_c}{\sigma'_0} \tag{9}$$

Depending on if a clay is normally (OCR = 1-1.3) or over consolidated (OCR > 1.3) the properties of the clay changes, especially its deformation behaviour.

3 Laboratory tests

3.1 CRS test

The CRS test is a single drained compression test for the elastic and plastic behavior of a clay sample. CRS, as in Constant Rate of Strain, is a type of oedometer test where the stress demand is recorded for a constant rate of deformation, usually 0.0025 mm per minute (which gives a compression of 18 % for a 24 h test cycle). Readings of vertical force, deformation and pore pressure are made every two minutes and the compression curve is plotted in a stress-strain graph with linear axes. The effective stress, σ', in the specimen can be expressed as:

$$\sigma' = \frac{P}{A} - \frac{2}{3} u_b \tag{10}$$

where P is the vertical force, A is the area on which the force is applied and u_b is the pore pressure at the undrained surface of the specimen (SIS, 1991). The laboratory procedure is described in chapter 7.3.2.

3.2 Triaxial test

One of the most useful tests for determination of soil properties is the triaxial test (Simons, Menzies & Matthews, 2002). Compared to other tests it has the advantage of control over drainage conditions and the soil specimen can be reconsolidated to in-situ stresses before testing. The triaxial test allows for many different types of testing since the user have a lot of control over the boundary conditions. Both active and passive tests can be performed, drained or undrained, and the user can control the stress conditions in the sample, including the pore pressure. The testing equipment consists of an oil filled cell in which a cylindrical sample is tested, see figure 13.

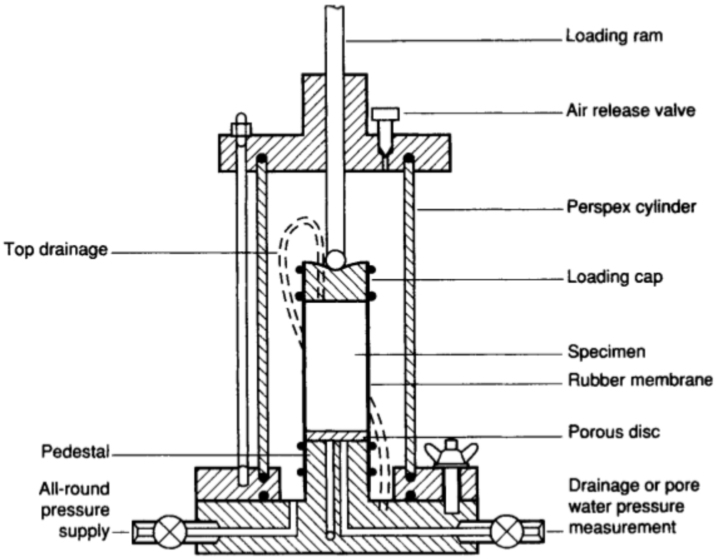

Figure 13 Triaxial apparatus (Craig, 2004)

The laboratory procedure is described in chapter 7.3.1.

3.3 Fall cone test

The fall cone test was for a long time one of the main methods for shear stress determination of clay samples in Sweden. In recent years, the results from fall cone tests have been interpreted as indications of the shear strength. As seen in Figure 14, the fall cone apparatus is a stand with an arm at which a fall cone can be attached. The cone is lowered to the clay surface from where the cone is released. The penetration depth, *i*, is read on a scale attached to the arm. Depending on the properties of the sample, cones with different mass and angle can be used. An indication of the undrained shear strength and remoulded shear strength as well as the sensitivity can be determined with fall cone tests. Earlier experience is that the results are time-dependent, hence this should be considered in the evaluation.

Figure 14 *The fall cone apparatus arrangement for the determination of undrained shear strength and remoulded shear strength respectively.*

A semi-empirical formula, commonly used in Sweden, for the relation between cone penetration depth and shear strength was presented by Hansbo as:

$$\tau = Kg\frac{m}{i^2} \tag{11}$$

where K is a parameter, essentially depending on the angle of the cone, g is gravity, m is the mass of the cone and i is the cone penetration depth (Wiesel, Hansbo & Broms, 1985). The recommendations for Swedish clays are to set the parameter K to 1.0 for cones with an angle of 30 degrees and 0.25 for cones with a 60 degree angle.

3.4 Water content

To determine the water content, w, a sample is weighted and dried at about 105 degrees for at least 24 hours before it is weighted a second time. The definition of the water content is:

$$w = \frac{m_w}{m_s} \tag{12}$$

where m_w is the mass of water and m_s is the mass of solid matter. The water content is usually given in percent.

3.5 Density

The density, ρ, is the ratio between the mass and the volume of a given amount of soil (see equation 13) and is determined by weighing a soil sample of a known volume.

$$\rho = \frac{m_s + m_w}{V} \tag{13}$$

where V is the volume.

3.6 Liquid limit

The liquid limit, w_L, of a sample can be determined by the Casagrande cup method (see e.g. Karlsson & SGF, 1981). However, the practice in Sweden is to use the fall cone method. A soil is said to be at its liquid limit when the penetration, i, of a 60 g cone with the angle of 60 degrees is 10 mm in a completely remoulded sample (Karlsson & SGI, 1981). In order to be able to determine the liquid limit without changing the water content of a sample, an empirical relation has been developed that can be used for penetration depths in the range from 7.0 mm to 14.9 mm (see equation 14).

$$w_L = M \cdot w_i + N \tag{14}$$

where w_i is the water content of a sample with cone penetration depth i and M and N are constants (see e.g. Karlsson & SGI, 1981).

3.7 Plastic limit

The plasticity of a clay sample can be estimated by rolling the material into a thin thread. If the thread is not broken, the soil is plastic (otherwise non-plastic). In order to determine the plastic limit, w_P, a sample of about 20 g is divided in two pieces. The sample is remoulded and then rolled on an absorbing paper placed on a hard surface (or on a glass plate, as is the common practice internationally). For Swedish clays, the sample usually needs to be dried, for example on a gypsum plate, before it is possible to roll it. The plastic limit is reached when the rolled thread crumble at a thickness of 3 mm. The procedure is then repeated for both pieces and the water content is determined (Karlsson & SGF, 1981).

4 Disturbance of soil samples

A great deal of the disturbance of a soil sample occurs in conjunction with the actual sampling. In addition to the disturbances during sampling, the samples will also be subjected to disturbance during transportation and storage, as well as during test preparation in the laboratory. Some types of disturbance will occur instantly, while other types will increase over time.

Even though some disturbance is inevitable, the rate of disturbance can be improved in many cases if the mechanisms and processes of disturbance are identified and used to improve sampling and laboratory tools. Knowledge of the staff performing sampling and testing is also very important (Clayton, Matthews & Simons, 1995). The importance of careful and correct sampling and handling of the samples has also been stressed by Magnusson, Sällfors and Larsson (1989) in their benchmark study of CRS oedometer tests, on the variations in results when sampling and testing were performed by different companies and laboratories.

Some of the major factors in terms of sample disturbance are discussed in the following section. Other factors like storage temperature and humidity, vibrations, shocks etc. will not be discussed in detail.

4.1 Mechanical deformation

Mechanical deformation of the sample occurs mainly during sampling and affects the soil both inside and outside the sampler due to the breakdown of the soil structure.

In general, two different methods are used when taking the samplers to the sampling depth; sampling with and without pre-augering. Pre-augered holes are usually stabilized with water or drilling fluid in order to decrease the stress changes in the ground. By carefully removing the soil above the sampling depth, the disturbance of the soil below the borehole floor can be kept low (Hvorslev, 1949). This procedure is usually used for large diameter samplers.

Piston samplers with a smaller diameter are usually driven to the required depth without pre-augering. This method is not as time-consuming and demanding as the pre-augering method. The hole is in this case formed by displacing the soil around the sampler. The soil beneath and around the casing will be subjected to compaction, remoulding and displacement (see Figure 15). According to Hvorslev (1949), the soil to a depth of up to three sampler diameters below the sampler may be disturbed when the displacement method is used. Shear distortions during displacement sampling causes changes in effective stress and a break-down of the surrounding soil (Clayton, Matthews & Simons, 1995).

During sampling, the friction between the soil and the inside of the sampler will destroy the structure of the outer zone of the sample. How far into the sample the disturbed zone will reach depends on the type of clay sampled. According to Bjerrum (1973), clays with low plasticity and low sensitivity are sensitive to frictional disturbance. An example of convex curvature due to inside friction can be seen in Figure 16.

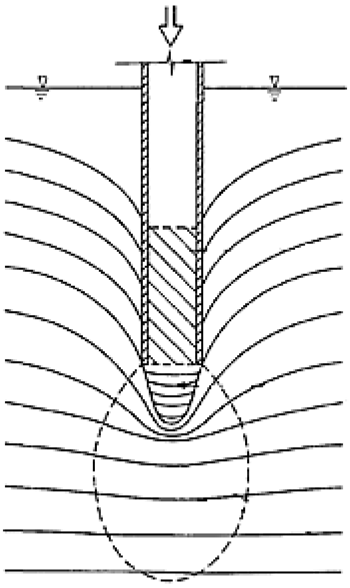

Figure 15 Disturbance of soil beneath and around a sampler (from Clayton, Matthews & Simons, 1995).

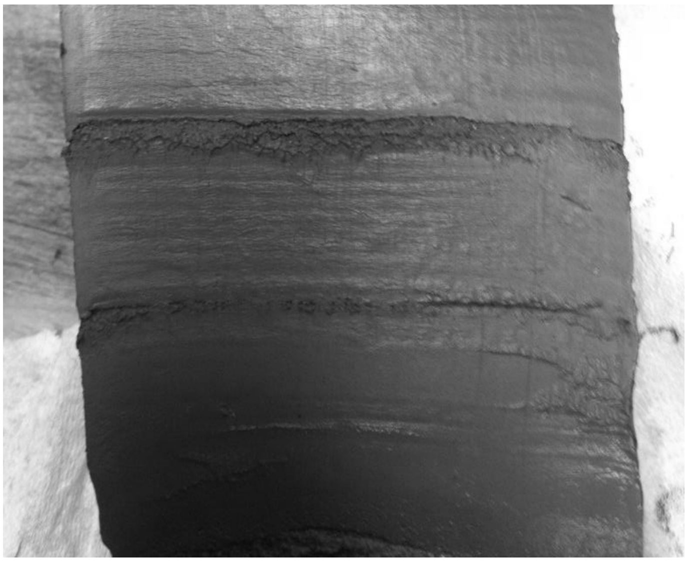

Figure 16 Convex curvature at the margin of a varved clay from Värmland, Sweden.

4.2 Changes in stress conditions

During the sampling process the stress conditions in the sample will change from in-situ conditions, as can be seen in Figure 17. During all parts of the process, from drilling to prepared sample, permanent disturbances can occur. The sample will be reconsolidated to in-situ stress conditions during triaxial testing but not during oedometer tests. The disturbances will affect the stress paths acquired during testing.

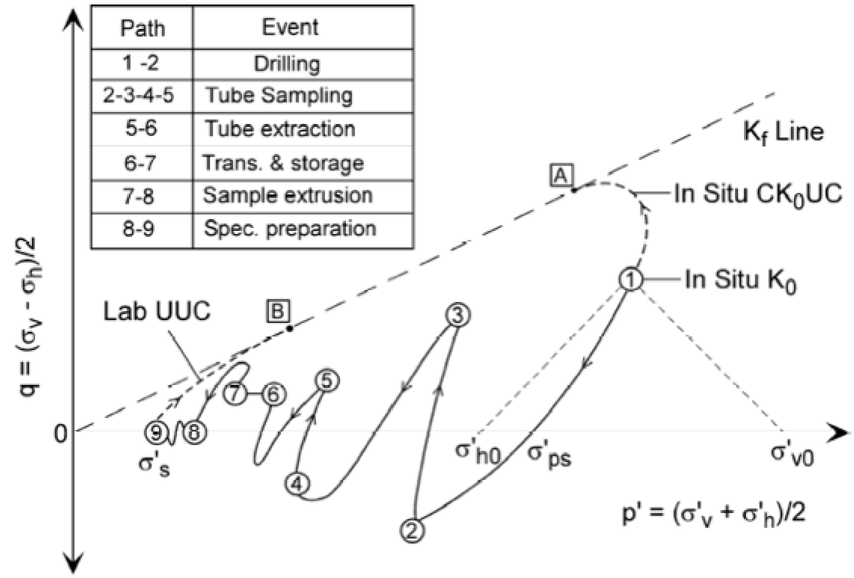

Figure 17 Change in stress conditions during the sampling process (Ladd & DeGroot, 2003).

According to Hvorslev (1949), the largest stress reduction will occur when the sampler is withdrawn from the ground, due to the vacuum that can form in the borehole beneath the sampler. The stress reduction is further amplified by the tensile stress that occurs when the sample is dislodged from the soil beneath if the sampler does not have any closure equipment. A stress reduction will cause the sample to swell. In order to lower the risk of high negative air pressures below the sampler, the pull up speed should be very slow and steady to allow the hole formed to close as the sampler is lifted (SGF, 2009).

After sampling, the total vertical stresses in the sample will be close to zero. The horizontal stresses will be reduced in case of the use of inside clearance, but the sampling tubes will reduce the swelling to some extent and some horizontal stresses will be retained. Over time, the pore water pressure will dissipate, thus causing the effective stress to approach the total stress.

Stress relief will not only occur in the samples but also at the borehole floor and walls. Especially in soft or low strength soils, a large reduction in the stresses can result in base heave and caving of the walls (Clayton, Matthews & Simons, 1995).

4.3 Negative pore pressure

Due to the stress relief and the low permeability of clay, the pore water in a sample will develop a negative pressure during sampling. Studies in the subject (Bjerrum, 1973 and Skempton & Sowa, 1963) have shown that the theoretical value of the negative pore pressure can be estimated to be in the range of 40 to 60 per cent of the in situ vertical effective stress (for a K_0 of 0.5). However, measurements of real samples have given much lower values, typically 20% for good quality samples.

Weak grain structure, due to disturbance, lowers the ability to hold a negative pore pressure. Measurements of the suction can therefore be used as indicators of sample disturbance, provided that the sample is saturated, though attention should be given to the fact that dehydrated samples can show high suction values too, which is not a sign of low disturbance.

4.4 Redistribution of pore water

Induced strains at the periphery of the sample will destroy the structure and remould the clay (Simons, Menzies & Matthews, 2002). In normally or slightly over consolidated clays this will increase the pore pressure due to a reduction of the void ratio. Pore pressure gradients in the sample will cause the pore water to migrate. Over time the pressure will be equalized and the pore pressure will be the same in the middle of the sample and in its periphery. This in turn causes the water content in the sample to be less at the periphery than in the middle of the sample (see Figure 18).

In stiffer, over consolidated clays the behaviour at the periphery will be different. Due to shearing, the clay wants to dilate and negative pore pressure will be produced during sampling. Over time the pressure difference will be equalized when water from the middle of the sample will be drawn out into the periphery of the sample (see Figure 19).

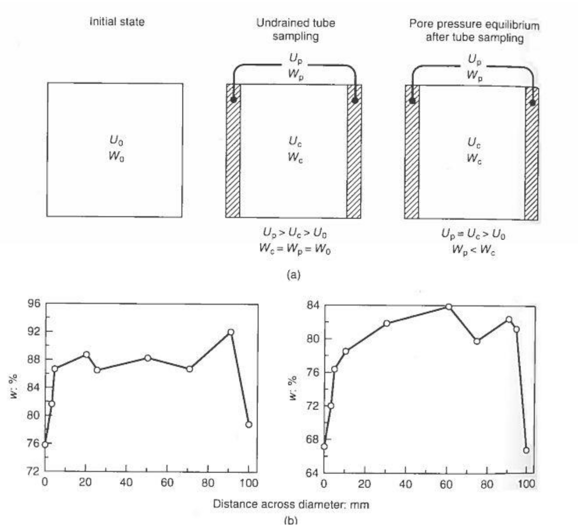

Figure 18 Pore pressure and water content distributions in a normally consolidated clay sample (Simons, Menzies and Matthews, 2002).

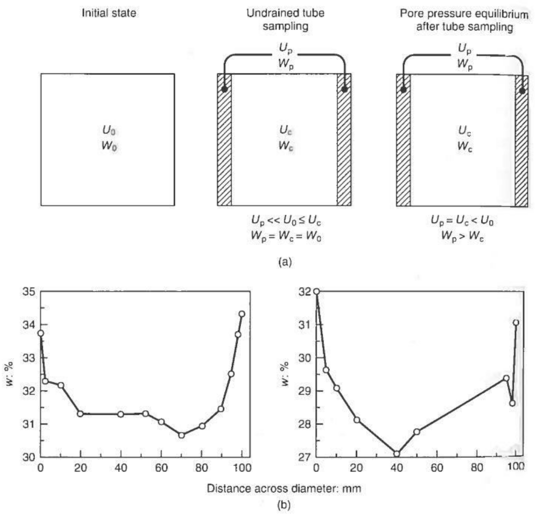

Figure 19 Pore pressure and water content distributions in an over-consolidated clay sample (Simons, Menzies and Matthews, 2002).

When sample tubes and lids are not being watertight, the samples can be subjected to dehydration at the surface, leading to changes in the distribution of pore water. Such changes will lead to changes in average effective stress and consequently changes in strength and compressibility (Clayton, Matthews & Simons, 1995).

4.5 Chemical changes

During sampling and storage the soil sample is subjected to a different environment than in the ground. Samples in contact with air will be subjected to oxidation, which may change the properties of the sample. Other chemical processes that may have an impact on the sample is reactions with the sampling tubes or laboratory equipment which may lead to changes in chemical bonds or chemical content of the pore water.

4.6 Evaluation methods of sample disturbance

Different methods have been suggested for the evaluation of sample disturbance and can be divided into non-destructive and destructive assessments. As the names indicate, the non-destructive methods have the advantage of keeping the samples intact while the destructive evaluations usually can be done in conjunction with laboratory tests, like CRS oedometer tests, CAUC test and DSS tests. A few of the evaluation methods are introduced below.

4.6.1 Non-destructive assessments

So called bender elements can be used to obtain the shear wave velocity in a sample which can be compared to in situ values. The bender element method uses a transmitter to send shear waves through a sample. Using a receiver at the opposite end of the sample the travel time (t) through the sample can be recorded and knowing the length of the sample (L) the shear wave velocity (V_s) is calculated using equation 15 (Haegeman & Piriyakul, 2005).

$$V_s = \frac{L}{t} \tag{15}$$

The bender element system can be placed inside a triaxial apparatus in order for it to measure the wave speed after the sample has been reconsolidated. Knowing the shear wave velocity, the initial shear modulus (G_0) can be calculated using the density of the soil (ρ).

$$G_0 = \rho V_s^2 \tag{16}$$

The in-situ values of shear wave velocity are obtained using a probe (e.g. seismic cone) that is pushed into the ground to record a shear wave that is generated at the ground surface or in another borehole (Brouwer, 2007). The results from the laboratory are compared to the ones from field measurements and the ratio between them is used as a quality measurement.

4.6.2 Destructive assessments

The method of using measurements of volumetric strain at reconsolidation (ε_{vo}) for the evaluation of sample disturbance, as described in Larsson et al (2007), is frequently used in Sweden. The volumetric strain can be evaluated from oedometer or triaxial tests. The sample quality categories are recalculated from Lunne's criterion of void ratio (Lunne, Berre & Strandvik, 1999). If the specific gravity, G_s, is assumed to be 2.65, the void ratio can be recalculated to volumetric strain and plotted to the natural water content, as seen in e.g. Figure 52 (Larsson, Åhnberg & Löfroth, 2012). The method is based on the theory that the sample volume change will be larger if the grain structure is weakened by disturbance. Experience have shown that the method should be used with caution since old and dried out samples can give very good results in the evaluation.

The shape of stress-strain curves is usually a good indicator of sample disturbance and can be evaluated for both CRS tests and CAUC tests. As seen in Figure 20, a less disturbed sample has a distinct appearance with a low strain at failure, while a highly disturbed sample has a more gentle shape and a higher strain at failure (Kallstenius, 1958).

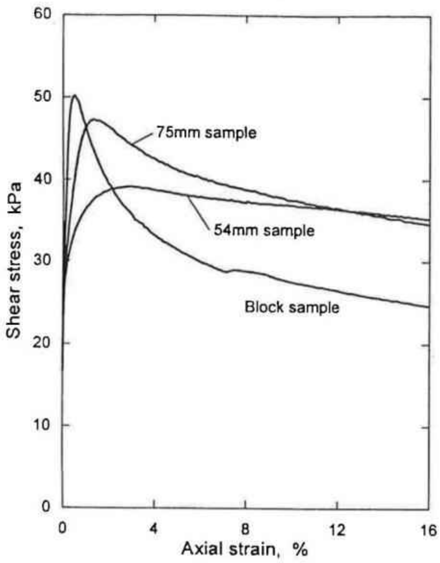

Figure 20 Different shapes of the stress-strain curves of CAUC tests from Norwegian samples (Lunne, Berre & Strandvik, 1999).

A large scatter in test results may indicate disturbance of the samples, since the results from a disturbed sample will differ more to the in-situ state than the results from an undisturbed sample. Considerations have to be taken, however, to the fact that highly inhomogeneous soils can show a decrease in scatter when being disturbed (Kallstenius, 1958).

5 Samplers

The need for undisturbed soil samples of high quality have been increasing over the last century. Demands on qualitative inputs for soil models and geotechnical judgments have resulted in the development of samplers that meet the requirements of undisturbed sampling. The ability to take undisturbed soil samples have been of interest in areas with difficult soil conditions, not least in Scandinavia because of its highly sensitive marine clays.

Even though it is called undisturbed sampling, the correct definition may rather be ‘sufficiently little disturbance for the actual application’, since it is impossible to take samples that are truly undisturbed (Kallstenius, 1958). The demands on the quality of the samples will differ between each case and generally economic aspects will be the limiting factor.

5.1 History of undisturbed soil sampling

According to Kallstenius (1963), it has been found that relatively high quality soil sampling tools were used in agriculture as early as 1750. In Sweden, the development of undisturbed sampling for soil strength tests were started at the turn of the twentieth century and Statens Järnvägar, the Swedish state railways, were developing the first piston clay samplers in the 1920s (Swedish Committee on Piston Sampling, 1961). Over the next decades, several other models of piston samplers were manufactured, each with different improvements in design and in decreasing amounts of sample disturbance.

In 1949, Hvorslev published the results of his extensive studies on soil sampling and sample disturbance in the groundbreaking report ‘Subsurface exploration and sampling of soils for engineering purposes’ (Hvorslev, 1949). In Sweden, many of his recommendations were implemented in new piston samplers in the following years. However, since there were a wide variety of samplers on the market, an even quality of the samples was hard to determine and comparisons of different samplers performed by SGI showed that the sample quality differed much (Swedish Committee on Piston Sampling, 1961). The need for a standard sampler led to the appointment of a committee that should investigate the subject and the main goal for the committee was to suggest a standard piston sampler that took high quality samples at a low cost, i.e. with a fast and easy sampling procedure. The study resulted in the report ‘Standard piston sampling’ (Swedish Committee on Piston Sampling, 1961) and the introduction of the new Swedish standard piston sampler, St I.

At the same time as the St I sampler was developed, one of the earlier sampler models from the engineering company Borros AB was changed to adapt to the directions given by the committee (Borros AB, 1964). The model was called St II and has in recent years become the dominantly model used.

5.2 Sampler design

The design of the samplers is very important when it comes to sample disturbance. Some of the key factors for a well-functioning design are presented below.

5.2.1 Area ratio

The area ratio is a ratio between the internal and the external diameter of the sampler (see equation 17).

$$area\,ratio = \frac{{D_2}^2 - {D_1}^2}{{D_2}^2} \tag{17}$$

where D_2 is the external diameter and D_1 is the internal diameter of the cutting shoe (see Figure 21).

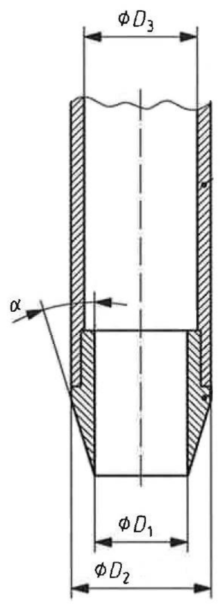

Figure 21 Definition of sampler diameters (SIS, 2006).

A larger area ratio will increase remoulding and disturbance during sampling and increase the risk of excess soil being forced into the samples (Hvorslev, 1949). However, the importance of the area ratio is reduced if the cutting edge is designed to minimize disturbance.

5.2.2 Layout of cutting edge

It has been pointed out in several studies that the design of the cutting edge of the sampler is of great importance in terms of sample disturbance. Simons, Menzies and Matthews (2002) have studied the relation between disturbance and cutting shoe design for five different samplers (see Figure 22). The investigation shows that a sharper edge causes less disturbance than a blunt edge. Figure 23 presents how the axial strain at the centerline of the samplers is changing during sampling, depending on the cutting shoe design. According to the figure, the sample will be subjected to increasing compression when approaching the cutting edge. Once inside the sampler, the stress is reduced causing a negative strain.

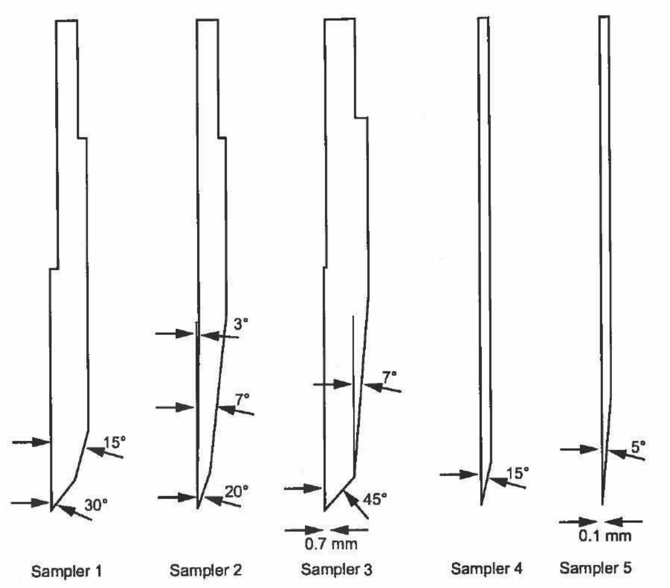

Figure 22   Different cutting shoe designs (Simons, Menzies & Matthews, 2002).

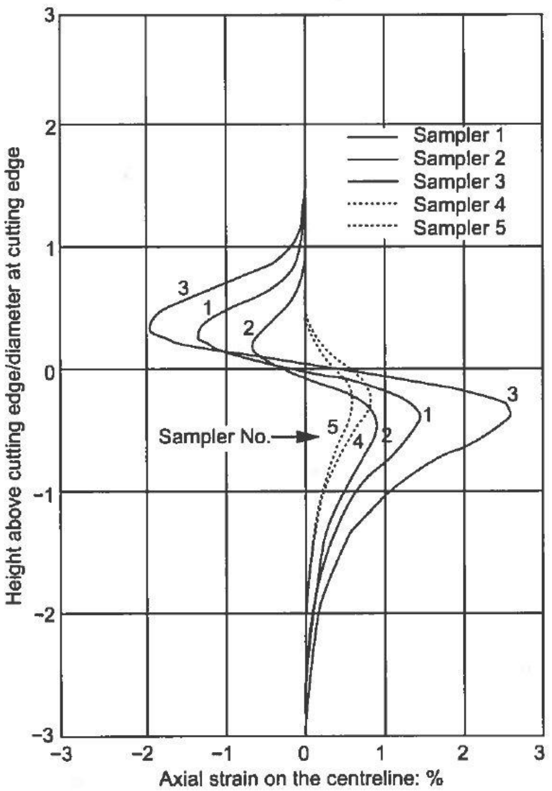

Figure 23   Axial strain in the soil during sampling for different sampler designs (Simons, Menzies & Matthews, 2002).

5.2.3 Wall friction and inside clearance

When the soil sample is pushed through the sampler, the surface will be affected by frictional forces from the inner walls of the sampler. In case of large inside frictions, the sample quality will be heavily influenced and in severe cases the sampler can get jammed. One of the methods that can be used to reduce the wall friction is to apply an inside clearance to the samplers (Clayton, Matthews & Simons, 1995). The inside clearance is the relation between the cutting edge internal diameter and the internal diameter of the sample tubes as seen in equation 18 below:

$$Inside\ clearance = \frac{D_3 - D_1}{D_1} \tag{18}$$

where D_3 is the internal diameter of the sample tubes (see Figure 21).

The inside clearance will decrease the wall friction, allow radial swelling and increase the radial stress relief in the samples. According to Kallstenius (1958) and Clayton, Matthews and Simons (1995) the inside clearance will have a negative effect on the sample quality due to the stress changes. Some argue, though, that a small inside clearance is necessary in order to avoid the differences due to manufacturing accuracy causing the risk of the tubes being smaller than the clay samples (Larsson, 1981).

5.2.4 Sampler design recommendations

Hvorslev (1949) states that the area ratio should be as small as possible as long as the structural strength of the sampler is sufficient. He recommends area ratios below 10 %, while the Swedish Standards (SIS, 2006) state that values between 15 % and 25 % may be used if it can be shown that the quality demands are met, though the lower value is the general limit.

According to the Swedish Standards (SIS, 2006), the taper angle, α (see Figure 21), should not exceed 5 degrees. With an angle of 5 degrees, the disturbance is kept low while maintaining a fairly robust and durable design. A larger taper angle, commonly 45 degrees, is usually applied to the very edge of the cutting shoe in order to make the edge more resistant to damage.

The optimum value of the inside clearance varies depending on sampler design and soil conditions. Practical experience from early studies has shown that values of 0.5 to 1.0 per cent can be reasonable for Swedish clays. Experiences with large diameter samplers in Norway, however, show that the inside clearance can be reduced and even removed (Larsson, 1981). In the Swedish Standards (SIS, 2006), the value should be less than 0.5 %. Hvorlev (1949) recommend an inside clearance of 0.75-1.5 %.

5.3 Different types of samplers

5.3.1 The Swedish standard piston sampler

The standard sampler in Sweden is called St II and is a piston sampler that is driven to the desired sampling depth by the displacement method. By rotating the rod connected to the sampler, the cutting shoe is pushed downwards causing the soil to enter the sampler. The friction along the inside walls of the sampler holds the samples in place when dislodged from the soil beneath. The samples are obtained in three sampling tubes which are pushed out by the piston. An outline of the sampler can be seen in Figure 24.

The Swedish standard piston sampler has proven to be a robust and user-friendly sampler that generally takes samples of sufficiently good quality in soft clays (Larsson, 1981). However, some limitations have been found over the years. In some conditions, especially in highly sensitive soils, i.e. quick clay, it has been hard to retrieve samples of satisfactory quality. In addition, shells, small stones or other objects can easily destroy large parts of the samples due to the small diameter.

As can be seen in Table 2, the cutting edge design satisfies the recommendations while the area ratio is much larger than the recommended values.

Figure 24 The Swedish standard piston sampler (Borros AB, 1964).

A full description of the standard Swedish sampler and how it is used can be found in the original manual (Borros AB, 1964).

5.3.2 The 60 mm piston sampler

The new piston sampler, which takes samples with the diameter of 60 mm, is a widened version of the Swedish standard piston sampler described above. The sampling method as well as the transportation and storage routine is the same.

One important advantage of sampling with this sampler compared to block sampling and large-diameter sampling is the cost effectiveness and the easy usage. Apart from slightly heavier and more bulky equipment and samples, the difference to the standard Swedish sampler is insignificant during sampling.

The ratio between the inside surface area of the sample cylinder and the area of the sampler end is changed for the new sampler, which means that less friction is probably mobilized compared to the force that is needed to dislocate the sample from the soil beneath. This may increase the disturbance of the samples. On the other hand, the area ratio is decreased which should have a positive impact on the sample quality.

During the testing of the samples, shear surfaces have been recorded in many of the upper tubes (see chapter 11.4.2), and have later been found also in several 50 mm samples. Similar failure zones have been identified by Hvorslev (1949).

5.3.3 Block sampling and large-diameter samplers

During the sampling of block samples, the clay is cut out in large pieces directly in the excavation pit and trimmed with a piano wire and the sample is then encased in wax and textile. The stress conditions are still changed, but other disturbance factors, like shear distortions, are decreased compared to standard piston sampling. Since the samples are large, pore water migration is less likely to have affected the core of the samples when tested. However, the specimens have to be cut and trimmed in the laboratory which may affect the disturbance. Blocks are preferred when several tests have to be performed on identical samples, as they may be cut out from the same block.

In order to retrieve samples of the quality of a block sample, a few large diameter samples have been developed, the most well-known are probably the Sherbrook sampler and the Laval sampler. Recently, a 200 mm sampler has been developed by SGI (Löfroth, 2012 and Larsson, Åhnberg & Löfroth, 2012)

Even though the sample quality is generally increased with these samplers compared to the standard piston samplers, the sampling is more expensive and time-consuming and is usually not feasible for standard projects.

5.4 Investigations of improving soil samplers

Over the years, several studies have been published on the topics of the evaluation and improvement of different soil samplers. The effect of changes in design parameters like sampler diameter, cutting shoe angles, inside clearance and area ratios has been investigated.

During the development of the standard Swedish piston sampler, a sampler with the diameter of 50 mm was compared to one with the diameter of 60.5 mm (Swedish Committee on Piston Sampling, 1961). The results showed a slight improvement in sample quality when the diameter was increased. A larger scatter was determined in the fall cone test results for smaller diameter samples compared to larger. However, the improvements of a larger diameter sampler were not found to be large enough to justify the heavier handling of samplers and samples. Similar results were found by Kallstenius (1958). He concludes that the improvements due to increased diameters are detectable, but very small.

In the late 1960s and early 1970s, reports from studies in Norway and Canada indicated a significant improvement in sample quality when increasing the sampler diameter and SGI started a new investigation of the Swedish standard sampler (Larsson, 1981). A new 124 mm sampler was produced and compared to the standard sampler, a 95 mm sampler from Norway and the 127 mm Osterberg sampler. No differences were determined in compression properties and shear strength. Small differences, however, were found in the elastic modulus during triaxial tests that favoured the samples of larger diameter.

Around a decade later, the Laval and Sherbrook large diameter samplers were developed in Canada and reported to take samples in the same quality class as by block sampling. Comparisons with pistons samples indicated relatively large improvements with the new samplers. A Laval sampler was borrowed and tested by SGI in 1981 (Larsson, 1981). Oedometer tests showed a 1.5 to 2 times higher M_0 for the Laval samples compared to standard samples. Both active and passive undrained triaxial tests gave somewhat lower shear strengths (a decrease by 2-3 kPa) and a lower strain before failure for the Laval samples. Results from CRS tests showed slightly higher values on both σ'_c and M_L for the standard samples. In conclusion, the study states that the Laval sampler took samples in parity with the best standard samples, but not substantially better.

The new SGI 200 mm large diameter sampler, mentioned above, is compared to the standard piston sampler by Löfroth (2012). The aim of the development of the new sampler was to obtain good quality samples even in clays which are very easily disturbed. CRS tests on highly sensitive clay showed advantages of the new 200 mm sampler in form of higher preconsolidation pressure, higher uniformity of the stress-strain curves and lower strain before failure. The modulus M_L was slightly higher for the standard samples. Corresponding triaxial tests gave higher shear strengths and somewhat lower strains before failure for the large diameter samples while differences in the direct simple shear tests were insignificant. Similar results but with smaller differences were seen in samples with normal sensitivity.

In Norway, several studies of sampler comparisons have been published. Lunne, Berre and Strandvik (1999) compared the Sherbrook sampler with the 54 mm Norwegian standard piston sampler and a 75 mm Japanese piston sampler. The Sherbrook samples were found to be superior to the piston samples, with greatest differences found in sensitive, low plastic clays. Volumetric strain was consistently higher for the piston samplers than the Sherbrook, the results from the 75 mm sampler indicating a lower disturbance than the 54 mm sampler.

Hagberg, Long and El Hadj (2007) investigated the difference between the standard Norwegian 54 mm sampler and the Geonor 76 mm sampler. Parallel 54 mm and 76 mm samples were taken in uniform marine clays at two different sites in the Oslo region, Norway. Index tests showed no difference in sample quality between the different samplers, while the curves from the triaxial tests might have indicated a slightly lower disturbance of the 76 mm samples, e.g. somewhat more distinct failure at a lower strain. The CRS oedometer tests show only insignificant differences and almost all tests are found in the “poor” or “very poor” category in the evaluation of volumetric strain at reconsolidation.

The documentation of a project that compared the NGI 54 mm sampler to the Sherbrook sampler is presented by Long, El Hadj and Hagberg (2009). The 54 mm samples were taken and processed by three different companies in order to determine the differences in sample quality caused by different sampling and laboratory routines. Differences in sample quality of the 54 mm samples could be found between the companies and were found to be due to relatively small variations in the sampling operation. The results of the comparison between the different samplers showed a significant quality increase for the Sherbrook sampler. The CRS elastic modulus of 54 mm samples were only 50 % of the large samples. In the triaxial tests, the stress-strain curves from Sherbrook samples showed clear peaks and in general a more distinct behaviour.

Table 2 Dimensions and ratios for some of the different samplers mentioned above.

Samplers	**Length [mm]**	**Taper angle**	**D_2 [mm]**	**D_1 [mm]**	**D_3 [mm]**	**Area ratio [%]**	**Inside clearance [%]**
st II, 50 mm	700	5°	60.0	50.0	50.2	31	0.4
new 60 mm	700	5°	70.0	60.25	60.5	26	0.4
NGI, 54 mm	800	5°	64	54	54.3	44	0.6
NGI, 95 mm	1000	16°	101.6	96.3	96.6	11	0.3-1.3
Geonor, 76 mm	800	15°	80	76	76	11	0
Japanese, 75 mm	800	6°	78	75	75	4.0	0

6 Test sites and laboratories

Two test sites were chosen for the preliminary tests; ‘Regionens Hus’ in central Göteborg and ‘Lerum Centrum’ in Lerum municipality (see Figure 25). Additional tests were in a later stage carried out on samples from Nödinge. The laboratory tests were performed at two different laboratories: the Geo Laboratory at CTH in Göteborg and the Bohusgeo Laboratory in Uddevalla. More information about the laboratory tests are found in chapter 7.3.

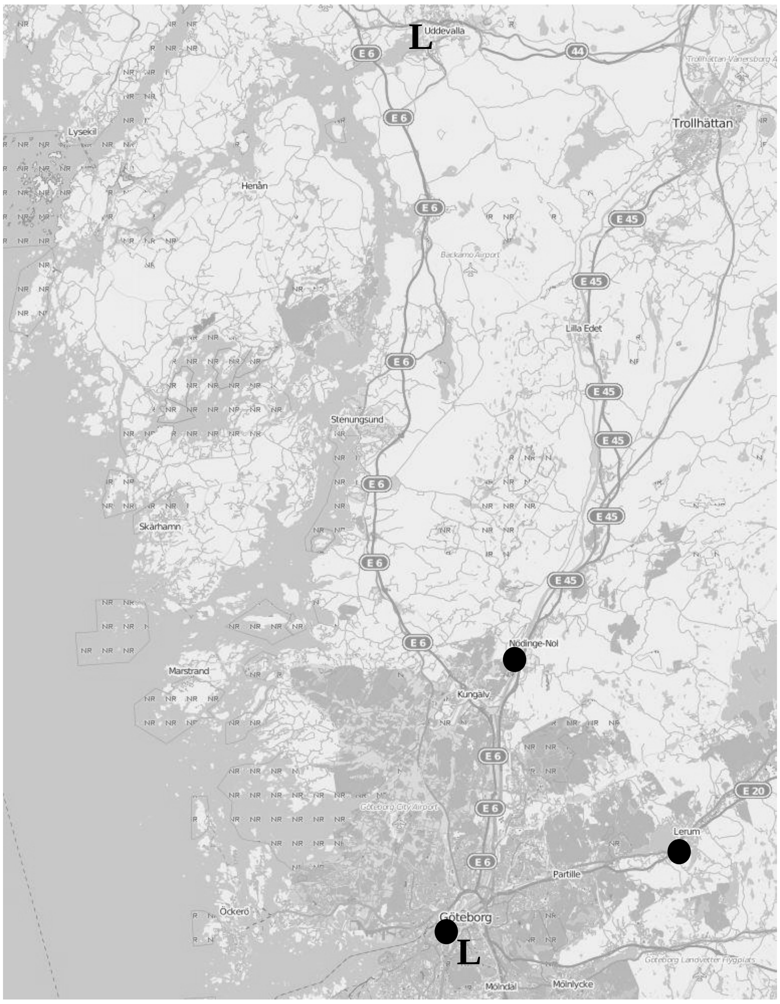

Figure 25 Map of the investigated region with test sites and laboratories marked (from OpenStreetMaps).

6.1 Lerum Centrum

Lerum is a town situated around 25 km east of Göteborg. The river Säveån passes through the town and the river valley is filled with deep layers of clay of a thickness of around 20-30 m. The sensitivity of the clay is very high and the experience of earlier sampling shows that the samples are easily disturbed. Lerum Centrum is the center of the town and is situated close to the river.

The samples were taken on a flat area on the edge of the riverbank of Säveån (see Figure 26 and Figure 27).

CRS-tests were performed on samples from 5, 10, 12, 16 and 18 meters below ground level. Both the 60 mm sampler and the standard 50 mm sampler were used on each level.

Figure 26 Lerum Centrum test site with Säveån River in the background.

Figure 27 Säveån River and slope at Lerum Centrum test site.

6.2 Regionens Hus, Göteborg

Regionens Hus is situated in the center of Göteborg, close to the Central Station. Since the area is under development, extensive soil investigations have already been done prior to this study. Very thick layers of clay are overlaid by fill materials and the clay is still consolidating because of the load from the fills. The clay is fairly homogenous and has been deposited in a marine environment. Earlier CPT soundings show that the clay deposits are found down to a depth of around 90 m below surface (Bohusgeo, 2012).

The sampling site was chosen to be in the middle of a parking lot close to the new Regionens Hus building (see Figure 28 and Figure 29). For this study, undisturbed samples with the diameter of 60 mm were taken at six levels: 10, 11, 15, 20, 25 and 30 m below surface. Triaxial and CRS tests were performed on the samples in the Chalmers Geo Laboratory. The results are compared to earlier corresponding tests performed on samples taken with the standard 50 mm piston sampler.

Figure 28 Regionens Hus test site

Figure 29 Regionens Hus test site

6.3 Nödinge

Since the study would benefit from additional test results, new clay samples were retrieved from a test site in Nödinge, about 25 km north of Göteborg, by the east river bank of Göta Älv river. The sampling was performed in the middle of a grazing area which had a low gradient slope towards Göta Älv river (see Figure 30). The site has been used for several research projects at CTH, hence laboratory data is available for comparisons and analyses.

At Nödinge test site, the soil consists of deep layers of soft marine clay on top of a few meters of frictional material. At a depth of 12-13 m the clay turns from gray to sulphide coloured.

Parallel samples of both 50 mm and 60 mm were taken on 5 and 12.5 m below surface. Triaxial tests and CRS tests were performed on the samples, as well as index tests.

Figure 30 Nödinge test site. Note the Göta Älv river to the left.

6.4 Laboratories

The laboratory tests on the samples from Regionens Hus were performed at the Geo Laboratory at Chalmers, Göteborg, while the tests on the samples from Lerum Centrum were done at Bohusgeo Laboratory in Uddevalla (see Figure 25). The samples from Nödinge were divided on the two laboratories. Nödinge CRS tests were performed at Bohusgeo and the triaxial tests at Chalmers.

7 Execution

7.1 Field Sampling

The samples for the study were taken by two different machine operators with similar, but not identical, drill rigs. One operator took the samples at Lerum C and Regionens Hus and the other one took the samples in Nödinge. The same 60 mm, but different 50 mm piston samplers were used at the different locations. The sampling programs at the different locations are presented in table 3, followed by a short description of the sampling procedure.

Table 3 Sampling programs

Test site	**Borehole nr**	**Date**	**Sample diameter [mm]**	**Sampling depth [m below surface]**
Regionens Hus	9	2013-02-18	60	10, 11, 15, 20, 25, 30
Lerum Centrum	2	2013-02-14	50	5, 10, 12, 16, 18
	2	2013-02-13	60	5, 10, 12, 16, 18
Nödinge	P1	2013-04-15	60	5, 12.5
	P2	2013-04-15	50	5, 12.5
	P3	2013-04-15	60	5, 12.5
	P4	2013-04-15	60	12.5
	P5	2013-04-15	50	12.5

The top most layers of soil profiles usually consists of other soils than clay, in urban areas usually some type of fill material and in undeveloped areas organic topsoil. Since the sampler is pushed down through the layers of clay any fill, friction soil or asphalt needs to be removed first, which is done through drilling. The sampler is pushed down to 35 cm above the sample depth and the inner tube is then extended down to 35 cm below the sample level. This way the sample depth will be in the middle of the tube. After a few minutes the extended sampler is withdrawn to the surface at a slow speed (SGF, 2009). Depending on the clay the waiting time varies which is needed in order for the clay to stick to the inside of the tube due to its thixotropic properties. At the surface the inner tube is retracted thus pushing the sample tubes out. A plastic film and a cap are then put on each side of the sample tubes to prevent pore water from evaporating and the sample from oxidizing.

7.1.1 Lerum C

The sampling at Lerum C was performed on the 13th and 14th of February, 2013. The first day 60 mm samples were taken and the next 50 mm ones. All samples from

Lerum C were taken to the laboratory at Bohusgeo for index and CRS testing. Due to temperatures below freezing the samples were stored either in a car or a heat source was placed in the storage box. At this location there were some problems with stones in the clay between eight and twelve meters. Because of this a cutting edge broke and a stone got lodged in a sample tube, making it necessary to change the sampling depth from eight to ten meters. The preliminary assessment of the clay and other remarks can be seen in Appendix A.1.1 & A.1.2.

7.1.2 Regionens Hus

The samples at Regionens Hus were taken on the 18th of February, 2013. Only 60 mm samples were taken since they only were going to be used for a comparison with previously made tests on 50 mm samples. The sampling was performed on a parking lot making it necessary to drill through the asphalt and the underlying fill material down to around 5 m below the surface. The preliminary assessment of the clay and other remarks can be seen in Appendix A.1.3.

7.1.3 Nödinge

Due to unreliable results at Lerum C it was decided that another location was needed for further testing. The location was chosen based on previous studies claiming homogenous clay layers, thus making it suitable for a comparative study. The samples from one borehole were taken to Chalmers for triaxial testing and comparison with previously made triaxial tests. The samples at Nödinge were taken on the 15th of April, 2013. Samples were taken from five different boreholes at either one or two levels. In two boreholes the 50 mm sampler was used and in the other three the 60 mm one. The samples from the two boreholes with only one level were clamped to prevent vertical swelling of the clay in the tubes. In many of the upper sample tubes there were problems with the clay not filling the tubes radially. Between one and five millimeters was missing at the top of the tubes. The preliminary assessment of the clay and other remarks can be seen in Appendix A.1.4-A.1.8.

7.2 Transportation and storage

At all of the sample locations the tubes were put in wooden boxes isolated with Styrofoam. To prevent the tubes from moving around inside the boxes they were filled with bubble wrap and foam rubber. They were then transported by car to the laboratories at Bohusgeo and Chalmers. The sample tubes from Nödinge going to Chalmers were transported in a backpack by bus. At both Bohusgeo and Chalmers the samples were stored in climate-controlled rooms where the temperature is kept at 7°C and the humidity is kept at around 75%.

7.3 Laboratory testing

Index testing was performed, according to the methods described in chapter 3, at both Chalmers and Bohusgeo on the 50 mm and 60 mm samples. The CRS tests were mainly performed at Bohusgeo and all triaxial tests at Chalmers. The triaxial tests were only performed on 60 mm samples.

7.3.1 Triaxial testing

The equipment at Chalmers consisted of four different triaxial cells, two of which were fully automated and controlled by a computer. The other two had manually controlled backpressure and cell pressure. The ones used varied depending on availability. Because of this the time between sampling and testing varies somewhat for the different tests. As can be seen in the results the accuracy of the manually controlled cells is not perfect resulting in a slight difference in the stresses after reconsolidation. The samples from 11 m at Regionens Hus and from 12.5 m at Nödinge were tested in the manually controlled cells.

The triaxial cells at Chalmers are all adapted for samples with a diameter of 50 mm and length of 100 mm. Therefore 5 mm of the periphery of the samples were removed before testing. For this a trimming device was used which had a circular cutting edge 50 mm in diameter. The 60 mm tube was put in the trimming device and the clay was then pushed up through the cutting edge and thus creating a sample of the correct dimensions, see Figure 31.

Figure 31 Soil specimen being trimmed from 60 mm to 50 mm using trimming device.

This procedure created small fractures on the cut surface however these were generally very shallow. To determine if the trimming device had any effect on the clay properties two of the samples were cut with a soil lathe instead. In that case a 100 mm long 60 mm sample was clamped in a rotating device with a 50 mm spacer attached to it. The sample was then cut piece by piece with a piano wire as the device was rotated, see Figure 32.

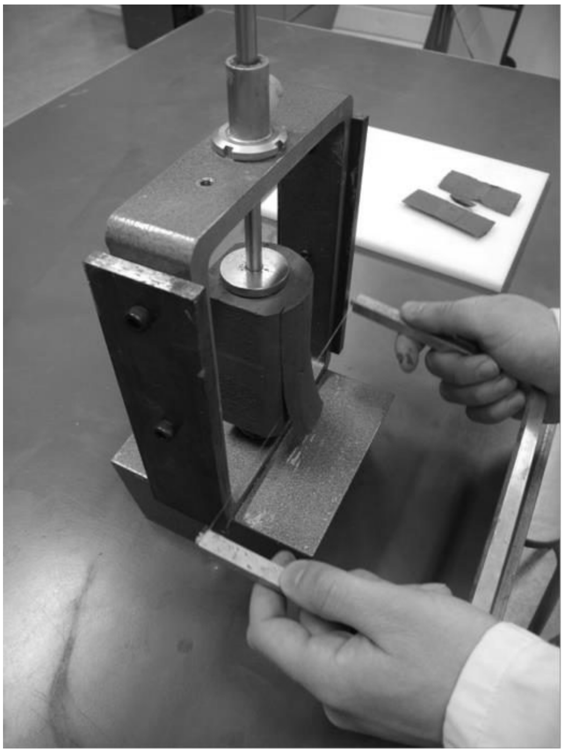

Figure 32 Soil specimen being trimmed from 60 mm to 50 mm using a soil lathe.

After the sample with the right dimensions was finished a saturated filter paper was applied to its surface. After weighing the sample it was put into the triaxial cell where filter stones was put at each end of the sample and a rubber membrane was put around it, see Figure 33. The cell was then closed and filled with oil, after which the test could be started.

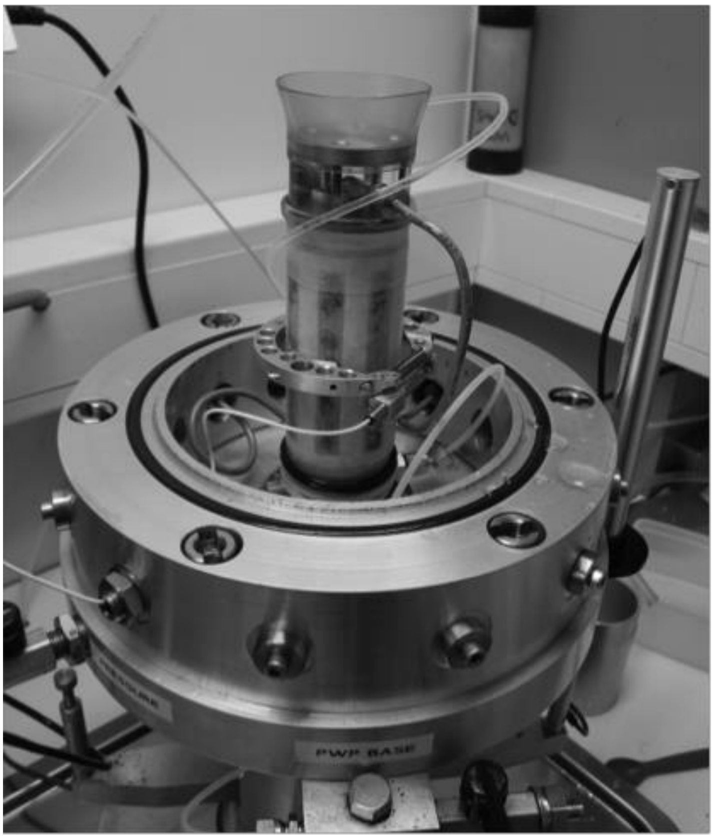

Figure 33 Soil specimen mounted in the triaxial cell.

After the sample had been mounted it was consolidated to in situ stress conditions. To prevent the stresses getting to close to the failure envelope it was first isotropically consolidated and then the deviator stress was applied to reach the correct stress conditions. The sample was then left to consolidate for 12-24 hours before it was vertically compressed at a speed of 0,01 mm/minute. Between consolidation and compression the shear wave velocity was measured on samples from 20 m and 30 m using a bender system mounted in the triaxial cell. The stresses and strains were recorded throughout the test procedure to obtain the stress path, which then could be evaluated. After the tests were completed the specimens were weighed and inspected to detect any anomalies.

7.3.2 CRS testing

A cylindrical sample with the diameter of 50 mm and the height of 20 mm was mounted in a silicon greased PTFE (Polytetraflouroethylene, colloquially referred to as Teflon) ring and inserted in the test cell (see Figure 34 to Figure 36). The cell was then placed in the press and a filter stone was placed on top of the sample. For a more in detailed description of the execution of a standard CRS test, refer to the Swedish standard (SIS, 1991).

Figure 34 Clay sample being mounted in a PTFE ring.

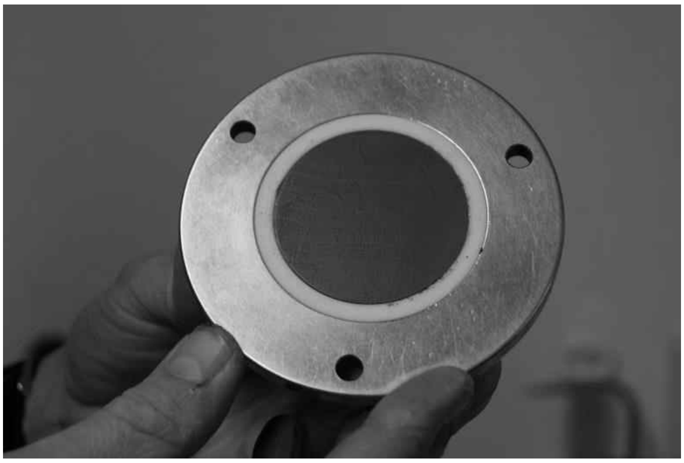

Figure 35 Clay sample inserted in the standard CRS test cell.

Figure 36 Arrangement of CRS test equipment during test.

Except for a few tests at the start of Lerum Centrum test series, all CRS tests were prepared by the same technician in order to rule out differences in laboratory handling.

7.3.2.1 Trimming equipment and implementation

Since the CRS apparatus is designed for a specimen of a diameter of 50 mm, the diameter of samples taken by the 60 mm sampler had to be reduced. For the CRS tests, the samples were trimmed with a cutting shoe with the same design as the standard sampler cutting shoe, i.e. with an angle of 5 degrees. The material of the cutting edge was polished stainless steel and the cutting shoe was lubricated by silicon grease in order to lower friction and reduce the mechanical deformation of the sample. As can be seen in Figure 37, the edge was mounted on top of the sampling tube and the clay pushed into the cutting shoe with little or no stress relief.

Figure 37 The cutting shoe mounted above a 60 mm sample tube.

For the test set 1 of samples from Lerum Centrum, the 60 mm samples were trimmed through the edge and into the ordinary PTFE ring (see Figure 38). For test set 2 of Lerum Centrum and for all Nödinge tests, the cutting edge was used directly as a test cell in the CRS apparatus (see Figure 39). When the new cutting edges where developed, new filter stone plates were also designed (see Figure 40). The change in test equipment may, among other things, have resulted in frictional changes, which are discussed in section 11.4.3.

Figure 38 Sample trimmed into the PTFE ring.

Figure 39 Arrangement of CRS equipment during test. Note the cutting shoe being used as a test cell.

Figure 40 Different design of filter stone plates, the standard version and the new version respectively. Note the external border for reduced friction on the new version.

8 Case study, Lerum Centrum

This chapter presents and analyses the test results of samples from Lerum Centrum. Samples from the standard 50 mm sampler are compared to 60 mm samples through index tests as well as CRS tests.

8.1 Index tests

The soil at Lerum Centrum consists of grey, silty clay with relatively low natural water content. Results from index tests show a trend of increasing shear strengths with depth (see Table 4, Table 5, Figure 41 and Figure 42). The clay is highly sensitive with peak values at 8 to 12 m and classified as quick clay down to around 18 m. In general, the 50 mm samples show slightly higher undisturbed shear strengths and slightly lower remoulded shear strengths compared to the 60 mm samples.

Table 4 Indicative undrained shear strength and sensitivity from fallcone tests on 50 mm samples of the Lerum Centrum clay.

Depth [m]	τ_{fu} [kPa]	τ_R [kPa]	S_t	Age [days]
4	23	0.19	125	14
5	20	0.22	92	14
7	23	0.16	145	14
8	28	0.09	315	14
10	27	0.09	306	12
12	29	0.10	290	6
14	35	0.16	221	14
16	34	0.25	140	3
18	34	0.46	75	1
20	35	0.59	60	14

Table 5 Indicative undrained shear strength and sensitivity from fallcone tests on 60 mm samples of the Lerum Centrum clay.

Depth [m]	τ_{fu} [kPa]	τ_R [kPa]	S_t	Age [days]
5	19	0.25	78	14
10	23	0.13	187	12
12	28	0.12	235	8
16	37	0.30	122	3
18	32	0.31	103	1

Table 6 Density, natural water content and liquid limits for 50 mm samples at Lerum Centrum.

Depth (m)	**ρ [t/m^3]**	**w_N [%]**	**w_L [%]**
4	1.81	42	27
5	1.73	52	35
7	1.68	59	38
8	1.68	61	39
10	1.69	57	35
12	1.71	54	34
14	1.82	42	25
16	1.72	55	40
18	1.77	48	30
20	1.80	44	36

Table 7 Density, natural water content, plastic and liquid limits, plasticity index and liquidity index for 60 mm samples at Lerum Centrum.

Depth (m)	**ρ [t/m^3]**	**w_N [%]**	**w_P [%]**	**w_L [%]**	**I_P**	**I_L**
5	1.74	52	23	31	8	3.39
10	1.70	56	25	36	11	2.83
12	1.73	55	-	34	-	-
16	1.75	53	26	38	12	2.20
18	1.78	50	23	37	14	1.99

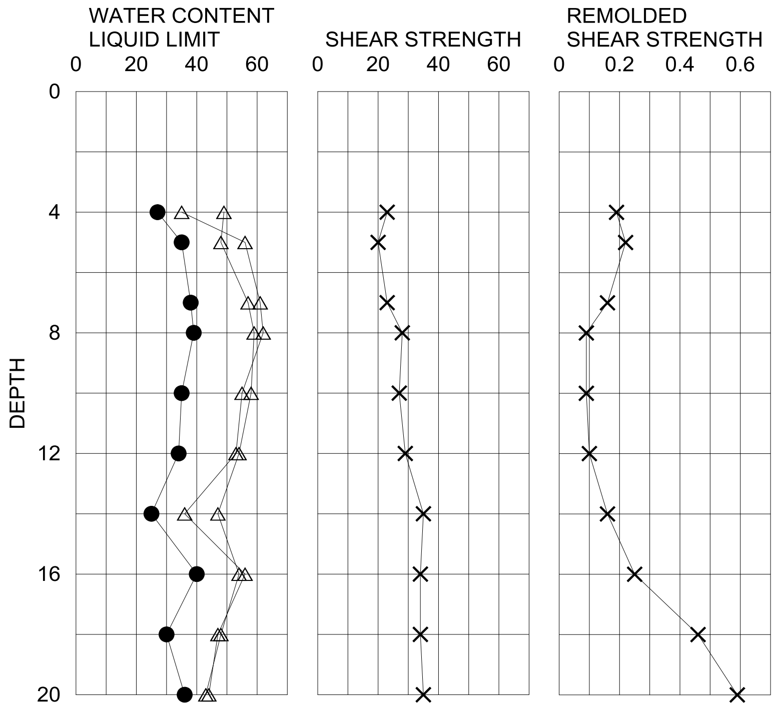

Figure 41 *Natural water content, liquid limit and shear strengths from index tests for 50 mm samples at Lerum Centrum.*

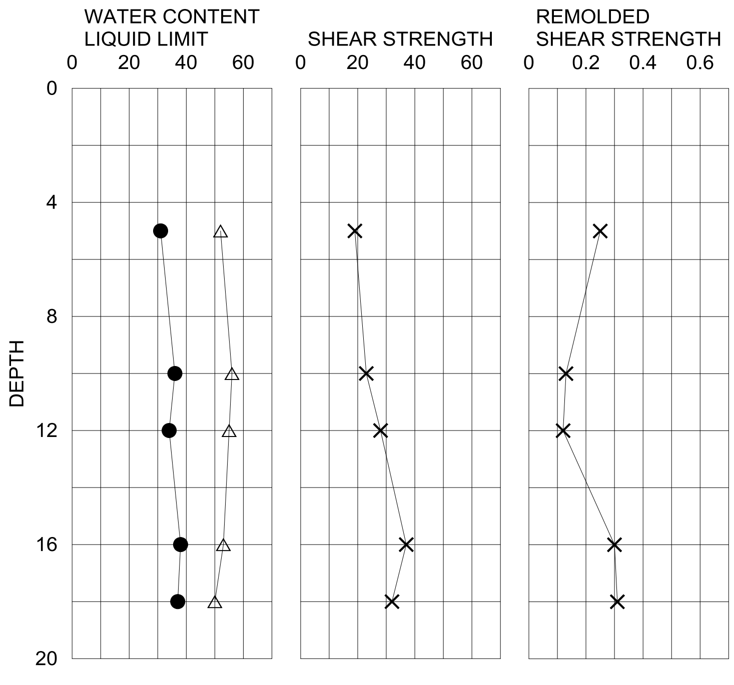

Figure 42 *Natural water content, liquid limit and shear strengths from index tests for 60 mm samples at Lerum Centrum.*

8.2 CRS tests

Measurements of density and natural water content along with structural differences like silt lenses and shell fragments show significant natural variations in the soil, especially at shallow depths, indicating that the clay is fairly inhomogeneous (see e.g. Figure 43). This is reflected in the CRS test results.

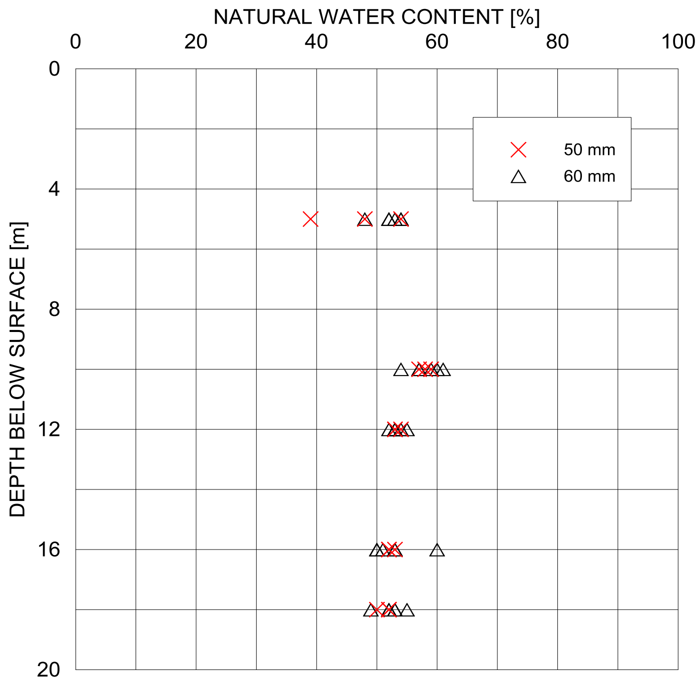

Figure 43 Variations in natural water content measured at CRS testing.

As can be seen in Table 8 to Table 12 and Figure 44, the differences in preconsolidation pressure between the different test sets show no consistent trends. In some cases the 50 mm samples give higher values, while the 60 mm samples give higher values in other cases. Worth noticing is the increased variation in preconsolidation pressure at the depths with the highest sensitivities. The modulus, M_L, is lower for the 60 mm samples compared to the 50 mm samples for the deeper sampling depths, most significant at 18 m (see Figure 45).

Table 8 Summary of preconsolidation pressure and plastic modulus evaluated from CRS-tests on samples from 5 m depth, Lerum Centrum.

σ'_c [kPa]			**M_L [kPa]**			**Age [days]**		
50 mm	**60 mm, test set 1**	**60 mm, test set 2**	**50 mm**	**60 mm, test set 1**	**60 mm, test set 2**	**50 mm**	**60 mm, test set 1**	**60 mm, test set 2**
135	130	136	862	671	599	14	14	50
131	142	130	1291	512	621	14	14	50
142	133	128	546	756	589	14	14	50

Table 9 Summary of preconsolidation pressure and plastic modulus evaluated from CRS-tests on samples from 10 m depth, Lerum Centrum.

σ'$_c$ [kPa]			**M_L [kPa]**			**Age [days]**		
50 mm	**60 mm, test set 1**	**60 mm, test set 2**	**50 mm**	**60 mm, test set 1**	**60 mm, test set 2**	**50 mm**	**60 mm, test set 1**	**60 mm, test set 2**
186	191	182	936	1076	1011	12	12	49
190	194	157	986	1202	891	12	12	49
195	182	149	1040	1267	869	12	12	49

Table 10 Summary of preconsolidation pressure and plastic modulus evaluated from CRS-tests on samples from 12 m depth, Lerum Centrum.

σ'$_c$ [kPa]			**M_L [kPa]**			**Age [days]**		
50 mm	**60 mm, test set 1**	**60 mm, test set 2**	**50 mm**	**60 mm, test set 1**	**60 mm, test set 2**	**50 mm**	**60 mm, test set 1**	**60 mm, test set 2**
208	190	203	1008	1163	1087	6	8	48
223	206	188	1122	1283	1082	6	8	48
237	214	199	921	1131	1011	6	8	48

Table 11 Summary of preconsolidation pressure and plastic modulus evaluated from CRS-tests on samples from 16 m depth, Lerum Centrum.

σ'$_c$ [kPa]			**M_L [kPa]**			**Age [days]**		
50 mm	**60 mm, test set 1**	**60 mm, test set 2**	**50 mm**	**60 mm, test set 1**	**60 mm, test set 2**	**50 mm**	**60 mm, test set 1**	**60 mm, test set 2**
274	255	287	1543	1336	1233	3	3	41
264	243	284	1483	1311	1061	3	3	41
255	250	284	1404	1372	1069	3	3	41

Table 12 Summary of preconsolidation pressure and plastic modulus evaluated from CRS-tests on samples from 18 m depth, Lerum Centrum.

σ'$_c$ [kPa]			**M_L [kPa]**			**Age [days]**		
50 mm	**60 mm, test set 1**	**60 mm, test set 2**	**50 mm**	**60 mm, test set 1**	**60 mm, test set 2**	**50 mm**	**60 mm, test set 1**	**60 mm, test set 2**
277	257	283	1344	887	907	1	1	42
264	251	267	1368	917	773	1	1	42
276	253	250	1417	999	986	1	1	42

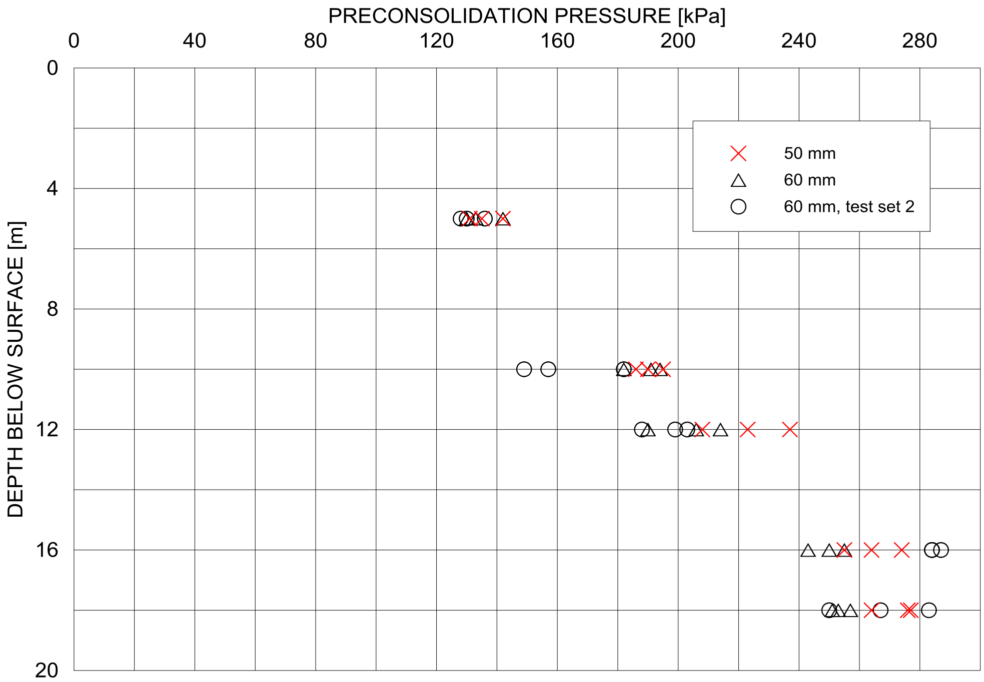

Figure 44 Evaluated preconsolidation pressure for all CRS-tests from Lerum Centrum.

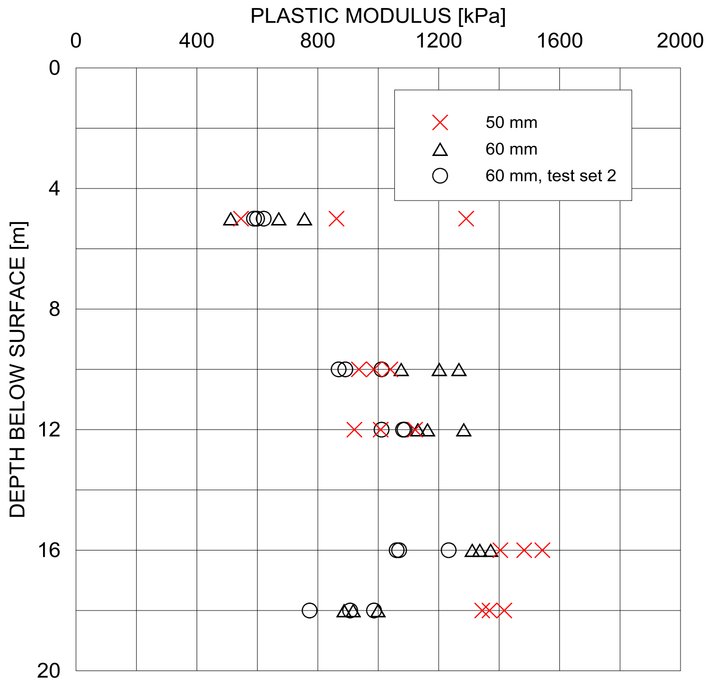

Figure 45 Evaluated plastic modulus for all CRS tests from Lerum Centrum.

Figure 46 to Figure 50 presents the stress-strain curves for the CRS tests of Lerum Centrum samples. The 50 mm curves are in most cases situated on top of the 60 mm curves, i.e. at the depth of 12 m (see Figure 48). One of the 50 mm samples at 5 m (Figure 46) appears to be disturbed since the shape is very flat with a diffuse knee. However, one of the other curves seems to approach the same line. At 10 m (Figure 47), two of the 60 mm tests from set 2 are not following the pattern of the other curves, the shape is similar, but the knee on these curves starts to form on a much lower stress. The test results from 16 m (Figure 49) stands out from the others because of the 60 mm, test set 2 samples all having both higher initial modulus, M_0, higher preconsolidation pressure and lower modulus, M_L, than the other tests from 16 m, which according to the theory indicates higher sample quality.

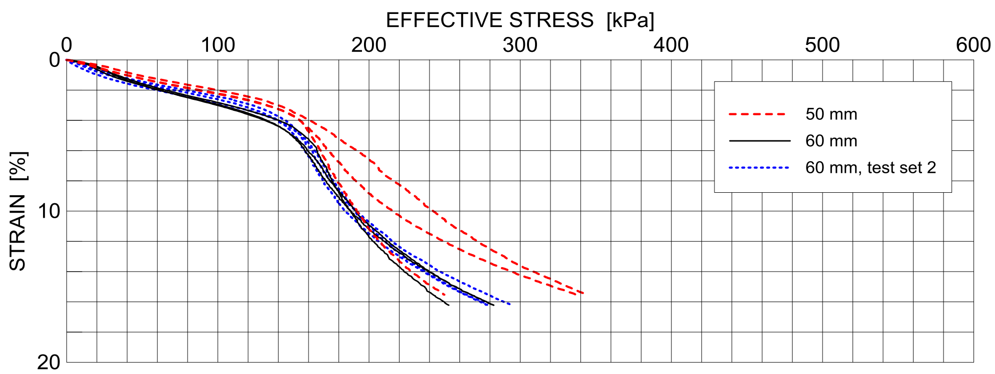

Figure 46 Stress-strain curves, Lerum Centrum, 5 m.

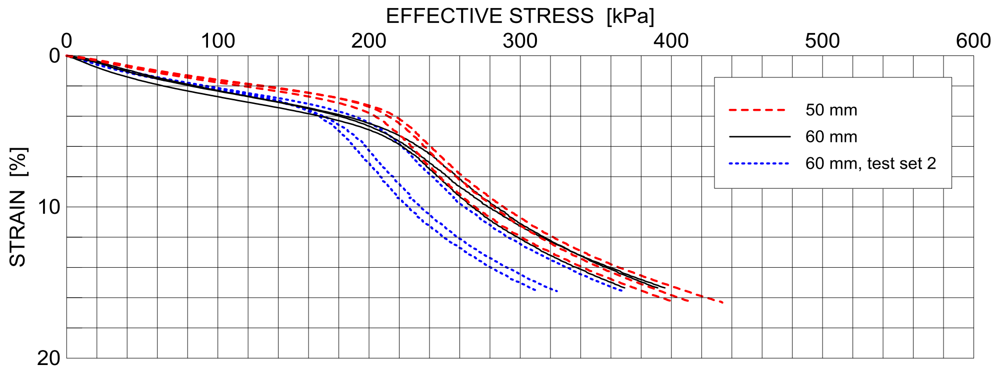

Figure 47 *Stress-strain curves, Lerum Centrum, 10 m.*

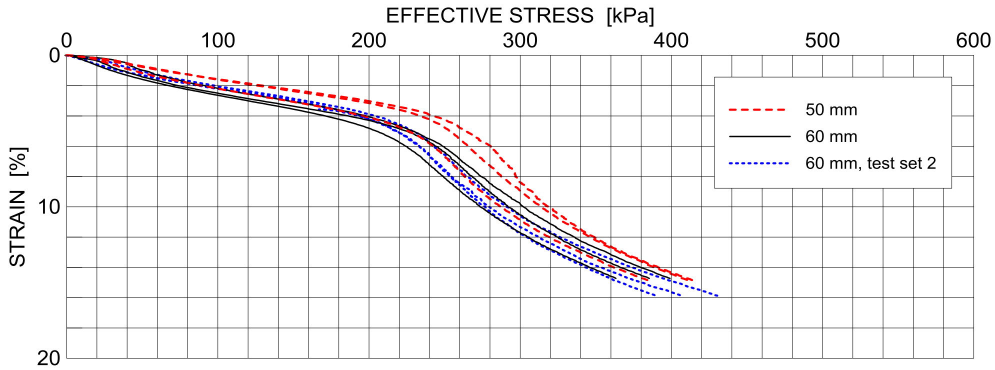

Figure 48 *Stress-strain curves, Lerum Centrum, 12 m.*

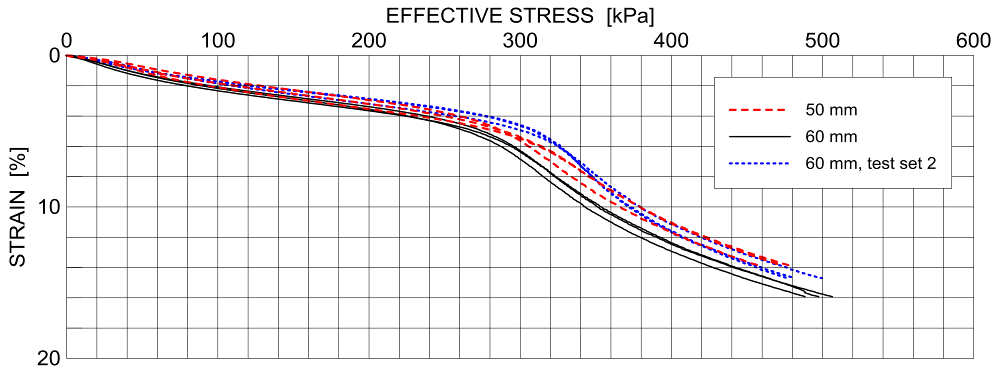

Figure 49 *Stress-strain curves, Lerum Centrum, 16 m.*

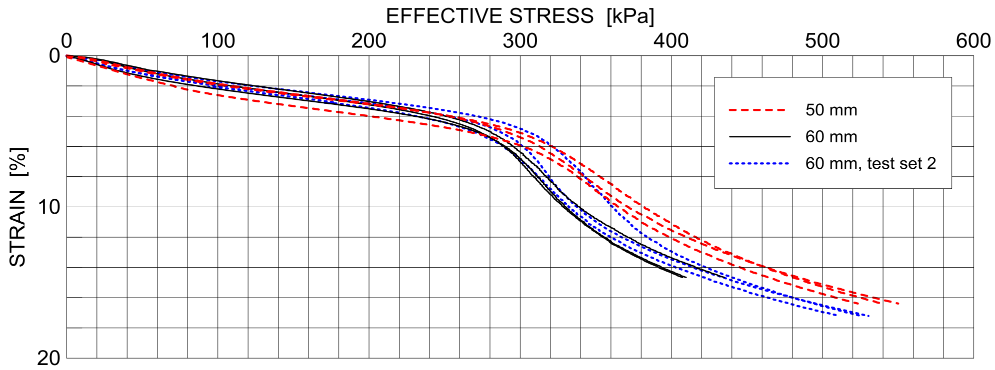

Figure 50 Stress-strain curves, Lerum Centrum, 18 m.

8.3 Analysis

Previous experiences from Lerum show that it is difficult to retrieve samples of satisfying quality, due to the high sensitivity of the clay. The tests in this study show a relatively large scatter that could indicate poor sample quality. However, some of the scatter in test results can be explained by natural variation. Therefore, it is hard to determine if the identified differences in the results are caused by differences in sample quality or not. For example, huge scatter in density can in part explain the different shapes of the CRS stress-strain curves at 5 m. The frequent occurrence of shell fragments and grits may also have influenced the test results. In general, it can be said that the tests that differ from the pattern could be disturbed, but it cannot be ruled out that differences may be due to inhomogeneity in the clay. Some of the samples have different properties than the others.

The differences in shape between the CRS results of the 50 mm and 60 mm samples, when comparing all results, can be clarified by normalising the stress-strain curves at the preconsolidation pressures (Figure 51). This makes it possible to compare results from different depths. Much of the scatter is depth dependent, with the deeper tests showing a less distinct shape. As can be seen in Figure 45 the plastic modulus is in general increasing with depth.

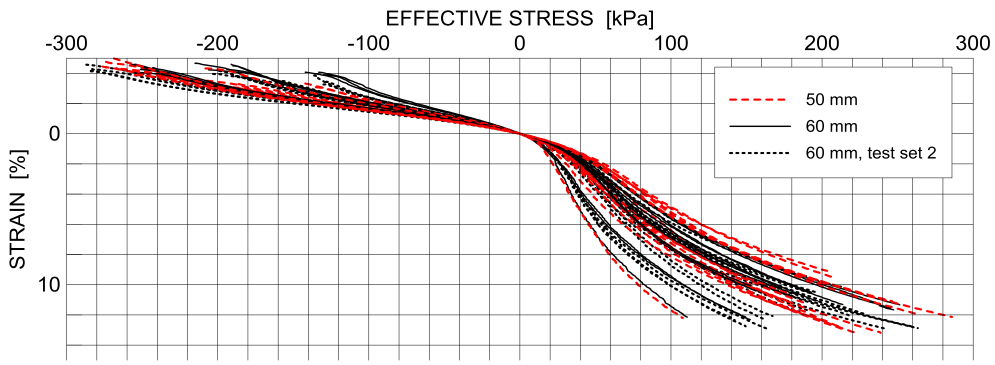

Figure 51 CRS test stress-strain curves normalised at the preconsolidation pressures, Lerum Centrum.

One possible explanation for the two tests in test set 2 at 10 m that differs a lot from the others is problems during the test preparations in the laboratory. However, the results do not indicate that the samples are more disturbed than the other ones. The test results from test set 2 at 16 m are the only results from Lerum Centrum that fully behave according to the hypothesis that the increase in diameter would manifest in both higher preconsolidation pressure and elastic modulus and lower plastic modulus, as seen in figure 48. At 12 m the result is the opposite, with indications of higher quality of the 50 mm samples.

The impact of changing the trimming method of the 60 mm samples between test set 1 and test set 2 is hard to determine, due to the variations in test results. Only at 16 m the results are clearly in the favour of test set 2.

As can be seen in Figure 52, the disturbance evaluation according to Larsson et al (2007) indicates a somewhat lower disturbance of the 50 mm samples in general, though there is no obvious trend. All tests are found in the good to fair category. Most of the results are well gathered together, while the large differences in natural water content at 5 m makes the 50 mm samples to show a larger scatter.

Due to the inconsistent results from Lerum Centrum, it is hard to draw any conclusions about differences in the sample quality. No significant improvements are found with the 60 mm sampler.

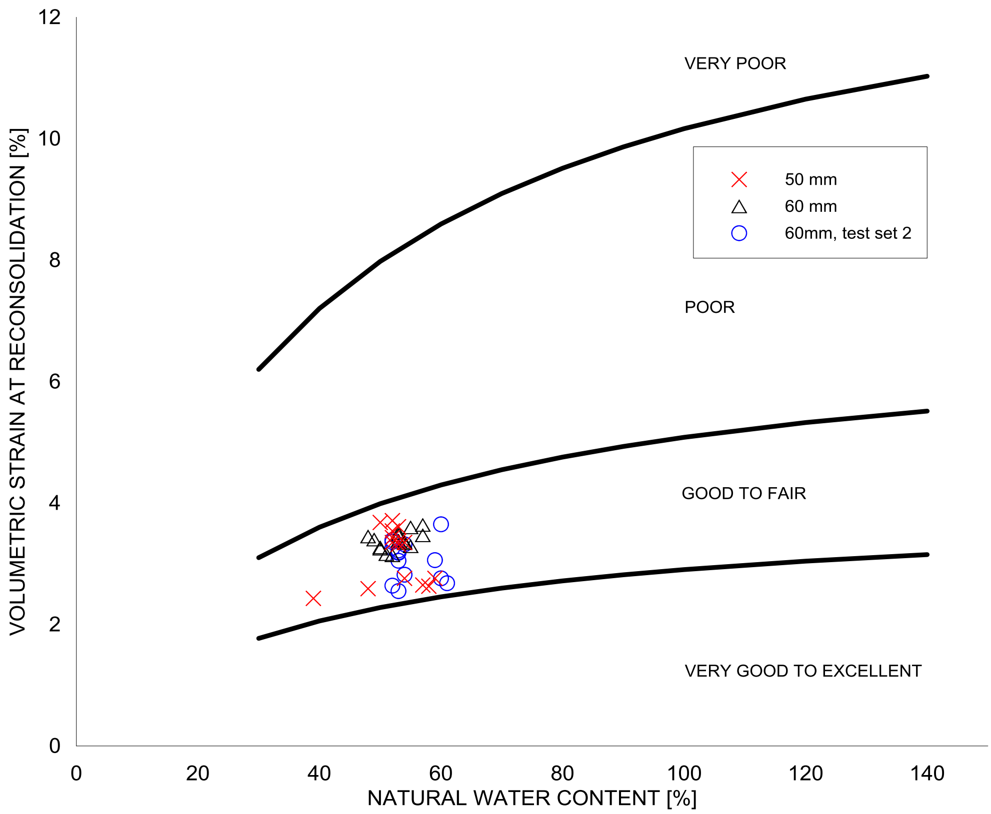

Figure 52 *Volumetric strain at reconsolidation for all CRS tests at Lerum Centrum (Disturbance evaluation according to Larsson et al, 2007).*

9 Case study, Regionens Hus

In this chapter the laboratory results for Regionens Hus will be accounted for and analysed. At Regionens Hus the main focus has been to compare results from triaxial tests on the samples taken with the 60 mm sampler and previous tests made at Chalmers. The study focuses on the clay at the depths 11, 20 and 30 m. It is on clay from these depths that the CRS and triaxial tests have been performed.

9.1 Index tests

As can be seen in the tables and figure below, the properties of the clay change, e.g. increased shear strength with depth. The clay is classified as medium-sensitive and the sensitivity reaches it maximum value at around 11 m. The colour of the clay also varies. At 11 m it is grey with some silt lenses, at 20 m it has a dark sulphide colour and at 30 m it is grey but with darker parts. The index tests show results that were expected from clay in the central parts of Göteborg. The 10 m sample was tested two months after the other samples which, as can be expected, has made it slightly stiffer as can be seen in Table 13.

Table 13 Indicative undrained shear strength and sensitivity from fallcone tests on 60 mm samples of the Regionens Hus clay.

Depth [m]	τ_{fu} [kPa]	τ_R [kPa]	S_t	Age [days]
10	18	1.02	19	134
11	16	0.57	28	7
15	24	0.94	26	4
20	28	1.82	16	7
25	35	2.84	13	4
30	37	2.30	17	3

Table 14 Natural water content, plastic and liquid limits, plasticity index and liquidity index for 11, 20 and 30 m depth at Regionens Hus.

Depth [m]	w_N [%]	w_P [%]	w_L [%]	I_P	I_L
11	73	28	61	33	1.37
20	64	31	66	35	0.93
30	68	31	74	43	0.86

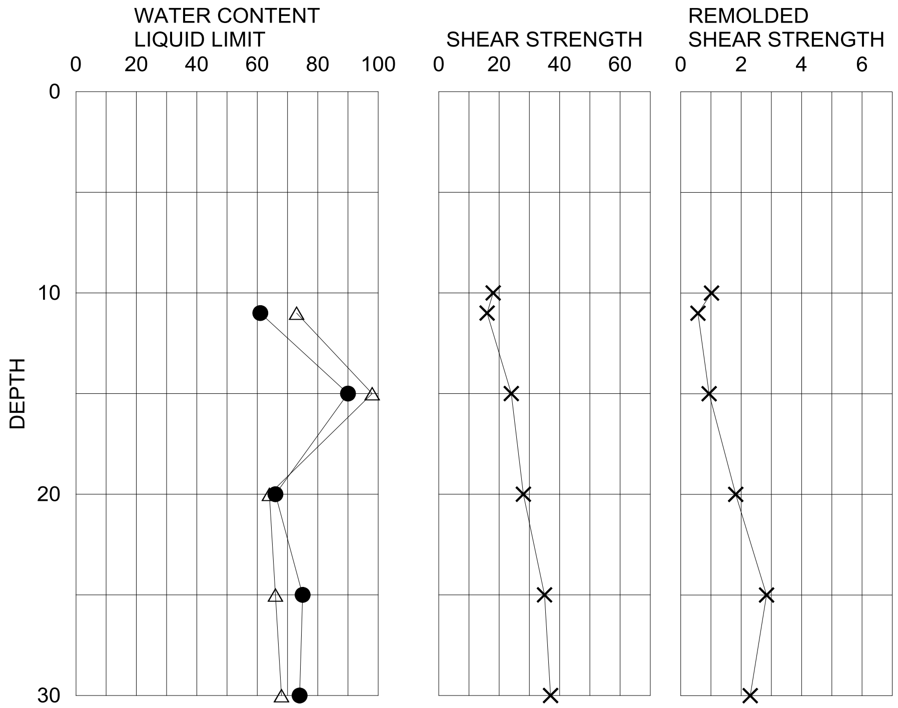

Figure 53 *Natural water content, liquid limit and shear strengths for 60 mm samples at Regionens Hus.*

9.2 CRS tests

The results from the CRS tests at Regionens Hus show the same tendencies as the ones performed on clay from Lerum C. The moduli M_0 and M_L are both lower for the test performed on the clay taken with the 60 mm sampler compared to the clay taken with the St II sampler. The drop in effective stress at 10% strain in the 11 m test is due to a pressure drop in the testing equipment. However, this did not affect the sample much and after a reset it continued loading with the expected deformation pattern. Worth noting is that the 60 mm sample was trimmed with equipment made at Chalmers which differs in design compared to the equipment at Bohusgeo's laboratory. The 11 m samples are compared to previously taken samples from 10 m. The one meter difference is assumed to have a small effect on the results, since the comparison is mainly focused on the shape of the stress-strain curves. Around 10 m the clay properties vary a lot. This can be seen in the stress-strain curves, which indicate good quality with well-defined knees and similar M_L, but with a large difference in σ'_c.

Table 15 *Summary of preconsolidation pressure and plastic modulus evaluated from CRS-tests of 50 mm and 60 mm samples, Regionens Hus.*

	σ'$_c$ [kPa]		**M_L [kPa]**		
Depth [m]	**50 mm**	**60 mm**	**50 mm**	**60 mm**	**Age [days]**
10	89	-	350	-	n/a
10	91	-	390	-	n/a
10	113	-	453	-	n/a
11	-	132	-	615	37
30	285	-	1183	-	n/a
30	-	276	-	827	1

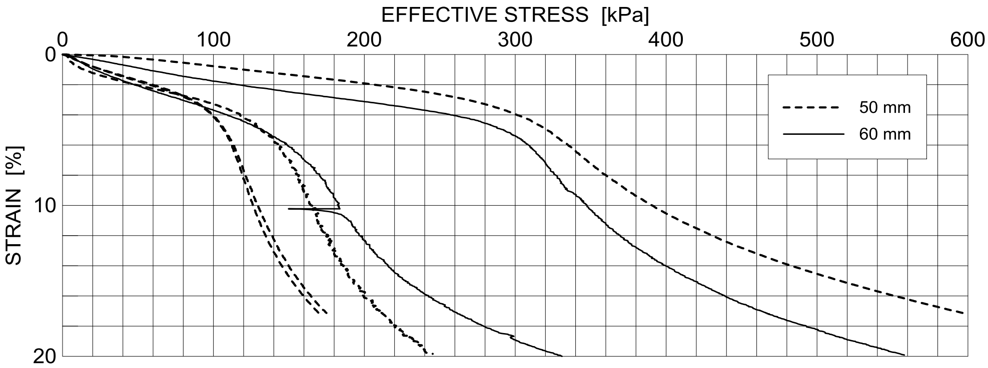

Figure 54 *Stress-strain curves for CRS tests, Regionens Hus, 10, 11 and 30 m respectively.*

9.3 CAUC triaxial tests

At Regionens Hus the triaxial tests were only performed on clay samples taken with the 60 mm sampler. A comparison with test previously made at Chalmers laboratory was then carried out. The samples were tested in a specific order, starting with the deepest samples and ending with the ones from the shallower depths. Trimming of the samples from 60 mm to 50 mm was performed with a cutting shoe in all but two cases. At 11 m and 20 m one of the samples was trimmed with a soil lathe instead to make sure that the cutting shoe did not affect the deformation behaviour of the clay. Some of the previously made tests were also trimmed using the two different methods, but in that case from 50 mm to 38 mm.

*Table 16 Evaluated shear strength and preconsolidation pressure from CAUC triaxial tests on Regionens Hus clay. *Previous tests from the central station.*

Depth [m]	**Test nr**	**τ [kPa]**	**σ'_c [kPa]**
11	1	31.4	101
11	2	35.2	109
20	1	55.5	156
20	2	53.0	153
30	1	86.7	278
30	2	83.3	248
11*	50 mm	33.1	97
11*	38 mm	32.5	98
20*	50 mm	55.3	165
30*	50 mm	71.1	245
30*	38 mm	73.1	234
30*	38 mm	75.1	238

At 11 m one of the samples had a seashell in it clearly affecting the failure surface. A lower shear stress was achieved in that sample as can be seen in Figure 56. That sample was also trimmed using the soil lathe.

The two tests on the samples taken with the 60 mm sampler at 20 m were performed a few weeks apart. The later one had begun to oxidize at that point which can clearly be seen in Figure 55 below. Most of the oxidized parts of the sample were removed and the total shear stress achieved is very close to the maximum shear stress of the first sample. The second sample was the one trimmed using the soil lathe.

Figure 55   Clay sample from 20 m in trimming device. The sample has begun to oxidize.

The results from the 30 m samples show the biggest difference in shear stress when they are compared to previous tests. Worth noting however is that one of the samples was consolidated to the wrong in situ stresses and that the earlier tests were performed on relatively old clay. Although even in those cases the trimmed samples achieved a higher shear stress.

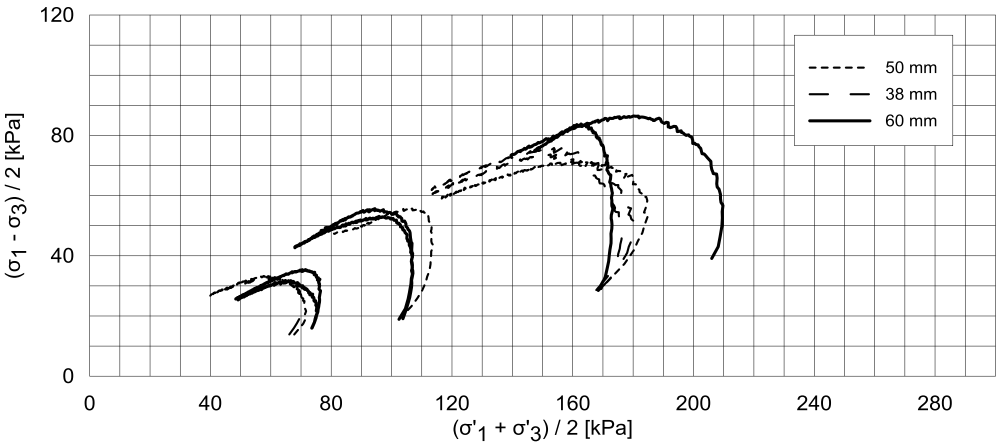

Figure 56   Summary of stress paths for all CAUC triaxial tests, Regionens Hus.

In the Figure 56 above a difference in stress path can be seen for the trimmed 60 mm samples, with the exception of one of the samples from 11 m. There is less curvature and they tend to have an almost vertical increase in shear stress. Different types of failure could explain the slight difference of the stress paths at peak shear stress. Depending on if the sample get a clean failure surface or gets barrel shaped at failure the stress path will differ.

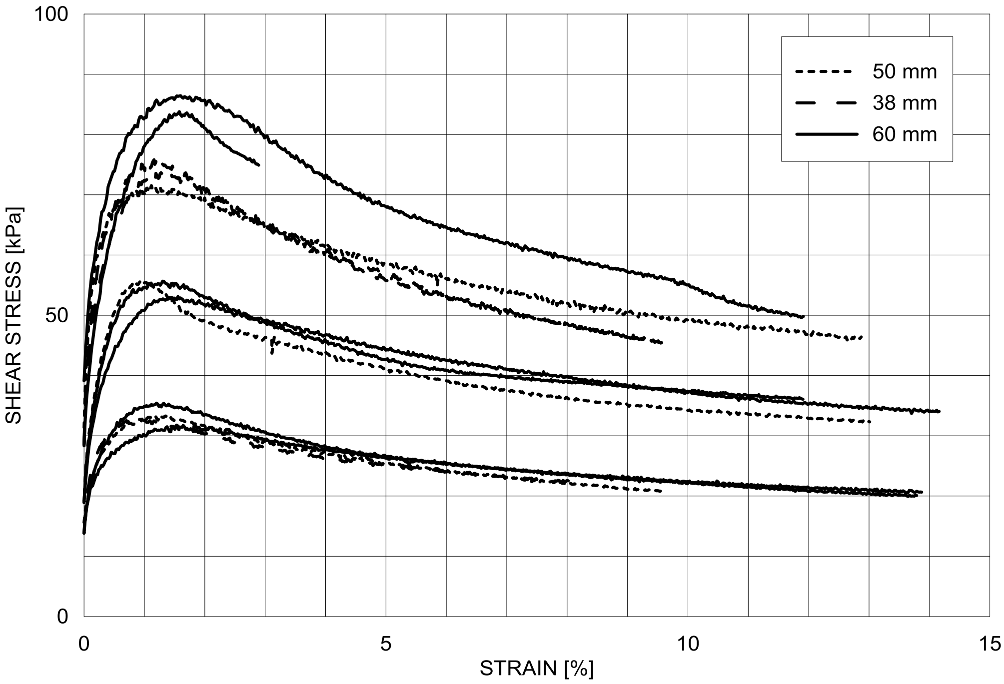

Figure 57 Summary of all stress-strain curves for CAUC triaxial tests at Regionens Hus.

The shear strength of the 60 mm samples lies very close to that of the 50 mm samples, with the exception of the sample from 30 m. The initial stiffness is also about the same for all samples. In some cases the 50 mm samples even show a higher stiffness.

9.4 Shear wave velocity

The shear wave velocity was measured in one sample from Regionens Hus, from 30 m depth. For comparison in-situ values and sample results from 50 mm samples are used (Wood, n.a.). The values come from measurements at the Göteborg Central Station, which is located very close to Regionens Hus.

Table 17 In-situ shear wave velocity at Göteborg Central Station (Wood, n.a.)

Depth [m]	V_S [m/s]
10	106
11	114
20	128
27	141
30	148

The shear wave velocity was measured in 50 mm samples of different age but also in trimmed 38 mm samples.

Table 18 Shear wave velocity measurements in 50 mm and trimmed 38 mm samples (Wood, n.a.)

Depth [m]	Diameter [mm]	$V_{S\,lab}$ [m/s]	$V_{S\,lab}/V_{S\,in\text{-}situ}$	Age
10	50	90	0,85	1 week
11	50	85	0,74	5 months
27	50	145,5	0,98	< 1 day
30	50	95	0,64	5 months
30	38	114	0,77	5 months
30	38	96	0,65	5 months

In the 60 mm sample a V_S of 141 m/s was measured, giving a ratio of 0,95. This is a high value, but the sample is fresh (only two days old) and also trimmed to a diameter of 50 mm. Based on the values in the table above it can be seen that the quality of samples decrease over time. The trimmed samples also show a higher quality.

9.5 Analysis

At Regionens hus only two CRS tests were performed on 60 mm clay samples and four results from 50 mm samples were used for comparison. A fair comparison cannot be made from this few results but they still show the same tendencies as the results from Lerum C, with lower moduli and similar preconsolidation pressure. The tests were performed at Chalmers laboratory using different equipment for trimming than at Bohusgeo's laboratory. The equipment at Chalmers is not purpose made for trimming samples for CRS testing as it is at Bohusgeo, which made it difficult getting the sample into the CRS ring without disturbing it. This might have contributed to the lower moduli.

In the results from the triaxial tests some deviating results could be seen for the 60 mm samples. At 11 m a seashell affected the shearing surface giving it a lower shear strength and at 20 m the sample had begun to oxidize also lowering the shear strength, although marginally. The 20 m sample also deformed more before reaching maximum strength. It is known that the 50 mm samples at 30 m were close to six months old when they were tested but for the other samples no such information has been available. Therefore the comparison could be misleading in the way that the 60 mm samples have been thoroughly studied throughout the sampling and testing process while the 50 mm samples may have had issues that are not known.

Although the two samples trimmed using the soil lathe achieved lower shear strength than the ones trimmed with the trimming device, both methods are assumed to be equally effective. In both cases the trimmed and stamped samples initially follow the same stress path and they converge at the end assuming the same friction angle. Only the peak strength differs which can be expected since the shearing surfaces were affected by discrepancies in the samples. The only visible effect of trimming with the trimming device was shallow fractures on the cut surface, but no apparent reduction of strength could be seen. The shear wave velocity measurements indicate a higher quality in the trimmed samples, although that was in older samples. Fresh samples show no increase in quality since it is high to begin with.

10 Case study, Nödinge

Results from index tests, CRS tests and CAUC tests on samples from Nödinge test site are presented and analysed below.

10.1 Index tests

The soil at sampling depth is grey clay with a green tint. The clay texture appears to be organic, even though ignition tests indicate a low organic content. Shell fragments occasionally occur in some of the samples. Fallcone tests (Table 19 and Table 21) indicate low shear strength and the clay is medium-sensitive and slightly over consolidated. Comparisons between the 50 mm sample and 60 mm sample show little difference, though the remolded shear strength at 5 m being somewhat higher for the 50 mm sample. The consistency limits and the natural water content can be found in Table 20 and Table 22. In general, the clay is homogenous and the natural differences in the material are small.

Table 19 Indicative undrained shear strength and sensitivity from fallcone tests on 50 mm samples of the Nödinge clay.

Depth [m]	**τ_{fu} [kPa]**	**τ_R [kPa]**	**S_t**	**Age [days]**
5	15	1.22	13	19
12.5	20	1.33	15	3

Table 20 Natural water content, plastic and liquid limits, plasticity index and liquidity index of 50 mm samples from 5 and 12.5 m depth at Nödinge.

Depth [m]	**ρ [t/m^3]**	**w_N [%]**	**w_P [%]**	**w_L [%]**	**I_P**	**I_L**
5	1.50	89	38	85	47	1.08
12.5	1.53	87	38	87	49	1.00

Table 21 Indicative undrained shear strength and sensitivity from fallcone tests on 60 mm samples of the Nödinge clay.

Depth [m]	**τ_{fu} [kPa]**	**τ_R [kPa]**	**S_t**	**Age [days]**
5	15	1.02	15	17
12.5	19	1.33	15	2

Table 22 Natural water content, plastic and liquid limits, plasticity index and liquidity index of 60 mm samples from 5 and 12.5 m depth at Nödinge.

Depth [m]	**ρ [t/m^3]**	**w_N [%]**	**w_P [%]**	**w_L [%]**	**I_P**	**I_L**
5	1.51	93	38	86	48	1.15
12.5	1.55	87	39	85	46	1.05

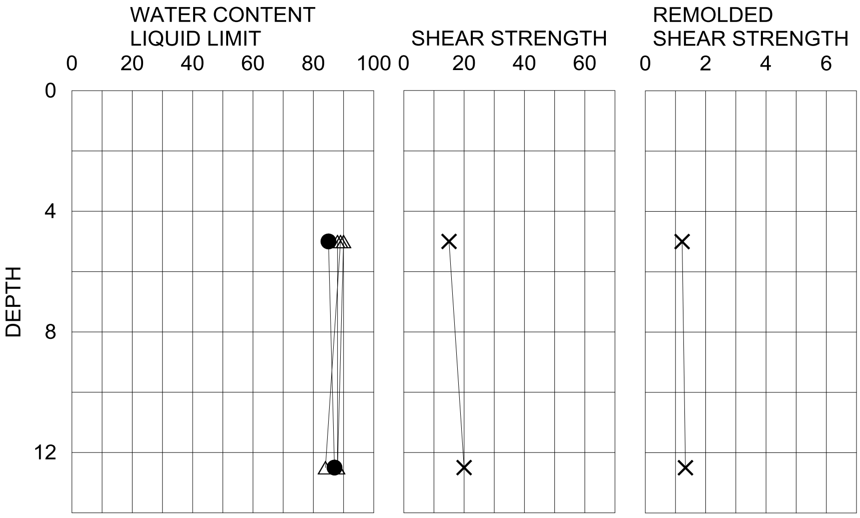

Figure 58 Natural water content, liquid limit and shear strengths for 50 mm samples at Nödinge.

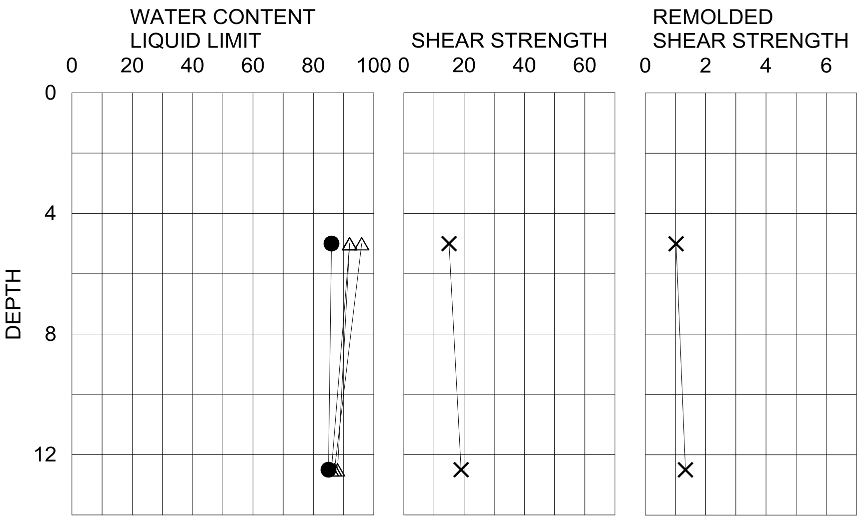

Figure 59 Natural water content, liquid limit and shear strengths for 60 mm samples at Nödinge.

Tests of pore water distribution in some samples can be found in Table 23. The results indicate the migration of pore water from the periphery to the core as described in theory section 4.4. However, attention must be given to the fact that the sample may have been subjected to dehydration due to plastic lids that does not seal the samples effectively. In addition, too few tests have been performed to provide absolute certainty.

Table 23 *Pore water distribution in samples from Nödinge. Note the trend of increasing water content from the periphery to the core.*

Depth [m]	**60-50 mm**	**50-30 mm**	**30-0 mm**	**Age [days]**
5	90%	92%	94%	2
12.5	81%	83%	84%	7
12.5	84%	85%	85%	14

10.2 CRS tests

The CRS tests at Nödinge is in general very uniform. Measurements of the natural water content in conjunction with the CRS tests show a variation of up to around 7 % (see Figure 60). The evaluated preconsolidation pressures for the CRS tests in Nödinge show a larger scatter at 5 m than on 12.5 m, while the modulus, M_L, show the opposite variation (Table 24 to Table 25 and Figure 61 to Figure 62). The variations are analysed in depth in sections 10.4 and 10.5. As can be seen in Figure 62, the moduli of the 60 mm samples tend to be lower than the moduli of the 50 mm samples.

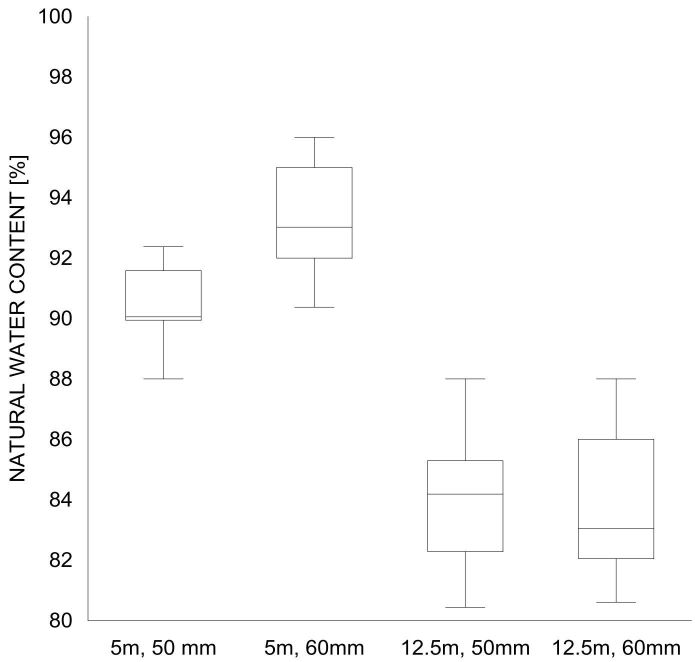

Figure 60 *Variations in natural water content from CRS testing, Nödinge. The plot is of Box-and-Whisker type and shows the maximum and minimum values, the median and the upper and lower quartile of the data series.*

Table 24 Summary of preconsolidation pressure and plastic modulus evaluated from CRS-tests on samples from 5 m depth, Nödinge.

σ'$_c$ [kPa]		**M_L [kPa]**		**Age [days]**	
50 mm	**60 mm**	**50 mm**	**60 mm**	**50 mm**	**60 mm**
47	39	276	227	19	17
48	38	239	227	19	17
48	42	251	224	19	17
49	48	280	290	21	20
47	45	239	231	21	20
33	42	345	240	21	20

Table 25 Summary of preconsolidation pressure and plastic modulus evaluated from CRS-tests on samples from 12.5 m depth, Nödinge.

σ'$_c$ [kPa]		**M_L [kPa]**		**Age [days]**	
50 mm	**60 mm**	**50 mm**	**60 mm**	**50 mm**	**60 mm**
76	82	366	430	3	2
76	81	423	366	3	2
76	79	482	389	3	2
77	82	429	451	8	4
80	81	442	363	8	4
78	79	433	417	8	4
82	83	495	441	11	9
81	78	434	457	11	9
79	77	492	427	11	9
78	76	550	459	16	15
80	79	493	385	16	15
82	81	545	467	16	15

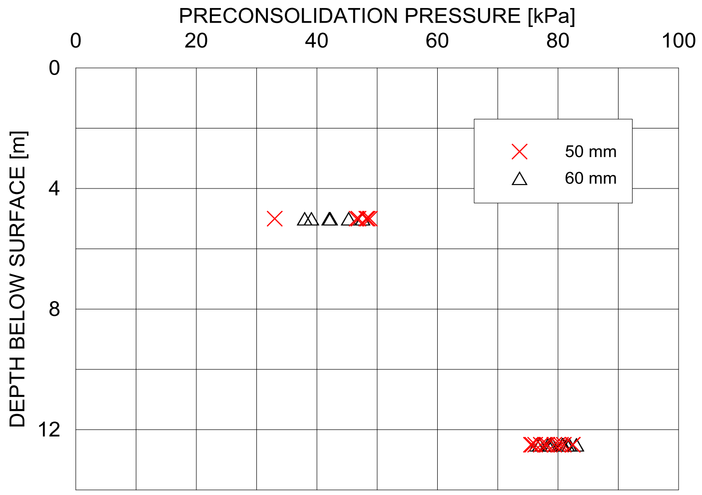

Figure 61   Evaluated preconsolidation pressure for all CRS-tests from Nödinge.

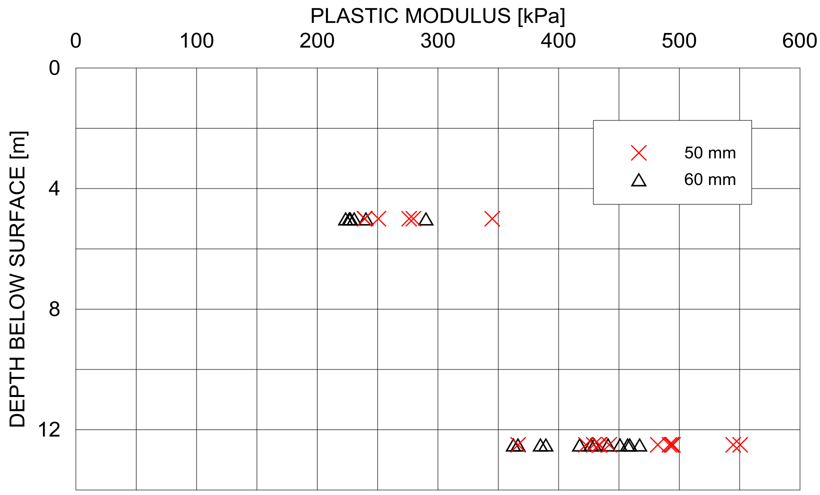

Figure 62   Evaluated plastic modulus for all CRS tests from Nödinge.

Stress-strain curves from the CRS tests in Nödinge can be found in Figure 63 and Figure 64. One of the 50 mm samples at 5 m is apparently disturbed, showing a very soft knee. At 12.5 m the scatter is low, indicating a homogenous material and good sample quality. At stresses above the preconsolidation pressure, the 50 mm tests show a larger scatter than the 60 mm tests. Two of the 60 mm sample curves show large strains in the very beginning of the test, making them stand out from the rest of the tests.

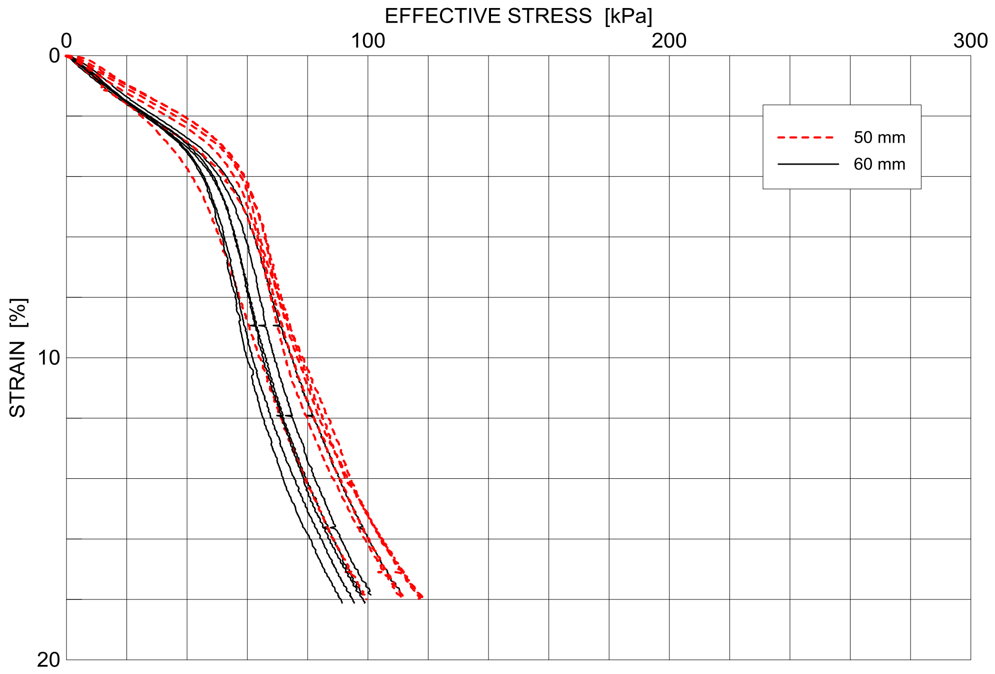

Figure 63 Stress-strain curves, Nödinge, 5 m

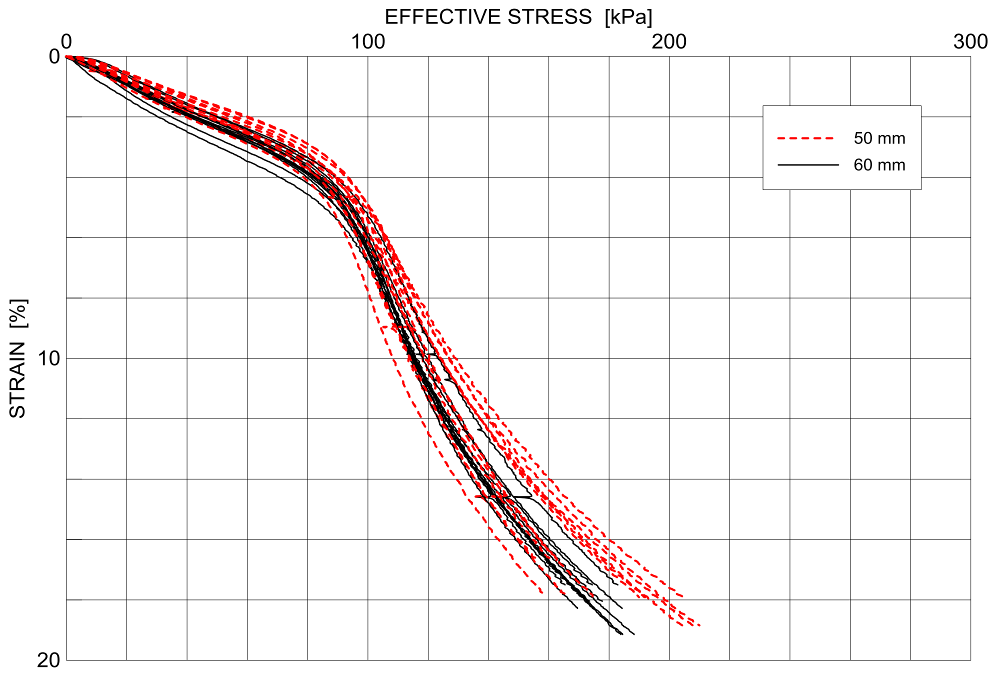

Figure 64 Stress-strain curves, Nödinge, 12.5 m

10.3 CAUC triaxial tests

At Nödinge three triaxial tests were performed; one on a sample from 5 m and two on samples from 12.5 m. All samples were trimmed using the trimming device and the results are compared to 50 mm samples tested at an earlier time. The testing procedure differed from the one at Regionens Hus as the shallower sample was tested first.

Table 26 Evaluated shear strength and preconsolidation pressure from CAUC triaxial tests on Nödinge clay of 60 mm samples.

Depth [m]	Test nr	Shear stress [kPa]	Preconsolidation pressure [kPa]
5	-	14.8	39
12.5	1	22.7	65
12.5	2	24.4	72
5	-	16.7	34
12.5	-	26.4	73

As can be seen in Figure 66 and Figure 67, the same tendency with lower shear strength have been measured for the samples from Nödinge, as was also the case at Regionens Hus. A lower stiffness for the 60 mm samples can also be seen.

During testing of the first sample from 12.5 m the triaxial cell broke, making continued testing impossible. Since the sample only had been subjected to very low pressures it was saved for testing at a later time. It was stored covered in plastic wrap inside a sealed jar for a week before testing could continue. At that point a new sample from 12.5 m was prepared and tested simultaneously. Due a slight difference in the pressure regulators on the triaxial cells used, the consolidation pressure differed somewhat between the samples. The colour of the samples, taken from the middle and lower tube, were different. The one from the middle tube was grey and the one from the lower tube was stained with a dark sulphide colour, see Figure 65.

From the results it can be seen that the first sample had a different deformation behaviour than the second one, with a lower shear strength and lower stiffness. The friction angle is also different for the first sample.

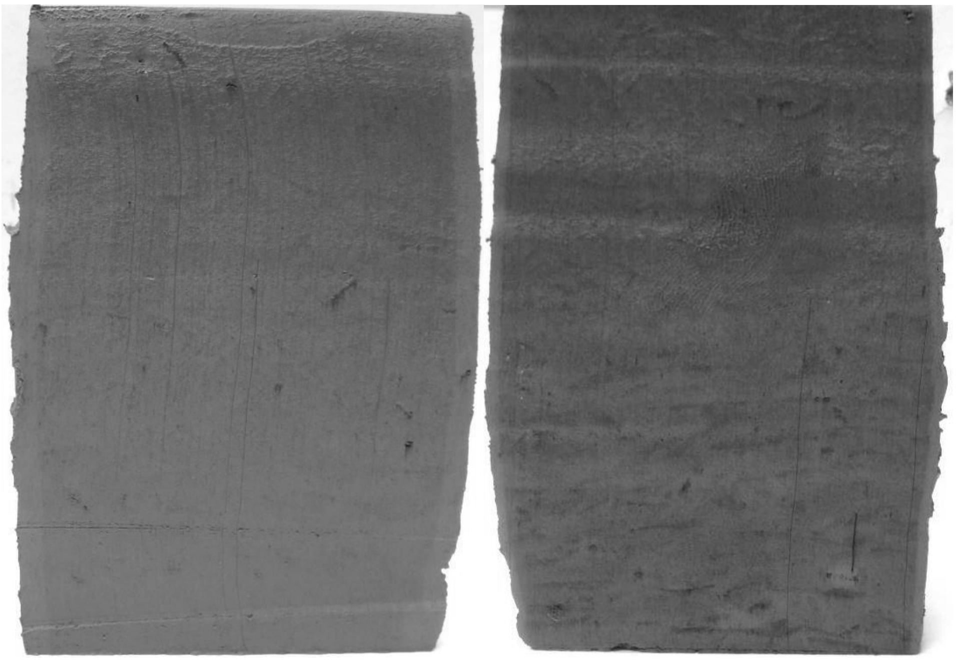

Figure 65 Tested samples from middle and lower tubes at 12.5 m with different colours.

The sample from 5 m showed the same behaviour as the second sample from 12.5 m. When compared to the 50 mm sample it had a lower shear strength and lower initial stiffness. At 5 m the clay exhibited a ductile behaviour with a deformation of around three percent at peak strength, both for the 50 mm and 60 mm sample, see Figure 66 and Figure 67. Worth noting is that the 60 mm sample had some small silt lenses in it which could have affected the deformation behaviour.

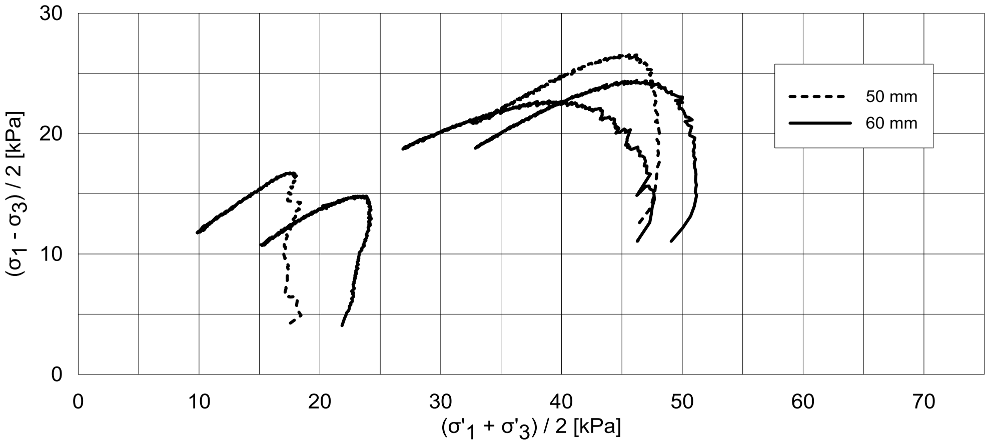

Figure 66 Summary of stress paths for all CAUC triaxial tests, Nödinge.

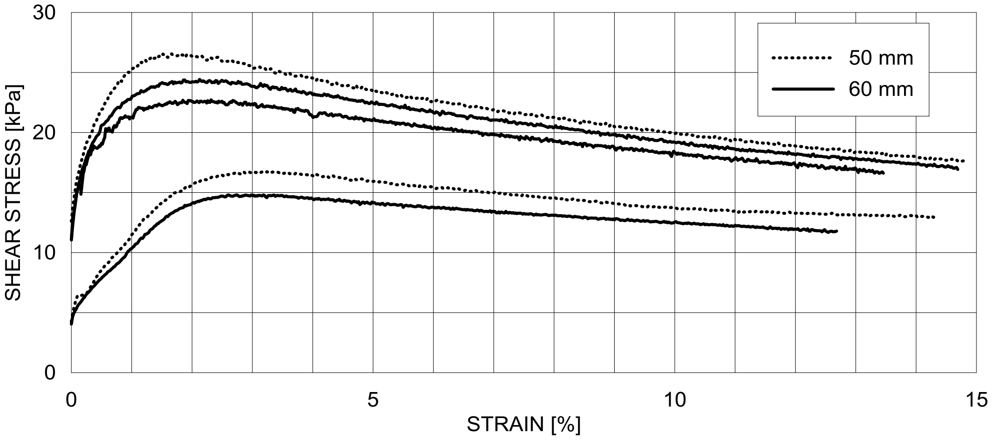

Figure 67 Summary of all stress-strain curves for CAUC triaxial tests at Nödinge.

10.4 Analysis

At Nödinge there were some problems during sampling with radially poorly filled tubes, i.e. the sample had a smaller diameter than the tube. At 5 m about 1-2 mm was missing in the upper tube and around 1 mm in the middle one. At 12.5 m a whole 5 mm were missing in the upper part of the upper tube and 1 mm in the middle tube. This meant that all horizontal stresses acting on the sample was gone which could have had an effect on the testing results. Since the samples where loose in the tubes they can also have experienced mechanical disturbance during transportation and handling. The first sample for triaxial testing from 12.5 m came from the middle tube and was the most affected by this of all the tested samples, however this sample was also stored unconfined for one week. These factors might have contributed to a great deal of the lower shear stress and stiffness in this sample. The 50 mm samples were transported on a vibration-reducing mattress, which could explain some of the difference in shear strength.

As mentioned the clay at 5 m had a very ductile behaviour and a slight green tint indicating that organic material might have been present in the clay. However an ignition test showed an organic content of around 5%, proving that this was not the case.

Previous studies from the test site in Nödinge show that the clay is fairly homogenous. This is proven by the uniform results from the CRS tests in this study. Normalized stress-strain curves for 5 and 12.5 m can be found in Figure 68 and Figure 69. The consistent trend of higher moduli of the 60 mm samples is clearly displayed for both depths. Variations of the normalised curves are very small at 5 m, while larger variations are found at 12.5 m.

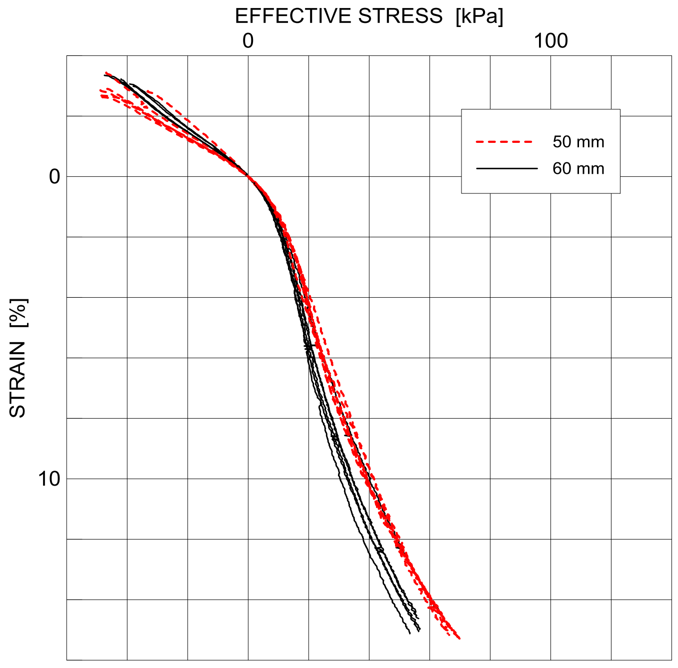

Figure 68 *CRS test stress-strain curves normalized in the preconsolidation pressures, 5 m, Nödinge.*

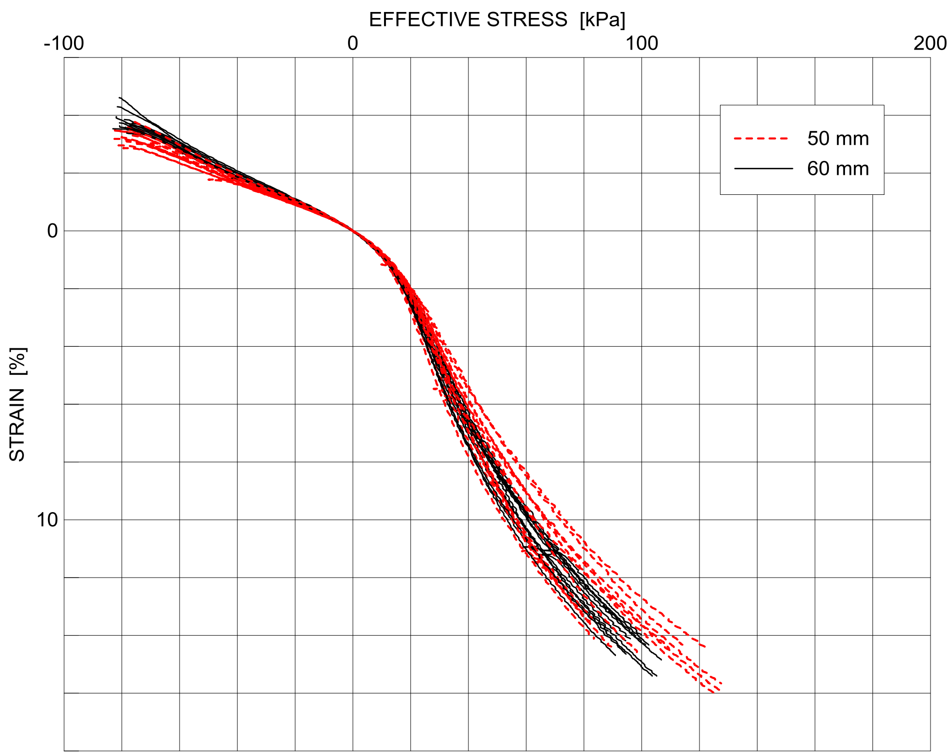

Figure 69 *CRS test stress-strain curves normalized in the preconsolidation pressures, 12.5 m, Nödinge.*

Figure 70 and Figure 71 show the disturbance evaluation of volumetric strain according to Larsson et al (2007). All tests are found in the 'very good to excellent' or 'good to fair' categories. The tests on specimens of the 60 mm sampler have a tendency to be found in the upper part of the test distributions, indicating slightly higher disturbance than the 50 mm samples.

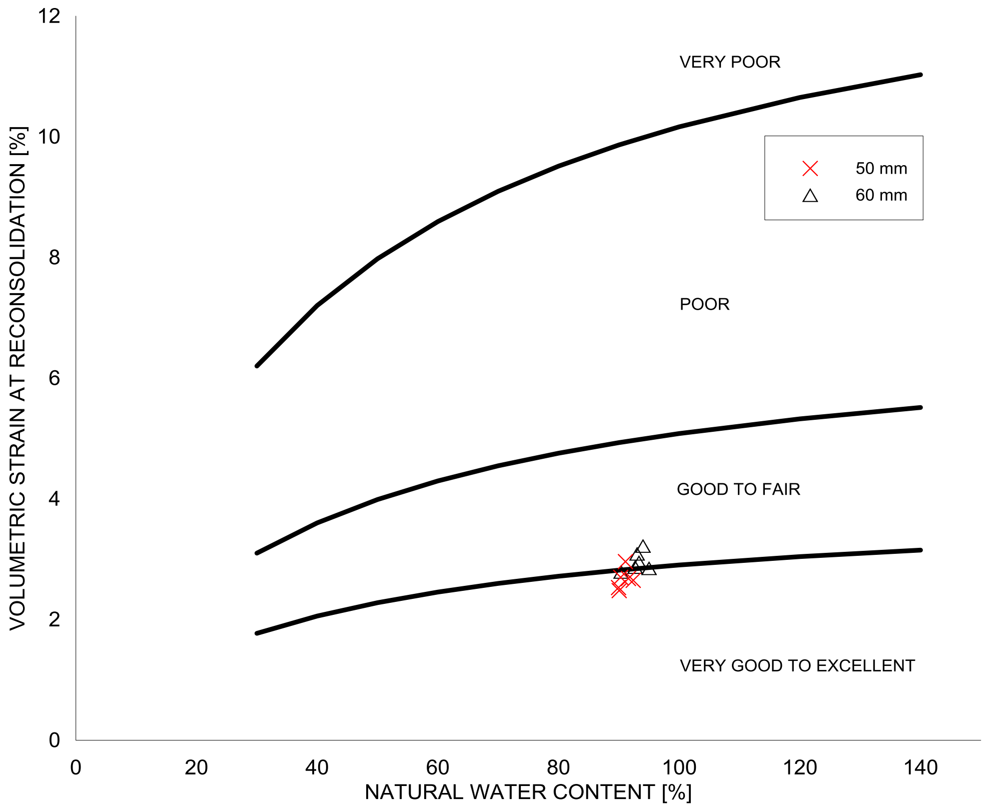

Figure 70 *Volumetric strain at reconsolidation for CRS tests at Nödinge, 5 m (Disturbance evaluation according to Larsson et al, 2007).*

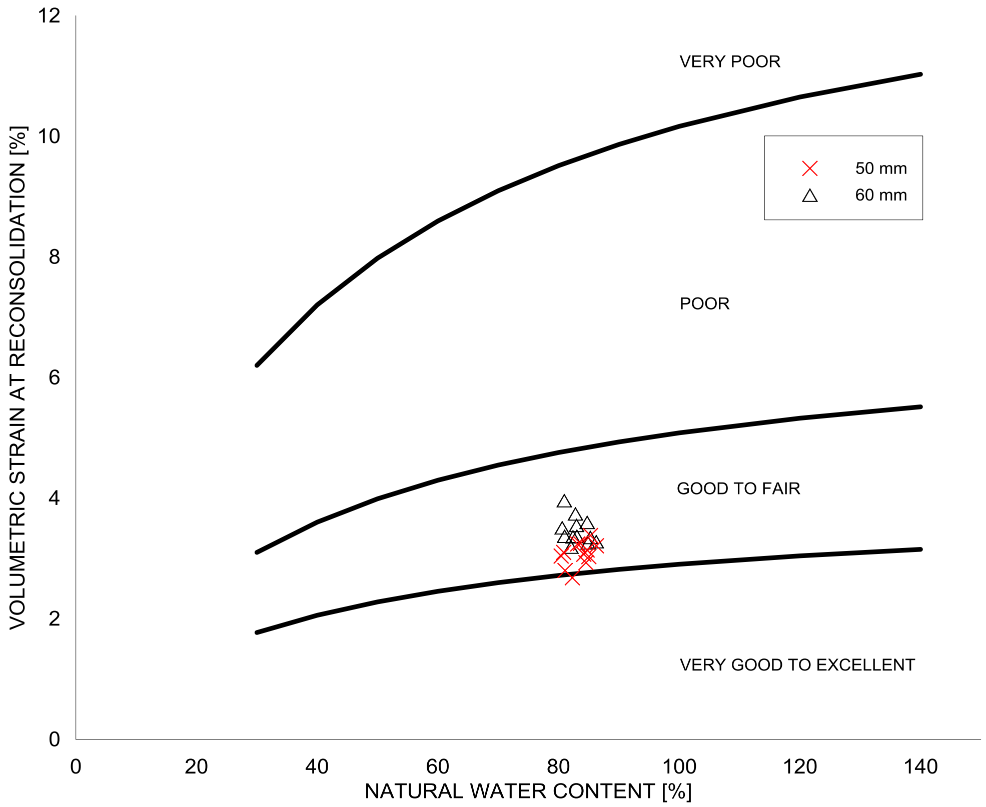

Figure 71 *Volumetric strain at reconsolidation for CRS tests at Nödinge, 12.5 m (Disturbance evaluation according to Larsson et al, 2007).*

In Figure 72 and Figure 73 the time effects on the preconsolidation pressure are analysed. It is hard to find any clear patterns that would indicate that the properties change over time. The patterns of the preconsolidation pressure at 12.5 m may indicate that the scatter and preconsolidation of the 50 mm samples increases with time (Figure 73) which could be explained by oxidation of the samples. Since the outer part of the 60 mm samples are removed before testing, the oxidation would not affect them as much.

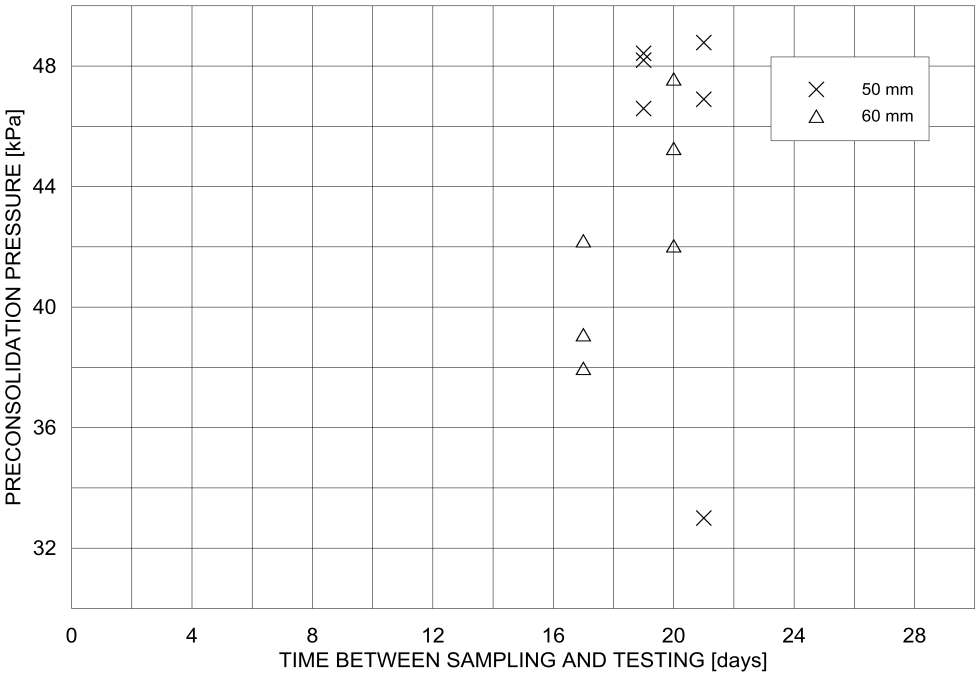

Figure 72 Time dependency of preconsolidation pressures at 5 m, Nödinge.

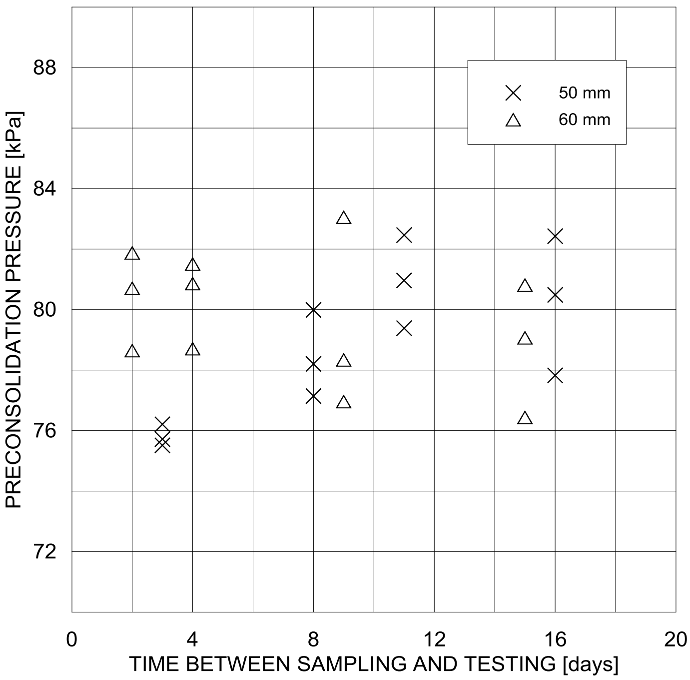

Figure 73 Time dependency of preconsolidation pressures at 12.5 m, Nödinge.

10.5 Statistical analysis

A statistical analysis has been performed on the CRS test results from Nödinge in order to emphasize and visualize the differences and the scatter. Comparisons are made between samples taken by the standard 50 mm sampler and samples from the new 60 mm sampler. Even though the quantity of the test results is very small from a statistical point of view, the trends of the variation can be analysed. In small statistical populations like this, single extreme values have a large impact on the outcome, hence that has been considered.

The statistical tools used in this analysis are mean value, μ, standard deviation, σ, and coefficient of variation, c_v, (see equation 19-21) and for all cases the data is assumed to form a normal distribution. The advantages and disadvantages of using either the standard deviation or the coefficient of variation as a measure of test results variations can be discussed. It should be stressed that since the standard deviation is expressed in the same unit as the data, the values will usually increase with higher data points.

$$\mu = \frac{1}{n}\sum_{i=1}^{n} x_i \tag{19}$$

where n is the number of values in the set.

$$\sigma = \sqrt{\frac{1}{n}\sum_{i=1}^{n}(x_i - \mu)^2} \tag{20}$$

$$c_v = \frac{\sigma}{\mu} \tag{21}$$

The statistical analysis is limited to the CRS test results from Nödinge. Even though many tests have been performed on the clay from Lerum Centrum, there are only 3 tests done on the 50 mm samples from each depth, which would make a statistical comparison very unreliable. The same can be said for the triaxial tests at Regionens Hus and Nödinge.

10.5.1 CRS tests on specimen from 5 m

A statistical analysis on some of the clay properties at 5 m is found in Table 27. The 50 mm tests give higher preconsolidation pressures and plastic moduli and lower volumetric strain, compared to the 60 mm samples. The variation is consistently higher for the 50 mm samples. However, one of the 50 mm samples is identified as disturbed and this particular sample has a large effect on the variation because of the small test population. Variations in permeability are large, which is mainly due to the imprecise evaluation method used in CRS testing.

Table 27 Statistical analysis on clay properties from CRS tests, Nödinge, 5 m.

	mean values		**standard deviation**		**coefficient of variation**	
	50 mm	**60 mm**	**50 mm**	**60 mm**	**50 mm**	**60 mm**
σ'_c [kPa]	45.3	42.4	5.56	3.31	12%	8%
M_L [kPa]	272	240	37	23	13%	10%
w_N [%]	91	93	0.88	1.42	1%	2%
ρ [t/m^3]	1.50	1.50	0.02	0.01	1%	1%
ε_{VO} [%]	2.67	2.96	0.15	0.15	6%	5%
k [m/s]	1.5E-09	1.3E-09	4.99E-10	1.80E-10	34%	14%

Mean values and standard deviations along the CRS stress-strain curves can be found in Figure 74 to Figure 77. For stresses below preconsolidation pressure, the standard deviation is significantly lower for the 60 mm samples than for the 50 mm samples, while being similar at higher stresses. As mentioned above, the 50 mm standard deviation values are affected by an apparently disturbed sample.

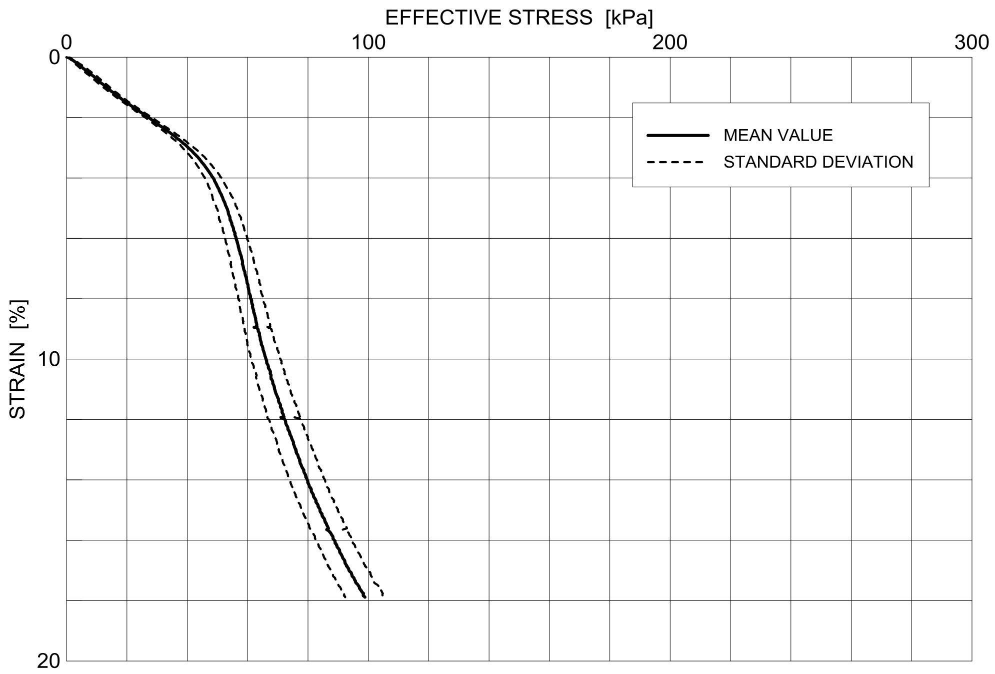

Figure 74 Mean value of CRS test stress-strain curves for 50 mm samples at 5 m, Nödinge. Range for the distance of ± one standard deviation marked.

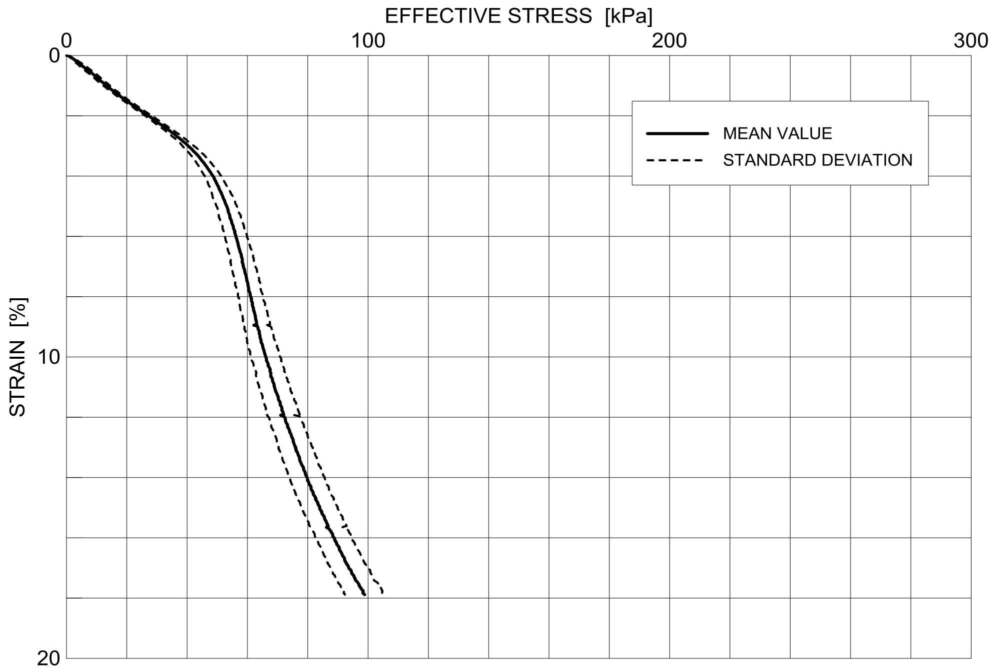

Figure 75　Mean value of CRS test stress-strain curves for 60 mm samples at 5 m, Nödinge. Range for the distance of ± one standard deviation marked.

The changes in standard deviation can also be presented by its own for different strains, as can be seen in Figure 76. To find out how the variation differs in relation to the mean values, the coefficient of variation is presented in Figure 77.

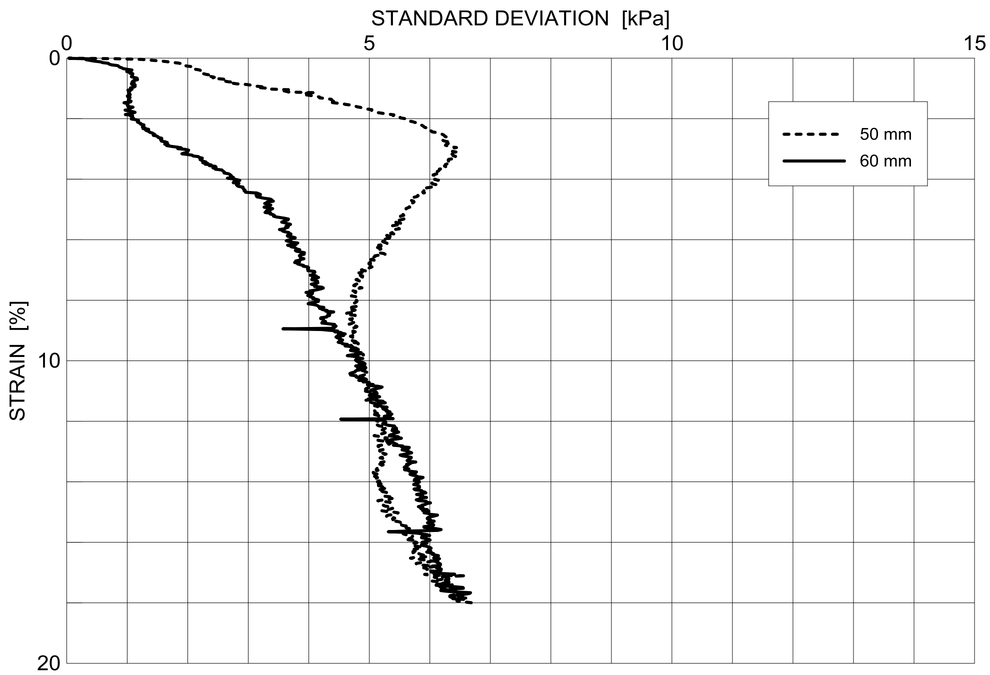

Figure 76　Standard deviation of the CRS test stress-strain curves at 5 m, Nödinge.

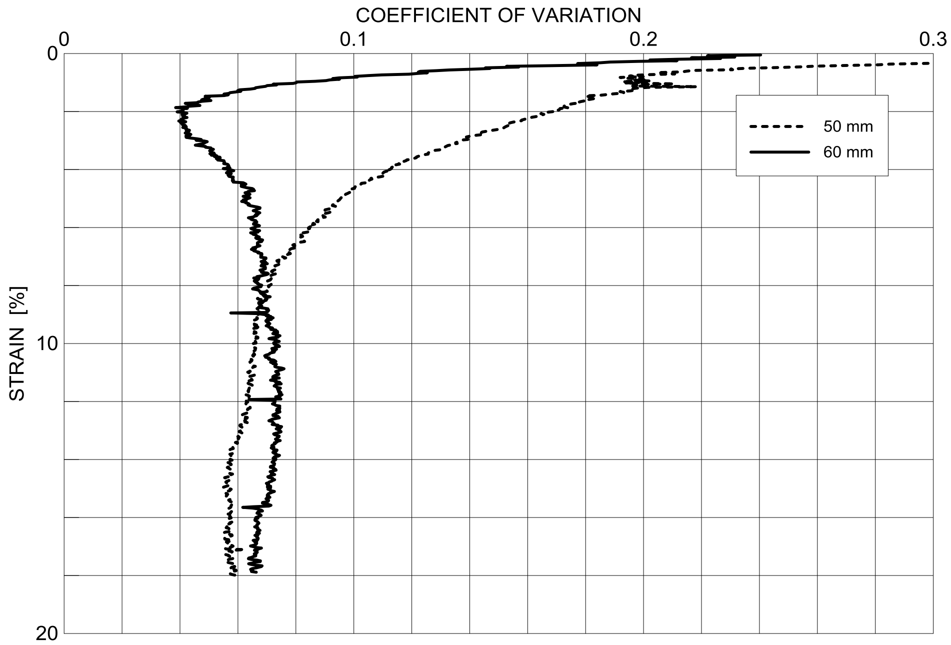

Figure 77 Coefficient of variation for CRS test stress-strain curves at 5 m, Nödinge.

10.5.2 CRS tests on specimen from 12.5 m

The statistical analysis of some clay properties at 12.5 m in Nödinge can be found in Table 28. Mean values are very similar comparing 50 mm and 60 mm samples. The evaluated preconsolidation pressure is almost the same, while the plastic modulus is higher and the volumetric strain is lower for the 50 mm samples. Variations are very similar for the different diameters, though the scatter in the 50 mm samples is slightly larger at preconsolidation pressure and elastic modulus.

Table 28 Statistical analysis on clay properties from CRS tests, Nödinge, 12.5 m.

	mean values		**standard deviation**		**coefficient of variation**	
	50 mm	**60 mm**	**50 mm**	**60 mm**	**50 mm**	**60 mm**
σ'_c [kPa]	79	80	2.4	1.9	3%	2%
M_L [kPa]	465	421	51	35	11%	8%
w_N [%]	83	83	1.83	1.77	2%	2%
ρ [t/m^3]	1.54	1.54	0.01	0.02	1%	1%
ε_{VO} [%]	3.07	3.46	0.19	0.21	6%	6%
k [m/s]	7.6E-10	8.1E-10	1.23E-10	1.64E-10	16%	20%

The mean values and the standard deviation range of all 24 CRS stress-strain curves from 12.5 m at Nödinge test site can be found in Figure 78 and Figure 79. It is evident that scatter is larger for the 50 mm samples compared to the 60 mm samples, when looking at stresses above the preconsolidation pressure.

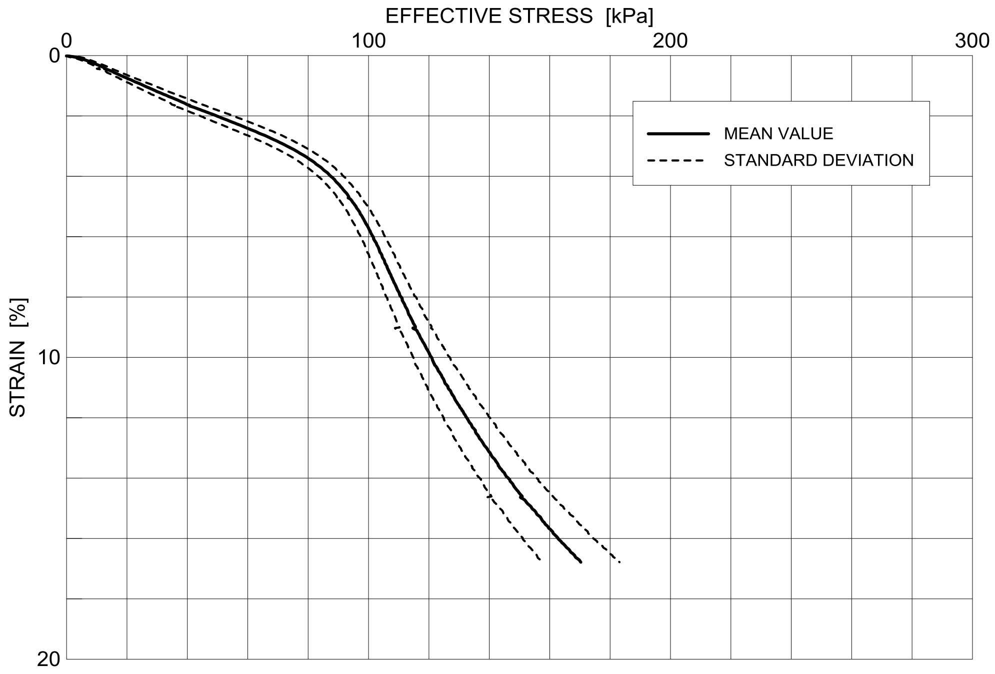

Figure 78 *Mean value of CRS test stress-strain curves for 50 mm samples at 12.5 m, Nödinge. Range for the distance of ± one standard deviation marked.*

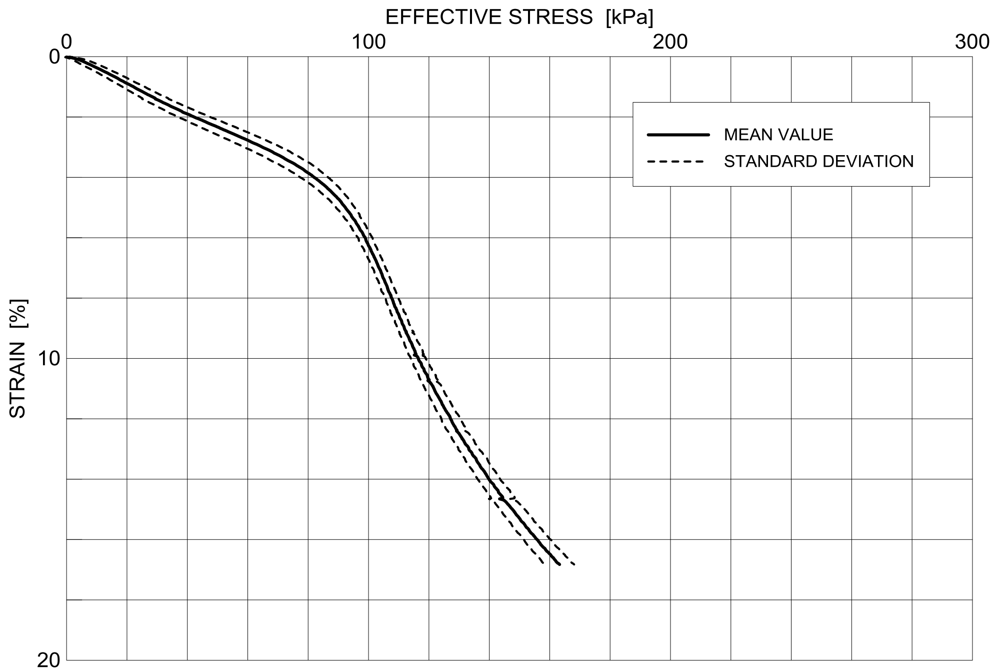

Figure 79 *Mean value of CRS test stress-strain curves for 60 mm samples at 12.5 m, Nödinge. Range for the distance of ± one standard deviation marked.*

The differences in scatter are further clarified in Figure 80 and Figure 81, where the standard deviations and coefficients of variation are plotted against the strain.

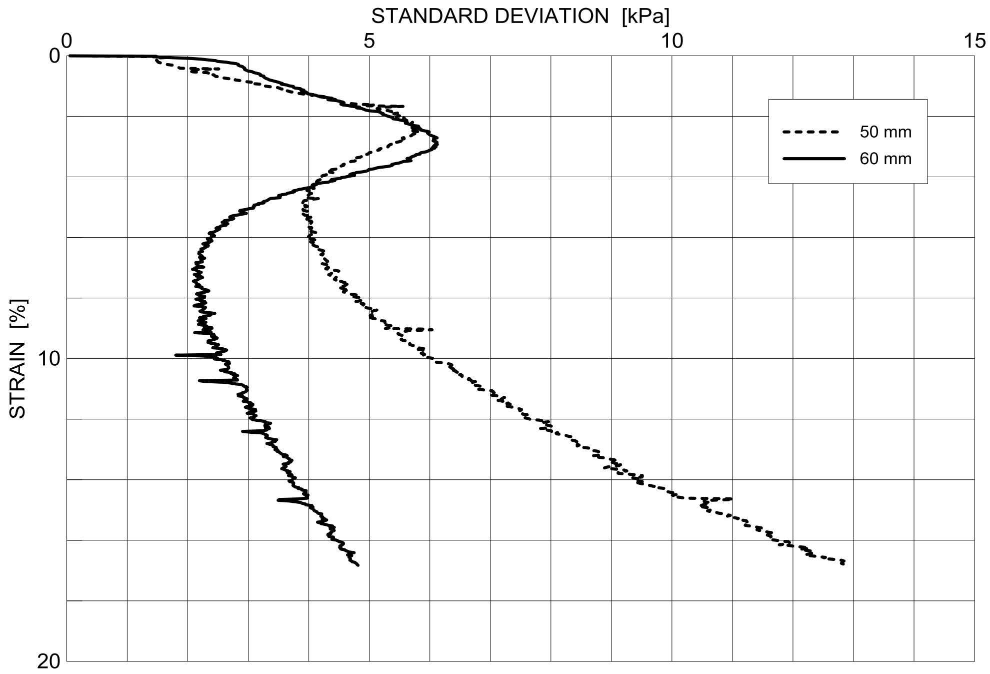

Figure 80 Standard deviation of the CRS test stress-strain curves at 12.5 m, Nödinge.

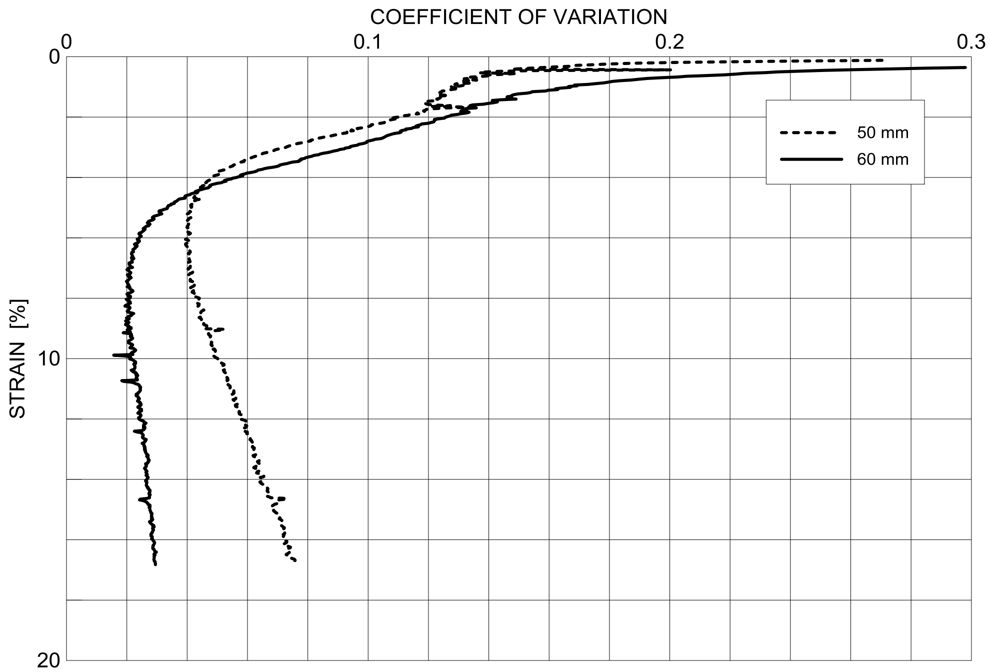

Figure 81 Coefficient of variation for CRS test stress-strain curves at 12.5 m, Nödinge.

11 Analysis

This chapter presents a general analysis of the test results, the sampling and the laboratory testing as well as an evaluation of the 60 mm sampler.

11.1 CRS tests

In general, the CRS tests show good quality. The uniformity of the test results indicates that the test preparations in the laboratory have been done in a proper way. All the tests were performed by the same person, though sampled by two different rigs and staff. Some of the results show a scatter that indicates that the clay is inhomogeneous in its natural state.

The different methods for evaluating the sample quality show no unison results. The most obvious advantage of the 60 mm samples is decreased scatter in the test results.

In general, the behaviour of the stress-strain curves differs depending on the sampling depth. Shallow samples show a more distinct knee between the elastic and the plastic phase. Deep samples are subjective to a greater stress reduction during sampling compared to shallow samples, which may lead to an increased rate of disturbance. Another factor that may affect this phenomenon is the difference in load rate of CRS tests. Since the CRS tests, as the name indicates, deforms the sample with a constant rate, the stress rate will differ depending on the clay properties, with deep samples getting a much faster stress mobilisation compared to shallow samples.

A trend when looking at all CRS tests in this study is that the 50 mm samples tend to form a convex stress-strain curve at the very beginning of the test cycle, while the 60 mm samples in general have a concave behaviour in the same stress range. Possible reasons for this behaviour could be the differences in laboratory handling or differences in test equipment used. It is important to discuss how the samples are affected by the trimming in the laboratory. Even though the theory states that the quality of the sample will improve when the outer, most disturbed, part of the sample is removed, it is likely that the samples will be subjected to additional disturbance when trimmed.

A trend in all CRS tests is that M_0 is lower for the 60 mm samples than the 50 mm samples, which is contrary to what is expected from theory. According to Larsson (1981), one of the reasons for this can be an increased stress relief during sampling due to negative air pressures beneath the sampler. The force needed to dislodge the sample from the ground increases with the new sampler.

The horizontal forces may differ between 50 mm and 60 mm tests due to differences in test equipment. In a few cases it seemed like horizontal forces may have been added when the 50 mm samples where inserted in the PFTE rings.

11.2 Triaxial tests

Based on the results from the triaxial tests on the new samples and the comparison with the 50 mm results, no clear conclusion can be made determining which is the best of the two samplers. At Regionens Hus the 60 mm samples had higher shear strength in two out of three cases. It was significantly larger only in one of those and in that one the 50 mm samples were up to six months old, making for an unfair comparison. At Nödinge, the 60 mm samples were clearly disturbed which could be seen as lower shear strengths at higher strains.

11.3 Shear wave velocity

Based on the results from bender testing on the 60 mm samples and the work done by Wood (n.a.), it is clear that the sample quality decrease with time. Even after one week the shear wave velocity has decreased significantly and after five months it is down to two thirds of the initial measurements. However if the samples are trimmed, the most disturbed part, in this case through oxidation, can be removed and the shear wave velocity will increase. At 30 m the increase is over 20 %. These results show that trimming samples could give a higher quality and thus the 60 mm sampler could be a good alternative if the samples can't be tested soon after sampling. Although it is also possible to trim 50 mm samples to 38 mm and get the same effect. The results also show that if the samples are fresh the 50 mm sampler take samples of very high quality making the extra work associated with the 60 mm samples a bit redundant.

11.4 Evaluation of 60 mm sampler

During the study, the 60 mm sampler have been tested and evaluated. Some of the findings and issues are presented in the following passages.

11.4.1 Feasibility of sampler

In terms of sampling, the new 60 mm sampler is almost as easy to handle as the standard sampler, which makes it superior to large diameter samplers in that aspect. Another advantage of the new sampler compared to large-diameter samplers is the much lower sampling cost. Except for the investment cost, the sampling with the new sampler is nearly as cost-efficient as the standard sampler, though the laboratory time is increased due to the trimming of the samples. Since the effect of oxidation and aging seem to be reduced when the sample diameter is increased, the 60 mm samples could be beneficial if the samples have to be stored for an extended period of time. However, according to this study, the differences in result between the investigated samplers, when there is any, is too small to make a change feasible for standard laboratory tests.

11.4.2 Identified issues

While studying the 60 mm samples in the laboratory, clean shear surfaces were detected in almost all upper tubes. An example of the shear surface can be seen in Figure 82. In most cases the surface appears at the top of the upper tubes, but is in some cases found further down (Figure 83). Since the clay is striped in some of the samples, it is possible to identify the surface as an active failure.

At first, this phenomenon only seemed to appear in the 60 mm specimen, but has later been found also in several 50 mm upper tubes.

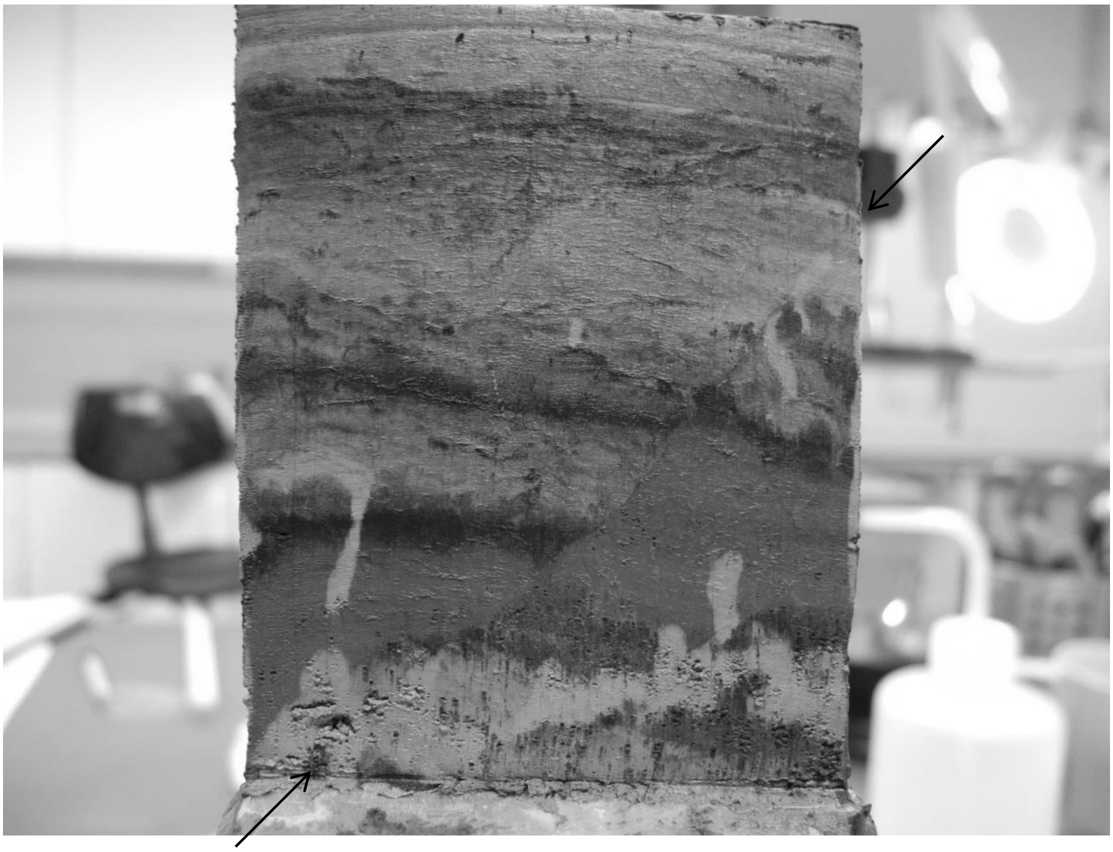

Figure 82 Shear surface in the upper tube at 15 m, Regionens Hus. Note that the sample is placed upside down.

A plausible hypothesis on why the failure appears could be that the clay is sheared beneath the sampler when the sampler is driven down through the ground (as described in section 4.1). For a larger sampler diameter, the sampler will affect the soil further down, thus creating a visible shear surface in the upper tube. One factor that could have an impact on this phenomenon is the speed at which the sampler is pushed down and displacing the soil. The clean, almost perfect failure is unexpected, though. A more expected scenario would have been to find remoulded clay at the top of the upper samples to a higher extent than before.

When the stresses are large enough to create a shear surface in the upper tube, it is not unlikely that deeper parts of the samples have been affected by large stresses too. The possibility of middle tube samples being affected by increased stress levels should be considered.

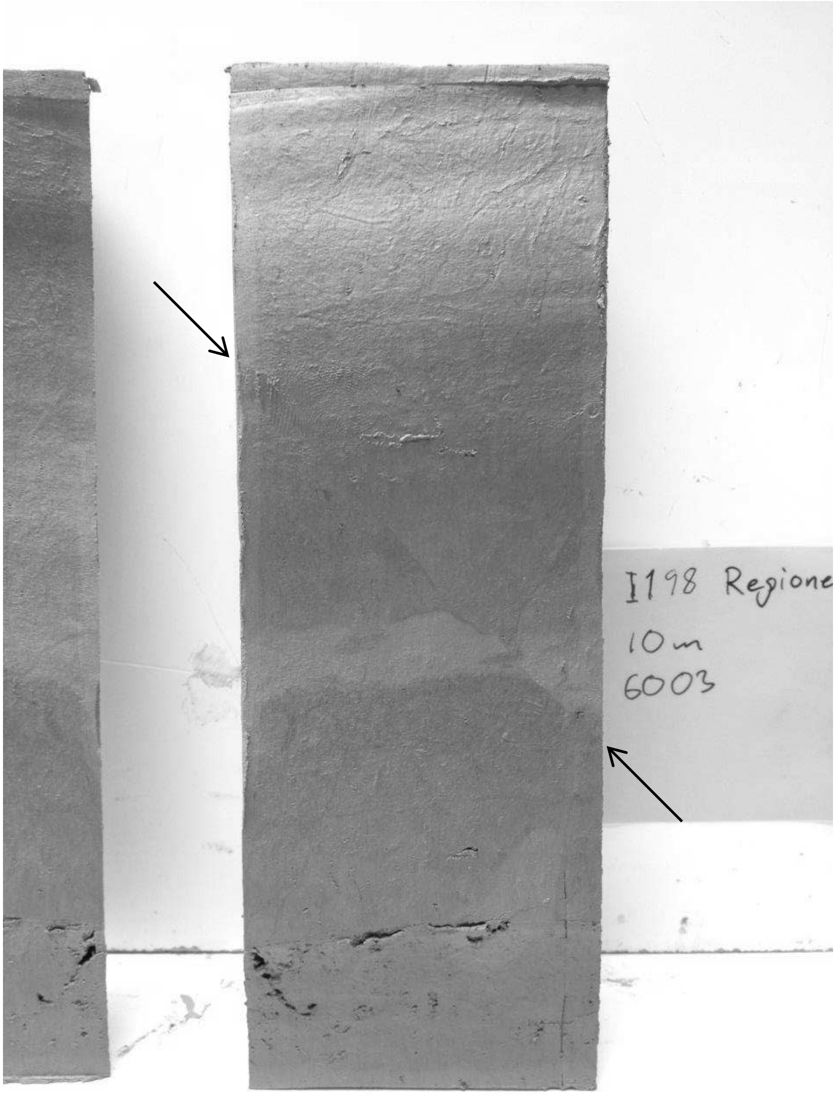

Figure 83 Shear surface in the middle of the upper tube at 10 m, Regionens Hus. Note that the sample is placed upside down.

According to the data specifications of the new sampler, the inside clearance was reported to be the same as for the standard sampler, i.e. 0.4 %. However, measurements on one of the cutting shoes indicate a more narrow opening than specified, which almost results in a doubling of the inside clearance. How this affects the sample quality has not been studied, but according to theory, the inside wall friction would decrease, while the horizontal stress release would increase. The accuracy of the inside clearance of the standard 50 mm sampler have not been investigated in this project.

11.4.3 Related equipment

New sampling tubes have been produced for the new sampler. The inside surface is slightly rough, increasing the friction during sampling. The thickness of the material is decreased compared to old 50 mm tubes, which in addition to the increased diameter, have led to a lowered stiffness, making them somewhat flexible. According to the method description (SGF, 2009) the tubes should not be flexible.

The new tubes were tested in order to find out if they were water tight and if they absorbed any water. According to the test, the tubes are water tight and the water absorbed into the fabric is negligible. However, the lids are made of plastic and do not seal the tubes as effectively as rubber lids. This could affect clay properties that are sensitive to dehydration of the samples. Samples that had been stored for a long period were highly affected by oxidation. However, the same can be said about the 50 mm samples with rubber lids.

The change of CRS equipment may have lowered the friction during tests. The observation that the 50 mm stress-strain curves generally placed above the 60 mm curves could possibly be a sign of frictional differences. One advantage of trimming the samples directly into the CRS oedometer cells instead of using the PFTE rings is that the sample diameter strictly matches the inside diameter of the cell. The PFTE is highly sensitive to temperature changes, which may worsen the fit between the sample and the cell.

12 Discussion and conclusions

The work in this study has been based on previously made studies addressing the issue of sample disturbance and investigations of different clay samplers, as can be read in chapters 4 and 5. The results of these studies show that a larger sampler can usually achieve higher quality samples (Lunne, Berre & Strandvik, 1999). However in some cases (Kallstenius, 1958 and Hagberg, Long & El Hadj, 2007) no significant increase in quality can be seen, as is the result in this thesis. If this is due to the design of the new sampler or other factors is not fully investigated. To be able to make this distinction, further studies are needed where the 50 mm and 60 mm samples are tested under more similar circumstances. Based on this study some suggestions for improvements are made in the following text.

At Lerum C both 50 mm and 60 mm samples were taken and tested making for a good comparative study. However the clay was very heterogeneous resulting in large differences even in samples taken with the same sampler at the same depth. Also, the effects of trimming the 60 mm samples are unknown, making the results somewhat unreliable. At Regionens Hus only 60 mm samples were taken and then compared to results from previous testing of 50 mm samples. The conditions during both sampling and testing of the 50 mm samples are not known. The sampling procedure at Nödinge was supposed to reduce these uncertainties as both 50 mm and 60 mm samples were taken and the clay there is very homogenous. Samples were taken from two depths in five boreholes to get a more substantial basis for the comparison. However the laboratory equipment for CRS testing still differs slightly between the different sample sizes and the results from triaxial testing were also compared to previously made tests. It is therefore recommended that 50 mm samples are taken for triaxial testing for a more fair comparison. To reduce uncertainties in the results it is also recommended that the same machine operator take all samples and that the same laboratory and equipment is used for all tests.

Since the dimensions are changed for the new sampler, the stress situation during sampling will also change. It is reasonable to think that the standard sampling method should be changed in order to compensate for the dimension changes. In this study the sampling procedure was the same for both the 50 mm and 60 mm sampler, which could explain some of the differences in the results. Especially since the clays at Lerum C and Nödinge are sensitive to disturbance. One way to compensate for the stress changes caused by a larger diameter could be to lower the sampling speed during all stages. The drill rig operator should also have knowledge about the effects of sample disturbance and the importance of following the standard method for sampling.

Both this and previous studies have shown that the age of samples is an important factor since the quality decrease over time due to oxidation and stress relief. The bender system and suction probe are good ways to measure quality of the sample and is considered to be a suitable complement to the normal testing equipment. By using these it is possible to see how the storage time has affected the sample. Lunnes quality measurement can only assess the quality after the sample has been tested. It might also indicate high quality even if the sample is dehydrated, making it an unreliable method for older samples.

The need for high quality samples for geotechnical designs make investigations in the subject very important. Considering the analysed results from this study showing lower to slightly higher quality for the 60 mm samples while keeping in mind the extra laboratory and field work connected to the new sampler, a change of standard sampler to a larger diameter cannot be recommended at this point. However further studies on sample quality are recommended to sort out any uncertainties that has arisen throughout this thesis.

13 Appendix contents

The Appendices are found on the attached CD.

N.B. texts in the Appendices are in Swedish.

14 References

Bjerrum, L. (1967). *Engineering Geology of Norweigan Normally-consolidated Marine Clays as Related to Settlements of Buildings*, The 7th Rankine Lecture, Geotechnique Vol 17, No 2, pp 81-118

Bjerrum, L. (1973). *Problems of soil mechanics and construction on soft clays and structurally unstable soils (collapsible, expansive and others).* Proceedings of the eighth International Conference on Soil Mechanics and Foundation Engineering. Session 4. pp. 111-159.

Bohusgeo (2012). *Rapport 2012-04-03, Nytt Regionens Hus Göteborg, Gullbergsvass 703:061, Göteborg, Planerad om- och nybyggnad, Geoteknik, Fält- och laboratorieresultat.*

Borros AB (1964). *Anvisningar för provtagning med standardkolvborr stII, typ Borro.* (Directions for sampling with standard piston sampler stII, type Borro. In Swedish.) Solna

Brouwer, JJ M. (2007). *In-situ soil testing*, IHS BRE Press, East Sussex

Clayton, C.R.I., Matthews, M.C., Simons, N.E. (1995). *Site Investigation.* Second Edition. Department of Civil Engineering, University of Surrey.

Craig, R.F. (2004). *Craig's soil mechanics.* Seventh edition. Taylor & Francis

Hagberg, K., Long, M., El Hadj, N. (2007). *Experience of 54 mm and 75 mm piston sampling in Norwegian soft clays.* 14th European conference on soil mechanics and foundation engineering, Proceedings, vol. 3, pp 1643-1649.

Hvorslev, M.J. (1949). *Subsurface exploration and sampling of soils for civil engineering purposes.* Report on a research project of the Committee on Sampling and Testing. Soil Mechanics and Foundation Division, American Society of Civil Engineers. Vicksburg, MS: Waterways Experiment Station.

Kallstenius, T. (1958). *Mechanical disturbance in clay samples taken with piston samplers.* Proceedings no. 16. Royal Swedish Geotechnical Institute. Stockholm: Ivar Hæggströms Boktryckeri AB.

Kallstenius, T. (1963). *Studies on clay samples taken with standard piston sampler.* Proceedings no. 21. Swedish Geotechnical Institute. Stockholm: Ivar Hæggströms Boktryckeri AB.

Karlsson, R., SGF (1981) *Consistency limits.* Performance and interpretation of laboratory investigations, part 6. D9:81. Stockholm: Spångbergs Tryckerier AB.

Kompetenscentrum Infrastruktur (n.a.). *En kortkurs om triaxialförsök på främst normalkonsoliderade och svagt överkonsoliderade leror.*

Ladd C.C., DeGroot D.J. (2003), *Recommended practice for soft ground site charaterization*: Arthur Casagrande Lecture, Proc. 12th Panamerican Conf. on Soil Mechanics and Geotech. Eng., MIT , Vol. 1, 3–57.

Larsson, R. (1981). *Får vi några ostörda prover med standardkolvborren? En jämförelse mellan prover tagna med standardkolvborr och Laval-provtagaren.* (Do we get any undisturbed samples with the standard piston sampler? A comparison between samples taken with the standard piston sampler and the Laval sampler. In Swedish.) Varia 60. Statens Geotekniska Institut. Linköping: Klintland Grafiska.

Larsson, R., Sällfors, G., Bengtsson, P-E., Alén, C., Bergdahl, U., Eriksson, L. (2007). *Skjuvhållfasthet – utvärdering i kohesionsjord.* (Shear strength – evaluation in cohesive soil. In Swedish.) Information 3, SGI. Linköping.

Larsson, R. (2008). *Jords egenskaper.* (Soil properties. In Swedish.) SGI information 1. Linköping.

Larsson, R., Åhnberg, H., Löfroth, H. (2012). *Stor provtagare för ostörda prover i lös finkornig jord.* (Large sampler for undisturbed samples in lose fine-grained soil. In Swedish.) Bygg & Teknik, Nr 1, 2012, January. pp 43-48.

Long, M., El Hadj, N., Hagberg, K. (2009*). Quality of conventional fixed piston samples of Norwegian soft clay.* ASCE Journal of Geotechnical and Geoenvironmental Engineering, 135 (2), pp 185-198.

Lunne, T., Berre, T. and Strandvik, S. (1999). *Sample disturbance effects in soft low plastic Norwegian clay.* Publikasjon 204, Norges geotekniske institutt, NGI. Oslo.

Löfroth, H. (2012). *Sampling in normal and high sensitive clay – a comparison of results from specimens taken with the SGI large-diameter sampler and the standard piston sampler St II.* Varia 637. Swedish Geotechnical institute. Linköping.

Magnusson, O., Sällfors, G., Larsson, R. (1989). *Ödometerförsök enligt CRS-metoden.* (Oedometer tests according to the CRS method. In Swedish.) Rapport R44:1989. Statens råd för byggnadsforskning. Stockholm: Svenskt Tryck.

Ou, C.Y. (2006). *Deep Excavation Theory and Practice.* Leiden: Taylor & Francis.

Piriyakul, K. and Haegeman, W. (2005). *Measurement of anisotropic shear wave propagation by bender elements in overconsolidated clay.* 3rd Biot Conference on Poromechanics, Oklahoma, USA, May 24-27. pp 561-566.

Rankka, K., Andersson-Sköld, Y., Hultén, C., Larsson, R., Leroux, V., Dahlin, T. (2004). *Quick clay in Sweden.* Report 65. Swedish Geotechnical Institute. Linköping

SGF (2009). *Metodbeskrivning för provtagning med standardkolvprovtagare, ostörd provtagning i finkornig jord.* (Method description for sampling with the standard piston sampler, undisturbed sampling in fine-grained soil. In Swedish.) Rapport 1:2009. Svenska Geotekniska Föreningen. Linköping.

Simons, N., Menzies, B. and Matthews M. (2002). *A short course in geotechnical site investigation.* London: Thomas Telford.

SIS (1991). *Geotekniska provningsmetoder - Kompressionsegenskaper – Ödometerförsök, CRS-försök – Kohesionsjord.* (Geotechnical test methods – Compression properties – Oedometer test, CRS test – Cohesive soil. In Swedish.) Svensk standard *SS 02 71 26:1991*. Stockholm: SIS Tryckeri.

SIS (2006). *Geotechnical investigation and testing – Sampling methods and groundwater measurements – Part 1: Technical principles for execution (ISO 22475-1:2006).* Svensk standard *SS-EN ISO 22475-1:2006*.

Skempton, A.W. and Sowa, V.A. (1963). *The behaviour of saturated clays during sampling and testing.* Geotechnique, Vol. 13, pp 269-290.

Swedish Committee on Piston Sampling (1961). *Standard piston sampling.* Proceedings no. 19. Swedish Geotechnical Institute. Stockholm: Ivar Hæggströms Boktryckeri AB.

Wiesel, C-E., Hansbo, S., Broms, B. (1985). *Skjuvhållfasthet.* (Shear strength. In Swedish.) Geotekniska laboratorieanvisningar, del 9. T2:1985. Byggforskningsrådet.

Wood, T. (n.a). *Jordmodellering/hållfasthets och deformationsegenskaper – med speciell fokus på provtagningskvaliteten vid fält- och laboratorieundersökningar.* (Soil modeling / strength and deformation behaviour - with special focus on sampling quality in field and laboratory investigations. In Swedish) [Unpublished PhD thesis] Chalmers University of Technology.

Yu, H-S. (2000). *Cavity Expansion Methods in Geomechanics*, Elastic-Perfectly Plastic Solutions, pp 32-94.