REPORT

Norwegian GeoTest Sites (NGTS)

LABORATORY PROCEDURES AND STANDARDS
FOR THE NGTS PROJECT

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Project

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Client:	RCN
Client contact person:	Herman Fabrot
Contract reference:	

for NGI

Project manager:	Jean-Sebastien L'Heureux
Prepared by:	Ragnhild Christin Hansen, Morten Andreas Sjursen, Nouri El Hadj, Anatoly Sinitsyn, Arne Instanes and Karl Ivar Volden Kvisvik
Reviewed by:	Tom Lunne, Ørjan Nerland, Priscilla Paniagua

Summary

This report gives an overview of the laboratory testing procedures used in the NGTS project.

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Review and reference page

1 Introduction

The Norwegian Geo-Test Sites (NGTS) research infrastructure, with funding from The Research Council of Norway, creates a national research test site facility for geotechnical research. The five national test sites are located in Norway and on Svalbard. The research consortium consists of NGI and NTNU, SINTEF/UNIS and NPRA (Norwegian Public Roads Administration). The research project aims at developing five sites as field laboratories for the testing and verification of innovative soil investigation and testing methods. The sites cover the soil conditions of soft clay, quick clay, silt, sand and permafrost. At the end of the project, the test sites will serve as reference sites for the industry, public authorities, research organizations and academia where benchmarked data can be used by several generations of scientists and engineers to develop soil material models, new investigation methods, new foundation solutions and advance the state-of-the-art. The five sites will be operative for at least 10-20 yrs.

To ensure quality of the data generated in the project and to ensure as much uniformity of the results as possible it is important that all partners performing work at the sites follow the same test procedures and standards.

The present report gives an overview of the main laboratory testing methods and techniques to be used in the NGTS project. Laboratory tests on the NGTS sites shall be performed according to the referenced standards and requirements listed in the present report.

2 Laboratory testing procedures

2.1 Index and chemical tests

***Method overview** NA-S5*		
Type of test – measuring principle (technique/instrument)	**Test material**	**Method reference ***
Determination of water content	soil	NS-EN ISO 17892-1:2014
Determination of liquid limit	clay	NS 8002 : 1982
Determination of plastic limit	clay	NS 8003 : 1982
Determination of percussion liquid limit	clay	NS 8001 : 1982
Determination of undrained shear strength with fall cone	clay	NS 8015 : 1988
Determination of undrained shear strength with unconfined compression test	clay	NS 8016 : 1988 ETC5-2.97
Determination of grain size distribution with sieving	sand/silt	NS 8005 : 1990
Determination of grain size distribution with hydrometer	sand/silt/clay	ISO/TS 17892-4 : 2004 NPRA R210, method no. 214
Determination of grain size distribution with "Falling Drop"-method	sand/silt/clay	NGI in-house procedure, (Moum, 1965)
Determination of unit weight – the pyknometer method	soil	NS-EN ISO 17892-3:2014
Determination of max and min dry unit weight	sand/silt	NGI in-house procedure
Determination of unit weight	soil	NS-EN ISO 17892-2:2014
Performation of oedometer test (incremental loading)	soil	NS 8017:1991
Performation of oedometer test (continous loading)	soil	NS 8018:1993
Performation of permeability test (triax-, DSS- and oedometer cell)	soil	CEN ISO/TS 17892-11, ("constant head")
Determination of organic content	soil	NS-EN 1997-2:2007, Annex N, Annex X.4.2.2
Direct simple shear test	soil	CEN ISO/TS 17892-10
Triaxial testing (CAUc and CAUe)	soil	CEN ISO/TS 17892-9
Gmax tests with bender elements	soil	ISO 19901-8:2014
Determination of salt content: salinity	soil	ASTM 4542-15
Determination of thermal conductivity (ikke tilgang til pros.)	soil	ASTM D5334-00 ASTM D5930-97

Method overview NA-S5		
Type of test – measuring principle (technique/instrument)	**Test material**	**Method reference ***
Determination of heat capacity (ikke tilgang til pros.)	soil	ASTM D4611-16
Transportation, preparation and storage of frozen soil samples for laboratory testing	Frozen soil	ASTM STP 599
Sampling, machining and testing of naturally frozen soils	Frozen soil	(Still et al., 2013b)
Classification of frozen soils	Frozen soil	ASTM D4083-07
Standard terminology relating to frozen soil and rock	Frozen soil	ASTM D7099
The initial freezing temperature	Frozen soil	Probably will be elaborated by NGTS project
Creep properties of frozen soil samples by uniaxial compression	Frozen soil	Methodology needs to be selected. Brief description is given in (Andersland, Ladanyi, 2004). Moscow State University has elaborated a methodology as well
Strength properties of frozen soil at a constant rate of strain	Frozen soil	ASTM D5520
Standard test for methods for frost heave and thaw weakening susceptibility of soil	Soil	ASTM D7300-11
Standard test method for determining the effect of freeze-thaw on hydraulic conductivity of compacted or intact soil	Soil	ASTM D5918-13
Creep testing of frozen soils under triaxial (dynamic and static) test conditions	Frozen soil	ASTM D6035/D6035-M
Shear stress test	Frozen soil	(Kornfield, Zubeck, 2013)
Cyclic compression test	Frozen soil	Methodologies are outlined in (Oestgaard, Zubeck, 2013), some methods are described in (Aksenov, 2008)
Constant-stress creep test	Frozen soil	
Compressibility of thawing soils	Frozen soil	
Relaxation test	Frozen soil	
Thaw consolidation test	Soil	
Segregation potential of soils	Frozen soil	(Konrad, 1987)

2.1.1 Water content

General

Water content (w) is the ratio of the mass of free water to the mass of dry soil expressed as a percentage of the mass of solids. The method is described in detail in NS-EN ISO17892 Part 1.

Procedure

The water content is found by weighing a representative part of the soil in a drying oven maintained at 105°C to 110°C and shall be dried to a constant mass. In most cases, drying a fine soil for 16 h is sufficient.

2.1.2 Atterberg limits

General

Liquid limit (w_L) and plastic limit (w_P) are the highest and lowest water contents, respectively, at which the remoulded soil material is in a plastic state. Standard method described in NS 8003 is used to determine w_P and standard methods described in NS 8001 and 8002 are used to determine w_L, by Casagrande and Single Point method respectively.

Procedures

The percussion liquid limit is the water content in the soil at a certain consistency. The test is conducted using Casagrandes liquid limit apparatus. The method is described in NS 8001, Part 5.2 by means of strokes conducted at the same water content. When the number of strokes falls between 15 and 41, the water content of the sample is determined and the percussion liquid limit can be calculated according to the Standard.

The liquid limit, w_L, is performed according to the Single Point method as described in NS 8002, Part 5.2 by means of a 60 g fall cone. When the cone penetration falls between 7.0 and 15.0 mm, the water content of the sample is determined and the Liquid limit can be calculated according to the Standard.

The plastic limit, w_p, is performed according to NS 8003. The plastic limit is defined as the water content at which a tread of soil with a diameter of 3.2 mm crumbles.

2.1.3 Undrained shear strength from fall cone

General

The fall cone test is used to establish a quick measure of the undrained shear strength of the soil, both the undisturbed and the remoulded strength. The method is described in detail in NS 8015.

Procedure

The fall cone apparatus is produced by Geonor A/S. It measures the penetration of a cone into the specimen after it has been released from an initial stationary position with the tip of the apex at the surface of the specimen. Cones with different apex angles and masses are used. For hard clays, a 400 g cone with an apex angle of 30° is used, and for soft clays, a 100 g cone with an apex of 30°, a 60 g cone with an apex angle of 60°, or a 10 g cone with an apex of 60° is used. The calibration curve provided by Geonor is used.

To determine the sensitivity of a soil in the laboratory, a fall cone test is conducted on an undisturbed and on a remoulded specimen. The sensitivity is the ratio between the fall cone shear strength of the undisturbed and of the remoulded specimen.

2.1.4 Unconfined compression test

General

The unconfined compression test is used to establish a quick measure of the undrained undisturbed shear strength of the soil. The method is described in detail in NS 8000 and NS 8010.

Procedure

A specimen of soil material, shaped as a cylinder or a prism, is axially loaded in an upright position until fracture occurs in the sample. Axial strain in the sample with a certain compression is considered as the relationship between axial force and mean cross sectional area at this compression. Shear stress in a plane inclined 45° compared to the horizontal is equal to axial stress divided by 2. Undrained shear strength is indicated as the maximum value of the shear stress on a plane inclined 45°.

2.1.5 Density of soil (bulk density)

General

Density of soil is mass of soil per unit volume of the material, including any water or gas it contains. Density of soil is measured by the linear measurement method and the immersion in fluid method. The methods are described in detail in NS-EN ISO 17892 Part 2.

Procedure

The linear measurement method is suitable for the determination of the bulk density of a specimen of soil of regular shape, including specimens prepared for other tests. The specimens used are cylinders with circular cross sections. The principle of the method is to weigh a specimen of known volume.

The immersion in fluid method covers the determination of the bulk density of a specimen of natural or compacted soil by measuring its mass in air and its apparent mass when suspended in fluid. The method may be used when lumps of material of suitable size can be obtained.

2.1.6 Grain size distribution

General

Determination of the grain size distribution of sand and coarser material is performed according to NS 8005. Samples containing mainly silt or clay are subject to the falling drop method (Moum, 1965) or the hydrometer method, as described below.

Procedures

The grain size distribution is determined by the following three procedures:

I. Samples that contain mainly sand and coarser material are subjected to an ordinary dry sieve analysis. Materials containing more than 5-10% silt and clay particles are wet sieved on a 63 µm sieve before dry sieving. The method is described in NS 8005.

II. Hydrometer is a grain size distribution test for fine particles < 63µm. Hydrometer is an indirect analysis of grain distribution by measuring the density of a slurry sample (suspension), based on particles of different size have different sinking time in suspension (Stokes law).

III. For samples containing mainly silt or clay, the falling drop method is used (Moum, 1965). The falling drop method is a sedimentation method based upon Stoke's Law. A small sample of moist material is suspended in water, washed through a 75 µm sieve before being poured into a sedimentation tube. Droplets from a certain depth in the sedimentation tube are sampled with a calibrated micropipette after certain time intervals and then ejected into a glass column containing an organic liquid. The time required for each droplet to fall a certain distance in the organic liquid is measured. The concentration of suspended particles in each droplet can then be read from a calibration chart.

It is recommended to use both method II and III in order to check if the methods give similar results. It is necessary to always state which method has been used in the report.

2.1.7 Organic content

General

Organic content using the loss of ignition method is described in SVV HB R210 "Laboratorieundersøkelser" chapter 2.18 (2016).

Procedure

A 150-g specimen is placed in an oven with temperature 110° C and dried until it reaches constant mass. The specimen is pulverised and sieved, using a 500 µm sieve. Ca 20 g of the sieved material is placed on a porcelain dish and dried in the oven for two hours with the same temperature as earlier. After two hours the specimen is cooled to room temperature in an exsiccator. After cooling, exactly 10 g of the material is placed on a quartz dish in a furnace with constant temperature of 450° C for 24 hours. The specimen is then placed in an exsiccator and cooled to room temperature. After cooling the specimen is weighed and the loss of mass is determined.

The mass loss equals the organic content and is expressed as a percentage of the oven dried mass.

2.1.8 Density of solid particles

This procedure describes the pycnometer method by fluid displacement for determination of density of solid particles in samples comprising grain size smaller than 4 mm. The method is described in detail in NS-EN ISO 17892 Part 3.

The fluid pycnometer method is based on the determination of the difference in the volume of liquid required to fill the pycnometer with the sample being present. The density of solid particles is calculated from the dry mass of the soil particles, ρ_s, divided by their volume difference.

2.1.9 Salinity

General

Conductivity is measured direct from water. The method for determination of conductivity in water is described in detail in NS-ISO 7888.

Procedure

First, the pore water is squeezed out from the sample, either by compressed air or by centrifugation. NTNU recommends filtration after extraction with syringe filter 45 µm. The conductivity in the water is determined by inserting an electrode in the water sample so that the water surface is above the measuring points on the electrode. The highest sensitivity is automatically chosen (mS/cm or µS/cm), and then temperature and conductivity is found. The conductivity meter is calibrated with solutions with known salt content, according to the manual.

The procedure differs from the standard by the use of thermostatic bath at 25 ± 0.1°C. This is considered not to affect the results because of the temperature correction in the instrument.

2.1.10 Thermal conductivity

General

The rate at which heat flows through a material is a measure of its thermal conductivity, λ. It is a property of materials that expresses the heat flux f (W/m^2) that will flow through the material if a certain temperature gradient DT (K/m) exists over the material. The method for determination of thermal conductivity is described in detail in ASTM D5334-92.

Procedure

Thermal conductivity is measured from a sample tube with a radius of 40 mm (> 100 mm is recommended) with the use of the probe TP02. TP02 Non-Steady-State Probe consists of a needle with 2 thermocouple junctions (one of which acts as a reference) and a heating wire, it is inserted into the material that is investigated. In the base, a temperature sensor is mounted.

The probe must be fastened in a way such that the probe is inserted into the centre of the soil to the correct depth and is fixed without vibrations during the experiment. Run-time is in increments of 100 s at sustained effect in the steps high, medium and low. The test should be performed with short increments and low effect, according to the conductivity of the sample. It is important that the test material is in thermal balance before the test is started.

2.1.11 Heat capacity

General

Specific heat is a basic thermodynamic property of all substances. The value of specific heat depends upon chemical or mineralogical composition and temperature. Specific heat is an essential property of rock and soil when these materials are used under conditions of unsteady or transient heat flow. The method for determination of heat capacity is described in detail in ASTM D4611-16.

Procedure

This test method covers the determination of instantaneous and mean values of the specific heat of rock and soil. The testing procedure provides a specific heat over temperatures ranging from 25 to 300°C with the use of a calorimeter.

The test method employs the classical method of mixtures. The method of mixtures consists essentially of adding a known mass of material at a known temperature to a known mass of calorimetric fluid at a known lower temperature and determining the equilibrium temperature. The heat absorbed by the fluid and containing vessel can be calculated from calibrations, and this value equated to the expression for the heat given up by the hot material. From this equation, the unknown specific heat can be calculated.

2.2 Maximum and minimum density of sand

This is an in-house NGI procedure to determine max and min unit density (NGI Report, 2015). It is recommended however, to do several methods, as they show different results.

2.2.1 Minimum dry density

The minimum dry density, ρ_{dmin}, is determined by filling a mould with dry sand. The volume of the mould is about 200 cm^3 and the internal diameter 72 mm. The sand is first placed into a tube mounted centrally in the mould. The tube is then raised slowly so that the sand flows into the mould and fills it completely. The soil which piled above the top of the mould is then removed very slowly, using a straightedge, before weighing the material. The procedure is repeated five times, and the average value is computed.

2.2.2 Maximum dry density

The maximum dry density is determined by both a dry and a wet method. The dry or the wet sand (with the predetermined water content) is placed into the mould in thin layers each consisting of 20 to 25 g soil. Each layer is vibrated vertically for 1/2 minute. During all vibrations the surcharge is 4.2 kN/m^2. During the vertical vibrations the vibrator is applied on top of the surcharge. For the wet method in particular, for each layer a falling weight of 2.5 kg is raised 12" and dropped 5 times after the vertical vibrations. The material is then weighed.

It is recommended to also measure minimum and maximum density according to DIN method (DIN, 1996).

2.3 Direct simple shear test ASTM 6528

2.3.1 General

The Direct Simple Shear apparatus enables the determination of soil anisotropy in an efficient way. A test can be performed drained (constant vertical load) or undrained (constant volume).

A cylindrical specimen with cross-sectional area of 20, 35, 50 and 104 cm^2, and height of 16 mm is placed within a reinforced rubber membrane which prevents radial deformation but allows the specimen to be deformed in simple shear. The apparatus for this test is described by Bjerrum and Landva (1966) and Andresen et al. (1979).

2.3.2 Procedure

Static tests

Mounting, undisturbed material
Trimming and mounting of specimens are done using a kutting knife to achieve a perfect diameter. Due to the possible existence of a negative pore pressure in the clay specimens, special care is taken to avoid swelling due to the presence of free water. This is achieved by using dry porous stones at the start of the consolidation. When water is added, the height difference should be 0. Dry porous stones should be used in order to prevent detrimental swelling of the specimen initially.

Mounting, reconstituted sand
Recompacted sand specimens are built in using a slightly modified version of the method of undercompaction described by Ladd (1978). First the dry soil is mixed with water, to attain a typical water content around 5%.

For DSS test specimens the soil is tamped into the mould in one layer. For relative densities below 90%, compaction is normally done by hand tamping only. For relative densities above 90%, both hand tamping and vertical vibrations may be required in order to achieve the specified densities.

Alternative methods of reconstituting sand specimens may be used based on an ongoing R&D project at NGI (20160122).

Consolidation
A consolidated, constant volume test (DSS-CCV) is the most common static test performed. The axial (vertical) stress is increased in steps to the estimated effective consolidation pressure. At approximately 50 % of the estimated consolidation stress the porous stones are saturated with water of approximately the same salt concentration as the pore water of the clay. Undisturbed clay specimens are consolidated to what was believed to be a low estimate of the preconsolidation stress, p_c' (0.8* p_c'), and then unloaded to the estimated in situ effective vertical stress, p_0', before shearing. This method is used to ensure complete contact between the specimen and the reinforced membrane and thus to develop representative lateral stresses on the specimen, and to some degree counteract the negative effect of stress release and other disturbance effects during sampling and extrusion. It is believed that the specimen thereby experiences correct radial stresses before start of shearing.

Shearing
After consolidation the specimen is sheared by applying a horizontal shear stress. Keeping the volume constant simulates undrained conditions. The volume is kept constant by increasing or decreasing the axial stress during shearing. Normally a strain rate of 5% shear strain per hour is used. The change in the axial stress for a constant volume test is equal to the change in pore pressure for an undrained test where the total axial stress is kept constant. The reason for performing a constant volume instead of an undrained test is that drainage cannot be completely prevented and the saturation by means of back pressure cannot be performed in the simple shear device.

Reporting
The results are presented in three diagrams, horizontal shear stress against shear strain, pore pressure against shear strain, and effective axial stress against horizontal shear stress. In addition the water content, unit weight and volume change during consolidation are reported.

2.4 Triaxial testing (CAUc and CAUe)

2.4.1 General

The triaxial test is carried out in order to measure the stress-strain behavior and the shear strength parameters of a soil specimen under controlled stress conditions. The measurements during consolidation phase can also be used to evaluate sample quality, $\Delta e/e_0$. The triaxial equipment used at NGI is described by Berre (1982).

A cylindrical soil specimen is enclosed in a rubber membrane and placed inside a pressure chamber called a triaxial cell (Figure 1). An isotropic stress is applied to the specimen through the chamber pressure. In addition to the chamber pressure, the specimen is loaded axially by a piston passing through the top of the cell.

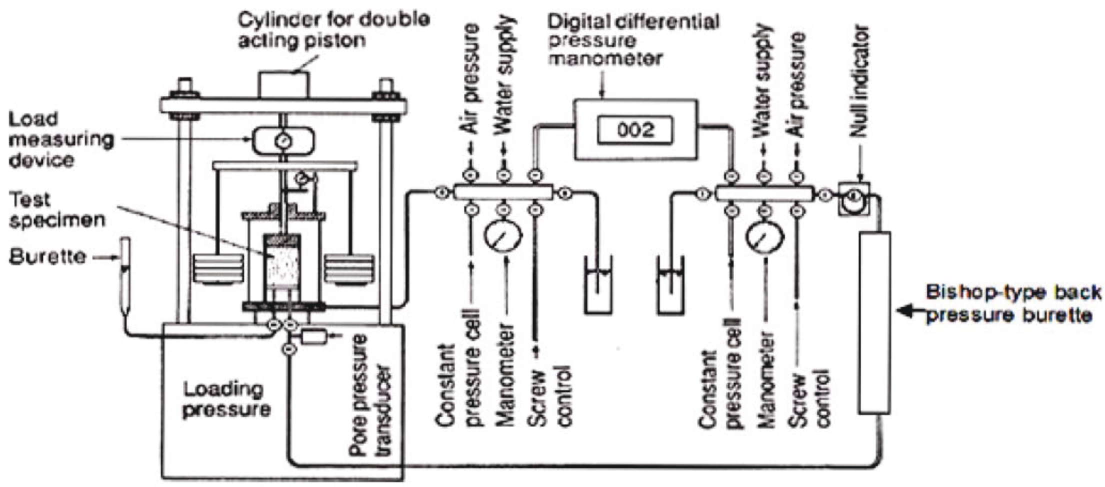


Figure 1 Sketch of triaxial test set-up

2.4.2 Procedures

Static tests

Mounting, undisturbed material
Trimming and mounting of triaxial specimens are done using a kutting knife to achieve a perfect diameter. Due to the possible existence of a negative pore pressure in the clay specimens, special care is taken to avoid swelling due to the presence of free water. This is achieved by using dry porous stones at the start of the consolidation.

About all clays swell when they have access to free water. This applies to NC-, OC-, shallow-, deep-, fresh- and old samples etc. The swell pressure might be low (e.g. 10

kPa) or high (> 100 kPa), dependent on the sample. Swelling damage the sample structure and should be avoided by using dry stones and add water after isotropic swelling pressure is applied.

Except for very soft clays, filter strips can be used. They are placed on the side of clay specimens to speed up drainage during consolidation and pore pressure equalization during undrained shearing. The filter paper strips are moist, but free water on the surface of the strips is wiped away before they are placed on the specimen. The strips are placed in spirals around the specimens to avoid filter paper correction on measured stresses.

Membranes should be stored in water for 24 hours, wiped dry on the inside and checked for any damage or leaks before use. For installing the membrane, a special mounting apparatus is applied.

When the specimen, membrane and filters etc. are mounted on the pedestal and the cell filled with water, a small cell pressure is applied (5 – 10 kPa) in order to prevent the flushing water from penetrating between the membrane and the specimen, and to prevent swelling. Flushing of the pore pressure system is then completed. Any vertical deformation is monitored during this process and remedial action may be applied if any anomalies are observed. Remedial action being adjusted the swelling pressure as needed to prevent specimen swelling. Thereafter consolidation is started. For details see Statens Vegvesen, R210 (2016). A comparison of consolidation procedures is also presented in Appendix A.

Mounting, reconstituted sand

Recompacted sand samples are built in using a slightly modified version of the method of undercompaction described by Ladd (1978). First the dry soil is mixed with water, to attain a typical water content around 5%.

For triaxial test specimens the soil is tamped into the mould in six layers. The initial layers are compacted to lower densities than succeeding layers so that the final density of each specimen layer is approximately uniform. For relative densities below 90%, compaction is normally done by hand tamping only. For relative densities above 90%, both hand tamping and vertical vibrations may be required in order to achieve the specified densities.

Alternative methods of reconstituting sand specimens may be used based on an ongoing R&D project at NGI (20160122).

After mounting, saturation of stones is done using flushing with CO_2.

Saturation

Clay specimens are first subjected to an isotropic stress equal to the estimated value of the initial negative pore pressure. Estimated initial negative pore pressure is isotropic cell pressure applied during flushing/saturation of the stones and the tube when the sample is at rest, neither swell nor consolidate. Water, with approximately the same salt concentration as the pore water of the soil is then flushed through the porous stones. Any sign of volume change or deformation of the specimen is prevented by regulating the cell pressure until stable conditions have been reached.

A back pressure is applied to increase the degree of saturation of the specimen and of the pore pressure measuring system, after the swell-pressure is applied. The backpressure is applied with open drainage.

For specimens assumed to be fully saturated in the field, an attempt is made to reach a B-value of at least 0.95 for static tests. For undrained tests on very soft clays, a back pressure of about 200 kPa may be sufficient. For very stiff clays and dense sand, back pressures up to 1500 kPa may be required.

Having achieved a suitable backpressure, a B-test is performed by closing the drainage system and increasing the cell pressure by $\Delta\sigma$, around 10 kPa for normally consolidated samples and up to 100 kPa for over consolidated samples. The B-test is usually acceptable if criteria is met within 3 minutes. *Report values after 1 min.*

Consolidation

The specimen is loaded to the specified consolidation stresses in two steps by anisotropic consolidation. The consolidation may be applied by two methods, either by

- First applying an isotropic pressure σ_3' and then obtaining the required anisotropic pressure σ_1' by adjusting the axial pressure. The same ramping rates are applied as given above. The axial pressure is applied immediately after σ_3' is obtained.
- Applying both the cell pressure (effective pressure σ_3') and axial pressure (effective pressure σ_1') simultaneously, with a slower rate for the axial load. Rate of pressure loading σ_3' is from 0.4 to 2.0 kPa/min and σ_1' from 0.1 to 0.5 kPa/min.

The first procedure is preferred due to experience from comparable tests conducted (NGI).

The consolidation stage is stopped when the change of volume of expelled pore water is of an insignificant magnitude and the slope of the volume/time curve reach a specimen volume change rate of less than 1 mm^3/3 min. This is a stability check, which verifies adequate saturation, no leaks in the membrane, hoses, fittings, etc. and that shearing can start.

Shearing

Static, anisotropic consolidated, undrained tests (CAU).

The specimen is sheared at a constant rate of axial strain. The total radial stress is kept constant while the total axial stress is increased in compression tests and decreased in extension tests. During the undrained anisotropically consolidated tests carried out on clay material (CAU), no drainage is allowed during shearing so that excess pore pressures may develop. The rate of strain is 1.4%/hour. 1.4%/h is used for the sake of pore pressure equalization, time to failure and time to intercept (skjæring). Other rates of strain may be used if effect of changes in rate is investigated. This must be noted in all test results.

Static, anisotropic consolidated, drained tests (CAD).

The specimen is sheared at a constant rate of axial strain. The total radial stress is kept constant while the total axial stress is increased in compression tests and decreased in extension tests. Drainage is allowed during shearing so no excess pore pressures may develop. Instead, the volumetric strain is measured and recorded. The rate of strain will be chosen low enough to avoid development of pore water pressure.

The test may be stopped when the axial strain reach a minimum of 15% in both CAU and CAD tests.

Dismounting specimen

After the cell pressure has been reduced to zero, the specimen is carefully removed from the triaxial cell and weighed. Material for classification testing may then be taken out.

Reporting

The results are presented in three diagrams, max shear stress against axial strain, pore pressure development against axial strain and stress path curve (NTNU-plot or MIT/NGI-plot). In addition the water content, density, classification, volume strain and/or pore number change and any correction model are to be presented with the results. Two corrections are included; correction for circular area change and correction for the effect of rubber membrane. Values used for E-modulus and thickness of the rubber membrane are examined before each test. A new rubber membrane is used for each test. Correction values for a "standard membrane" are used. These values change mathematically according to a simple index (extension) test at each membrane used. This mathematic change compensates different E-modulus and/or dimension/thickness in the actual membrane. This is approximately the same as measuring the actual E-modulus, diameter and thickness in every membrane. The membrane correction is applied on both vertical and radial stress during the whole test.

The latex E-modulus used is often 1400 kPa. NGI has found a typical E-modulus close to 1340 kPa.

For clay specimens sample quality shall be commented by reporting change in void ratio during consolidation of specimen to best estimate of in situ stresses.

2.5 Consolidation tests

2.5.1 General

Oedometer tests on undisturbed material are performed to determine the preconsolidation stress of the material as well as the compressibility and permeability. The results can also be used for evaluation of sample quality. A detailed description of equipment and procedures used at NGI is given by Sandbækken et al., 1986.

2.5.2 Procedures

Continuous loading (CRS)

This is the preferred method at NGI, with reference to NS 8018. In the continuous loading test the specimen is compressed with a constant rate. A vertical effective stress is applied in one step by using deadload. The continuous loading is then started and readings of the axial stress and strain are taken at certain time intervals. The specimen is allowed to drain freely at the top, but is undrained at the bottom. The rate of strain is selected such that the pore pressure at the bottom preferably shall be less than about 10% of the total applied stress. Higher pore pressure can be accepted during short parts of the test, but should never exceed 20% of the total applied stress.

Incremental loading (IL)

In the incremental loading test, the vertical load on the specimen is applied in increments. The duration of each load increment should be long enough to ensure 100 % consolidation, as described in NS 8017.

Mounting
The trimming and mounting of CRS specimen is done using a cutting cylinder or ring achieve a perfect diameter. The cylindrical specimen has a cross-sectional area of 10 cm^2, 20 cm^2 or 35 cm^2 and height 20 mm, and is placed within a steel ring which prevents radial deformation of the specimen. Due to the possible existence of a negative pore pressure in the clay specimens, special care is taken to avoid swelling due to the presence of free water. This is achieved by using dry porous stones at the start of the consolidation.

An axial (vertical) stress is applied on the top of the specimen and the resulting vertical compression of the specimen is measured.

Water of correct salinity, is added to the filter stones when a vertical stress high enough to prevent swelling has been applied. An unloading - reloading loop is often done when the vertical stress has become equal to about twice the estimated preconsolidation stress (see below). The purpose with this loop is to be able to correct for sample disturbance and to obtain reloading parameters. A second unloading – reloading loop can be done at the end of the test in order to determine reloading parameters at high stress levels.

The test is designed to determine the compressibility (or constrained modulus) and the coefficient of consolidation of the specimen tested. The coefficient of consolidation, c_v, is calculated from the equation

$$c_v = \frac{M \cdot k}{\gamma_w}$$

where M is the constrained modulus and k the coefficient of permeability. The k-values are checked by constant head permeability tests if there is any doubt about k-values back-calculated from time-compression curves (incremental loading) or from measured pore pressures (continuous loading). The uncertainty by deriving c_v from time-compression curves or measured pore pressures alone is then avoided and the c_v - value can be based on M-values corrected for sample disturbance and on k-values of in situ porosity and temperature.

For undisturbed specimens the test is also used to determine the effective vertical stress where the compressibility of the specimen starts to increase. This stress is represented by a more or less sharp bend in the stress-strain curve and is called the pre-consolidation stress (denoted p_c').

Reporting
The strain rate to be used for a test series should be determined based on a trial to see pore pressure development. The results are presented in diagrams (linear scale) where strain (ε), stress modul (M) and consolidation coefficient (c_v) are plotted against axial effective stress (σ_v'). the change in void ratio at a vertical stress equal to best estimate shall be reported for evaluation of sample quality.

2.6 Permeability test

Permeability in the triaxial tests and CRS tests is measured using the constant-head method. The permeability is measured after consolidation in the triaxial tests and after shear. The latter permeability test is carried out under initial consolidation stress and excess pore pressure dissipated. CRS permeability tests is carried out at a specified point in the test.

The permeability is calculated from Darcy's law as k = q/(A*i) where *q* is the flow rate, *A* is the specimen area, *i* is the hydraulic gradient. The total flow measured during a test is typically of a few cubic centimetres of water and the total time of measurement can vary from minutes to hours.

2.7 G_{max} with bender elements

2.7.1 General

The initial shear modulus (G_{max}) of a soil is an important parameter for a variety of geotechnical design applications and in small strain dynamic analysis. G_{max} may also be used as an indirect indication of various soil parameters, as it in many cases correlates well to other soil properties such as density, fabric and liquefaction potential as well as sample disturbance by comparing laboratory and field measurements. This modulus is normally associated with shear strain levels of about 0.001 % and below. The purpose of the piezoceramic bender element test is to provide the maximum shear modulus at small strains, G_{max}, by measuring the shear wave velocity in the test specimen.

The technique makes use of piezoceramic bender elements at each end of a soil specimen. The bender element is an electro-mechanical transducer which is capable of converting mechanical energy (movement) either to or from electrical energy. The bender element at one end of the specimen is thus used to generate a sinusoidal shear wave pulse, which propagates along the length of the specimen and the other element is used to determine the arrival time of the shear wave at the other end of the specimen. Reference is made to Dyvik and Madshus (1985) and Dyvik and Olsen (1989) for more details. The travel time and length travelled give the shear wave velocity. The shear modulus at small strains, G_{max}, can then be computed from the formula:

$$G_{max} = v^2 \cdot \rho$$

where: v = the shear wave velocity
ρ = density of the soil

At NGI measurement of G_{max} using bender element technique can be done at any stage of a triaxial, direct simple shear or oedometer test without interfering with the particular test.

For the bender element tests performed in connection with static DSS tests, two determinations are made for two of the tests, one at the maximum vertical stress and one at the final consolidation stress, σ'_{min}. One determination is made for the DSS test loaded directly to the consolidation stress before shearing.

Preparation and mounting
The bender element is a high impedance device and cannot be exposed to moisture as this will electrically short the transducer. The device is therefore cased in a vacuum treated, heat cured two-component epoxy. Great care must be taken to avoid open seams, cracks or air bubbles.

The cased bender element is then placed in a slot in the pedestal or top cap in the soil testing device. The bender element protrudes into the specimen as a cantilever and the surrounding soil particles move in the same back and forth movement as the tip of the element. This will result in shear waves which propagate through the specimen in a direction parallel to the length of the relaxed element.

2.8 Specimen preparation

There are different methods for trimming and preparation of a specimen for advanced testing. The methods are listed below:

1. Trimmed by wire saw
2. Trimmed with a cutting knife or a cutting ring
3. Trimmed with band saw
4. Trimmed by lathe
5. Core boring
6. Reconstituted/remoulded

If the soil is very soft (undrained shear strength less than approx. 7-9kPa) the sample may be extruded from the sample tube and directly into a triaxial membrane, oedometer ring or DSS cutting ring. The purpose is to reduce and minimize sample disturbance because of less critical and/or difficult sample handling operations of soft material.

2.8.1 Trimmed with a wire saw

There are two different wire diameters – a thin wire for material with undrained shear strength below approximately 40kPa and a thicker wire for material with higher strength.

2.8.2 Trimmed with a cutting knife

The trimming and mounting of DSS and CRS samples is done using a cutting knife to achieve a perfect diameter.

2.9 Laboratory testing procedures for frozen soils

2.9.1 Requirements to quality of samples of frozen soils for laboratory testing

Requirements to samples of frozen soils for laboratory testing depend on (i) the stage of structure development (prospect, design or monitoring stage) of and/or (ii) design requirements for the proposed structure or facility.

For some applications (prospect stage), or some types of structures (temporary structures, or structures that can tolerate some movement), samples obtained from auger flights or cuttings may be adequate (Andersland, Ladanyi, 2004). "For soil with high ice content, partly disturbed samples drive samples may be adequate", (Andersland, Ladanyi, 2004).

For applications, where consolidation and creep tests required (high ice content, heavy foundation loads, structures are sensitive to small movements), minimally disturbed samples will be required. Minimally disturbed samples can be obtained from monolith samples, or as core samples. Minimally disturbed samples should, in result of sampling, be unstressed (except that there will be stress relief due to removal of the sample from the natural load conditions after sampling) and undisturbed (with positive temperatures, occurring due to both, friction or influence of flushing media during sampling, and also during transportation and storage). Core samples shall be of sufficient quality to visually classify ground ice, and permit the calculations of water content, dry and bulk density.

Minimally disturbed core samples can be obtained from (i) block samples (Still et al., 2013b), and by using the following coring methods, which demonstrated sampled of appropriate quality: (ii) double tube, swivel-head core barrel of the "M" type (Gilbert et al., 2015; Hvorslev, Goode, 1963), (iii) single tube core barrel (Calmels et al., 2005; Dickinson et al., 1999), (iv) CRREL coring auger (Brockett, Lawson, 1985), (v) SINTEF-modified CRREL coring auger (Wold, Bæverfjord, 2013).

The following factors define the quality of the naturally frozen soil sample, (Still et al., 2013b): type of frozen soil, the in situ thermal condition at the time of sampling, the sampling method, the transportation and storage procedures, and the specimen machining prior to testing.

Routines for sampling for sampling frozen soils are presented in "NGTS report on field operations" [NGTS report 20160154-03-R].

2.9.2 Requirements to transportation, storage, machining and testing of samples of frozen soils

Detailed routines for transportation, preparation, and storage of frozen soil samples for laboratory testing are presented in (Baker, 1976). Some advances in these routines and description of routines in more condensed way are presented in (Still et al., 2013b).

Temporal storage in the field, transportation, and storage of frozen soils

Sublimation, evaporation, and thermal disturbance affect mechanical properties of frozen soils; thermal disturbance embraces both, thawing and temperature fluctuations (Baker, 1976). The degree of care in providing protection against mentioned above detrimental actions depends on the type of laboratory test.

After the samples are extracted from the ground, they shall be wrapped in cellophane, air shall be extruded from the bag, and afterwards the bag shall be sealed to prevent sublimation. Air can be extruded by using vacuum pump or forcing most of the air by hand prior to sealing. Placing some snow or ice in the bag can help to maintain humidity of the sample. Types of sealing: (i) heat sealing, (ii) locking nylon tie, (iii) "plastic zip."

Unfrozen samples that have been obtained for thaw consolidation, must not be allowed to freeze (so that they can be tested in truly undisturbed conditions). Frozen samples obtained for thaw consolidation, strength and creep tests, must be maintained at the in situ temperature, (Baker, 1976).

Temporal storage of packed samples in the field can be organized in the following ways: a portable freezer (i); insulated box packet with ice, dry ice, or freezer "jelly" packs; (ii) in; (iii) storing in the bore holes (made during sampling, for instance); (iv) in an uninsulated box (if ambient air temperatures are negative (a limit should be, probably, defined here, let's say, -2 ^{o}C in day time); (v) burring samples in snow.

Transportation of samples shall be performed in portable freezer or insulated box. [Additional requirements shall be summarized for transportation (Anatoly Sinitsyn)].

Storage and protection during laboratory testing
Samples shall be inspected upon delivery in the storage for ware of cellophane bags due to transportation. Samples in damaged bags shall be repacked. Storage of samples of frozen soils shall be performed at the temperature as close to the in situ temperature as possible. Freezers shall be equipped/connected to automatic system providing alternative power supply (from a generator, for instance) in case of the power cut.

Additional insulation (polyurethane bag, for instance) samples during temporal storage in the field, transportation, and storage are beneficial for reduction of temperature fluctuations.

Machining of samples of frozen soils

Rough and finishing methods are presented in (Baker, 1976). Cylindrical samples are normally used for unconfined compression tests. Standard (D7300-11, 2011) describes requirements for geometrical dimensions of a specimen for the latter test. Fabrication of cylindrical specimens from block samples is described in (Still et al., 2013b).

Testing of frozen soils

Common operations:

- Conditioning of the specimens to a specific temperature.
- Picturing; notes on visual classification according to (ASTM D4083-89, 2016), including location, size, distribution of ice and larger grains of soil; dimensions of a specimen; orientation of a specimen; label on sample (showing sampling site, and other information as bore hole number and depth).
- Notes on calibration (when it was done and results) of load cell and apparatuses measuring strains.
- Notes of failure mode of tested specimens.

2.9.3 Classification of frozen soils

Visual classification of frozen soil can be performed by using standard (ASTM D4083-89, 2016). This classification is independent from the geological history and mode of origin. Classification according to the latter standard does not include index properties as water (ice) content, dry density, and strength properties.

Classification involves three parts:

- Part 1 – the soil phase is identified independently of the frozen state using the Unified Soil Classification System.
- Part 2 – characteristics resulting from the frozen state are described.
- Part 3 – ice strata (found in the soil) are described.

Classification for Part 1 shall be performed according to (NS-EN 1997-2:2007+NA:2008, 2008). Procedures from (ASTM D4083-89, 2016) shall be used for Part 2 and Part 3.

According to (Still et al., 2013a), standard (ASTM D4083-89, 2016) is not adequate alone to classify frozen soils. Thaw settlement estimations and assessment of a need for mechanical testing of the frozen soil will be facilitated if one include in classification of the soil the following index properties: (i) the water content, (ii) dry and (iii) bulk density.

2.9.4 Index properties and chemical test of frozen soils

Index properties of thawed soils

Index properties of frozen soils which are defined on thawed material shall be obtained by using standards listed in sections 2.1.

Specific index properties

Initial freezing temperature can be determined by an approach described in (Andersland, Ladanyi, 2004) and based on the cooling curve. Lomonosov Moscow State University has developed a method based on the thawing curve. Some other specific index properties like *unfrozen water content* can be also determined.

2.9.5 Mechanical testing of frozen soils

Creep properties of frozen soil samples can be determined by using standard (D5520-11, 2011). Strength properties of frozen soil at a constant rate of strained can be determined by using standard (D7300-11, 2011). Methodologies for triaxial testing of frozen soil are presented in (Kornfield, Zubeck, 2013).

Methods for other mechanical tests are briefly reviewed in (Oestgaard, Zubeck, 2013).

2.9.6 Frost heave and thaw weakening properties

Frost heave and thaw weakening susceptibility of soils can be determined by using standard (ASTM D5918-13, 2013). Determination of the effect of freeze-thaw on hydraulic conductivity of compacted or intact soil can be determined by using standard (ASTM D6035/D6035M, 2013).

Methodology for defining the segregation potential was suggested by (Konrad, 1987).

2.9.7 Thermal properties of frozen soils

Thermal conductivity of soils can be determined by using thermal needle probe described in (D5334-14, 2014), heat capacity of soils can be determined by using standard (ASTM D4611-16, 2016).

3 References

Andresen, A., T. Berre, A. Kleven and T. Lunne (1979)
Procedures used to obtain soil parameters for foundation engineering
in the North Sea. Marine Geotechnology, Vol. 3, No. 3, pp. 201-266.
Also publ. in: Norwegian Geotechnical Institute,
Publication, 129.

Aksenov, V.I., (2008)
Zasolennyye merzlyye grunty arkticheskikh poberezhiy kak osnovaniya
sooruzheniy *(Saline frozen soils of Arctic coast as bases for structures)*
Vse o mire stroitel'stva (Everything about construction world), Moscow, 340.

Andersland, O.B. and Ladanyi, B., (2004)
Frozen Ground Engineering, 2nd Edition John Wiley & Sons,
Hoboken, New Jersey.

ASTM D4083-89, 2016. Standard Practice for Description of Frozen Soils (Visual Manual Procedure).

ASTM D4611-16, Standard Test Method for Specific Heat of Rock and Soil, 2016.

ASTM D5334-00 Standard Test Method for Determination of Thermal Conductivity of Soil and Soft Rock.

ASTM D5918-13, Standard Test Methods for Frost Heave and Thaw Weakening Susceptibility of Soils, 2013.

ASTM D6035/D6035M, Standard Test Method for Determining the Effect of Freeze Thaw on Hydraulic Conductivity of Compacted or Intact Soil Specimens Using a Flexible Wall Permeameter, 2013. West Conshohocken, PA

Baker, T.H.W. (1976)
Transportation, Preparation, and Storage of Frozen Soil Samples for Laboratory Testing. Soil Specimen Preparation for Laboratory Testing, ASTM STP599, American Society for Testing and Materials, Montreal, Canada, 22-27 June 1975, 88-112.

Berre, T. (1982)
Triaxial testing at the Norwegian Geotechnical Institute.
Geotechnical Testing Journal, Vol. 5, No. 1/2 pp. 3-17.
Also publ. in: Norwegian Geotechnical Institute.
Publication, 134, 1981, pp. 7-23

Bjerrum, L. and A. Landva (1966)
Direct simple shear tests on a Norwegian quick clay
Geotechnique, Vol. 16, No. 1, pp. 1-20
Also publ. in: Norwegian Geotechnical Institute, Publication No. 70.

Brockett, B.E. and Lawson, D.E. (1985)
Prototype drill for core sampling fine-grained perennially frozen ground CRREL, pp.

Calmels, F., Gardon, O. and Allard, M. (2005)
A portable earth-drill system for permafrost studies Permafrost and Periglac. Process. 16, 311-315.

D5334-14, A., Standard Test Method for Determination of Thermal Conductivity of Soil and Soft Rock by Thermal Needle Probe Procedure, 2014.

D5520-11, A., Standard Test Method for Laboratory Determination of Creep Properties of Frozen Soil Samples by Uniaxial Compression, 2011.

D7300-11, A., Standard Test Method for Laboratory Determination of Strength Properties of Frozen Soil at a Constant Rate of Strain, 2011.

Dickinson, W., Cooper, P., Webster, B. and Ashby, J. (1999)
A portable drilling rig for coring permafrsoted sediments Journal of Sedimentary Research 69(2), 518-527.

DIN (1996)
Bestimmung der Dichte nichtbindiger Böden bei lockerster und dichtester Lagerung.
DIN 18126: 1996-11 Deutsches Institut für Normung, Berlin, Germany.

Dyvik, R. and C. Madshus (1985)
Lab Measurements of G_{max} Using Bender Elements.
ASCE Annual Convention, Advances in the Art of Testing Soils Under Cyclic Conditions.

Dyvik, R. and T.S. Olsen (1989)
G_{max} measured in oedometer and DSS tests using bender elements.
International Conference on Soil Mechanics and Foundation Engineering, 12. Rio de Janeiro 1989. Proceedings, Vol. 1, pp. 39-42.
Also publ. in: NGI, Oslo. Publication, 181, 1991.

Geotekniske felt- og laboratorieundersøkelser – Laboratorieprøving av jord – Del 1: Bestemmelse av vanninnhold (ISO 17891- 1:2014)

Gilbert, G.L., Christiansen, H.H. and Neumann, U. (2015)
Coring of unconsolidated permafrost deposits: methodological successes and challenges, GEOQuebec 2015, Challenges from North to South, Des defis du Nord au Sud

Hvorslev, M.J. and Goode, T.B. (1963)
Core drilling in frozen soils. Permafrost International Conference, 11-15 November, Lafayette, Indiana, 364-371.

Konrad, J.-M. (1987)
Procedure for determining the segregation potential of freezing soils Geotech. Test. J. ASTM 10(2), 51-58

Kornfield, T.S. and Zubeck, H. (2013)
Triaxial Testing of Frozen Soils - State of the Art. Mechanical Properties of Frozen Soils, STP 1568, 76-85.Ladd, R.S. (1978)
Preparing Test Specimens Using Undercompaction
Geotechnical Testing Journal, GTJODJ, Vol. 1, No. 1,
March 1978, pp. 16-23.

Moum, J. (1965)
Falling drop used for grain-size analysis of fine grained materials. Sedimentology, Vol. 5, No. 4, pp. 343-347.
Also publ. in: Norwegian Geotechnical Institute. Publication, 70, 1966.

Norges Byggstandardiseringsråd (NBR). (1982) NS 8000
Geotechnical testing – Laboratory methods – Consistency limit, terms and symbols

Norges Byggstandardiseringsråd (NBR). (1982) NS 8001
Geotechnical testing – Laboratory methods – Percussion liquid limit

Norges Byggstandardiseringsråd (NBR). (1982) NS 8002
Geotechnical testing - Laboratory methods - Fall cone liquid limit

Norges Byggstandardiseringsråd (NBR). (1982) NS 8003
Geotechnical testing, Laboratory methods. Plastic limit

Norges Byggstandardiseringsråd (NBR). (1990) NS 8005
Geotechnical testing - Laboratory methods - Grain-size analysis of soil samples

Norges Byggstandardiseringsråd (NBR). (1990) NS 8010
Geotechnical testing - Laboratory methods – Soil constituents and structure. Terms and symbols

Norges Byggstandardiseringsråd (NBR). (1988) NS 8015
Geotechnical testing, Laboratory methods. Determination of undrained shear strength by fall-cone testing.

Norges Byggstandardiseringsråd (NBR). (1991) NS 8017
Geotechnical testing - Laboratory methods – Determination of one – dimensional consolidation properties by oedometer testing. – Method using incremental loading.

Norges Byggstandardiseringsråd (NBR). (1993) NS 8018
Geotechnical testing - Laboratory methods - Determination of one – dimensional consolidation properties by oedometer testing. – Method using continuous loading.

NS-EN 1997-2:2007+NA:2008 (2008)
Eurocode 7: Geotechnical design - Part 2: Ground investigation and testing (Eurokode 7: Geoteknisk prosjektering - Del 2: Regler basert på grunnundersøkelser og laboratorieprøver), 208.

NS-EN ISO 17892-2:2014. Geotekniske felt- og laboratorieundersøkelser – Laboratorieprøving av jord – Del 2: Bestemmelse av romdensitet (ISO 17892-2:2014)

NS-EN ISO 17892-3: 2015. Geotekniske felt- og laboratorieundersøkelser – Laboratorieprøving av jord – Del 3: Bestemmelse av korndensitet (ISO 17892-3:2015, korrigert versjon 2015-12-15)

NS-ISO 7888. Vannundersøkelse – Bestemmelse av konduktivitet (=EN 27888:1993)

Oestgaard, F.E. and Zubeck, H.K. (2013)
Triaxial Testing of Frozen Soils - State of the Art.
Practice of Testing Frozen Soils, 62-75.

Sandbækken, G., T. Berre and S. Lacasse (1986)
Oedometer testing at the Norwegian Geotechnical Institute. Consolidation of soils: testing and evaluation: a symposium.
ASTM, Special Technical Publication, 892, pp. 329-353.

Statens Vegvesen (Norwegian Public Roads Administration) 2016
Handbook R210, Handbook 014, Test procedure 14.445 "Humusinnhold ved gløding"

Statens Vegvesen (Norwegian Public Roads Administration) 2016
Handbook R210, Handbook 014, Test procedure 14.481 "Treaksialforsøk"

Still, B., Proskin, S., Zubeck, H. and Yang, Z. (2013a)
Frozen-Soil Classification With Index Testing. Mechanical Properties of Frozen Soils, STP 1568.

Still, B., Yang, Z.J. and Ge, X. (2013b)
Sampling, Machining, and Testing of Naturally Frozen Soils. Mechanical Properties of Frozen Soils, STP 1568, 49-61.

Wold, M. and Bæverfjord, M.G. (2013)
Testing of different methods for sampling coarse soils in warm coastal permafrost. Proceedings of the 22nd International Conference on Port and Ocean Engineering under Arctic Conditions June 9-13, 2013, Espoo, Finland.

Appendix A

Comparison of triaxial consolidation procedures

Pasquale Carotenuto
2018-02-12
20160154 – Triaxial consolidation procedures

- Comparison between CAUC tests:
    - ONSB41-11-A-1, Consolidated according to **NGI methodology**, i.e. isotropic stress increase to final radial effective stress followed by increase in deviatoric stress up to final axial effective stress
    - ONSB41-11-B-1, Consolidated according to **SVV methodology**, i.e. simultaneous increase in axial and radial effective stress at constant ratio
    - ONSB41-11-C-1, Consolidated according to **NGI methodology**, i.e. isotropic stress increase to final radial effective stress followed by increase in deviatoric stress up to final axial effective stress
- Below is the sample tube subdivision

BH -Tube - TEST ID	Depth			Assumed p0'	
	10.00				
	10.05				
	10.10	Kast			gamma,eff = -9+7.3*z
	10.15				
ONSB41-11-A-1	10.20	CAUC	------------ ---------->	65.3	
	10.25				
	10.30				
ONSB41-11-B-1	10.35	CAUC	------------ ---------->	66.4	
	10.40				
	10.45				
ONSB41-11-C-1	10.50	CAUC	------------ ---------->	67.5	
	10.55				
	10.60				
	10.65	Fc+Fcrem			
	10.70	wp/wl			
	10.75	gamma-s			
	10.80	GSD			

- Result summary table – see table nr 1
- Results plots:
    - Page 2 correlations
    - Page 3-4 shearing phase all tests
    - Page 5 consolidation phase all tests
    - App. Pdf single tests

- Observations on the results
    - From figure 1
        - As the sample depth increases, the initial specimen water content decreases
        - As the initial specimen water content decreases, the volumetric strain at end of consolidation decreases
    - From figure 2
        - As the volumetric strain at end of consolidation decreases, the normalized shear strength increases (linearly)
    - The undrained shear strength or volumetric strain at end of consolidation does not appear to correlate with consolidation method

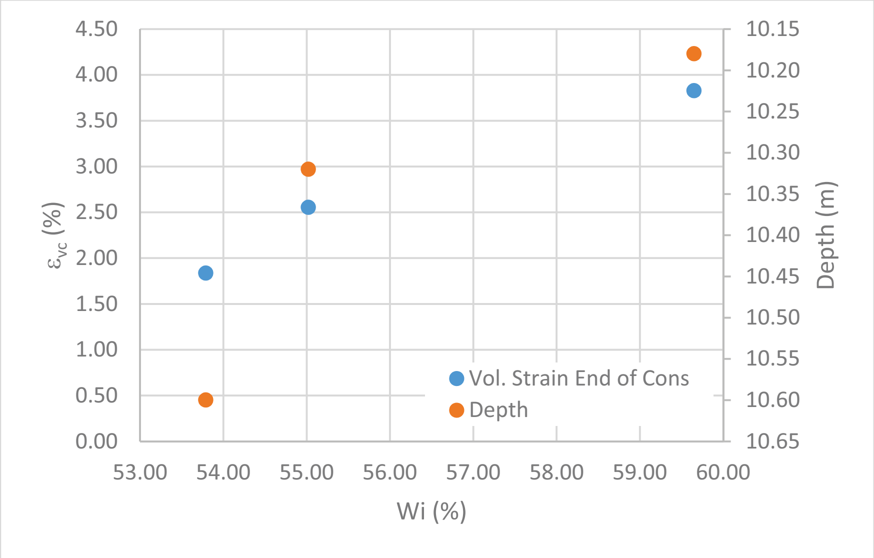


Figure 1 Initial water content versus volumetric strain end of consolidation and specimen depth

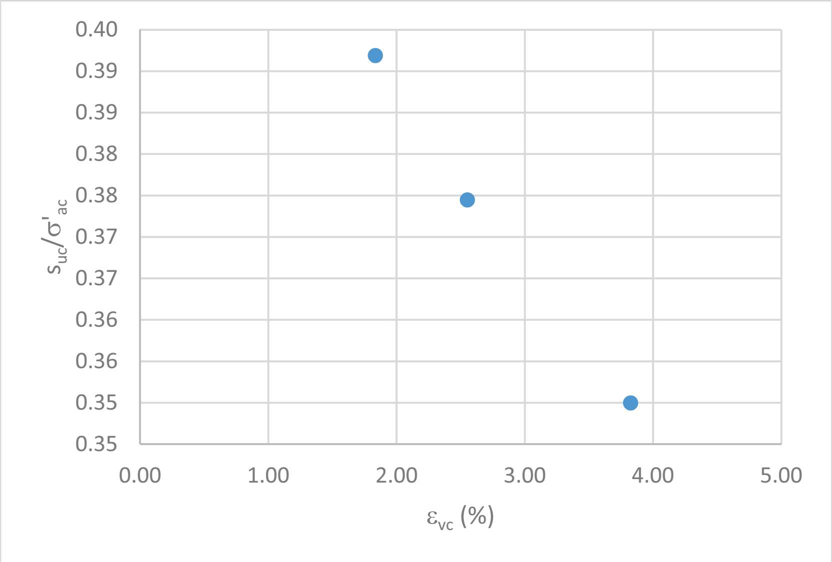
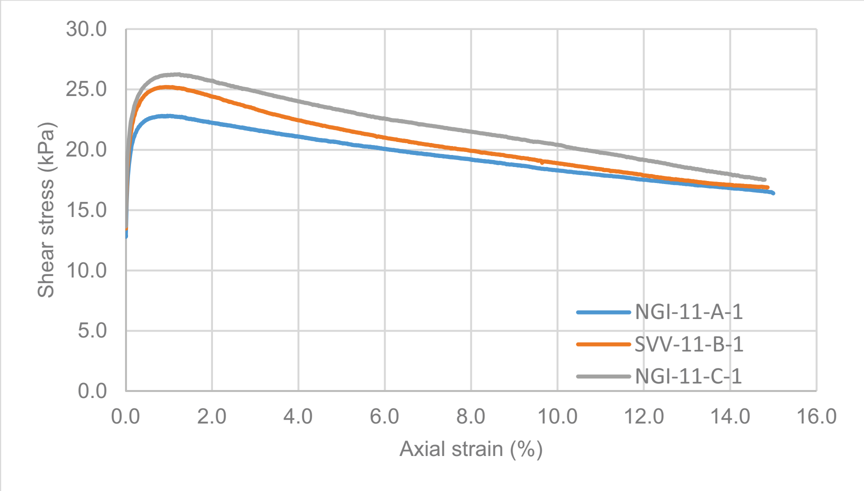
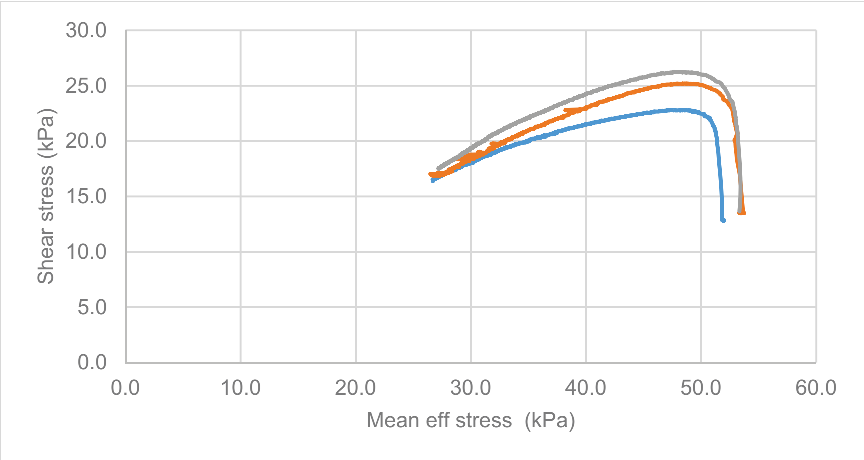
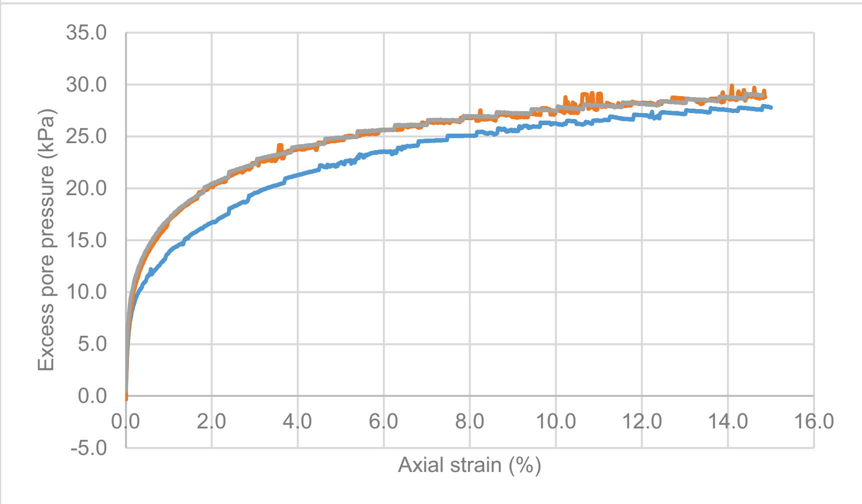


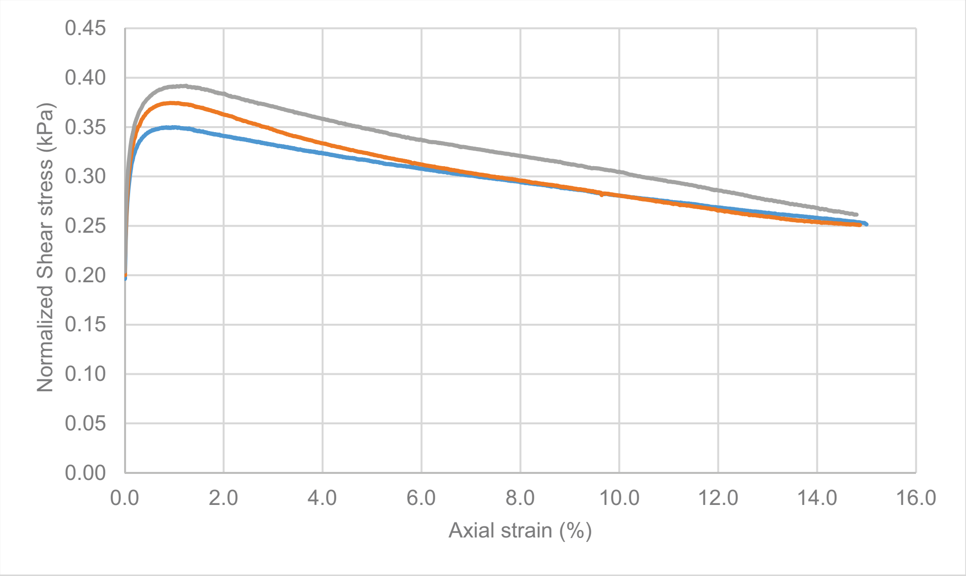
Figure 2 Volumetric strain end of consolidation versus undrained shear strength

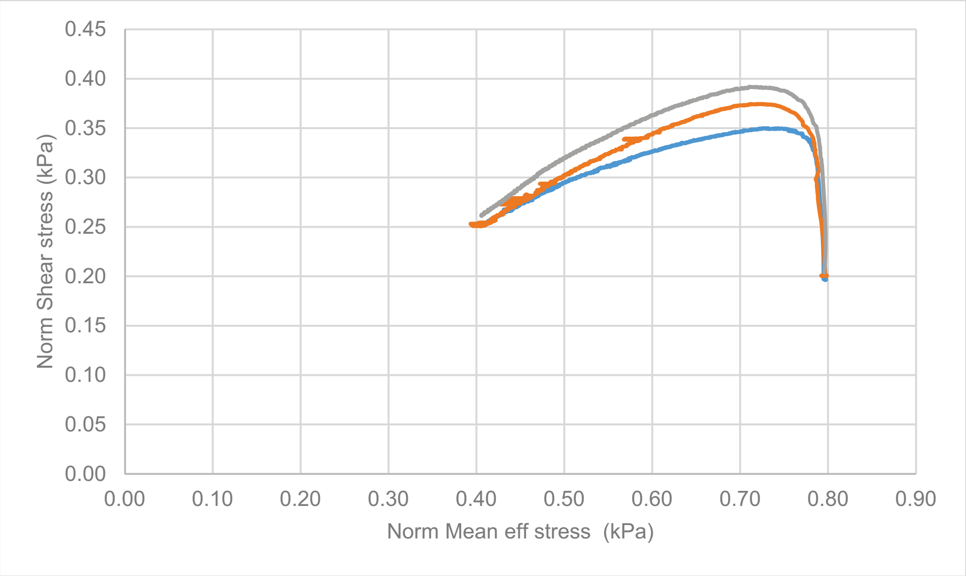
- Results: plots of undrained shear phase to failure








0.45
0.40
0.35
0.30
0.25
0.20
0.15
0.10
0.05
0.00
Normalized Shear stress (kPa)
0.0
2.0
4.0
6.0
8.0
10.0
12.0
14.0
16.0
Axial strain (%)


0.45
0.40
0.35
0.30
0.25
0.20
0.15
0.10
0.05
0.00
Norm Shear stress (kPa)
0.00
0.10
0.20
0.30
0.40
0.50
0.60
0.70
0.80
0.90
Norm Mean eff stress (kPa)

- Results: plots of consolidation phase

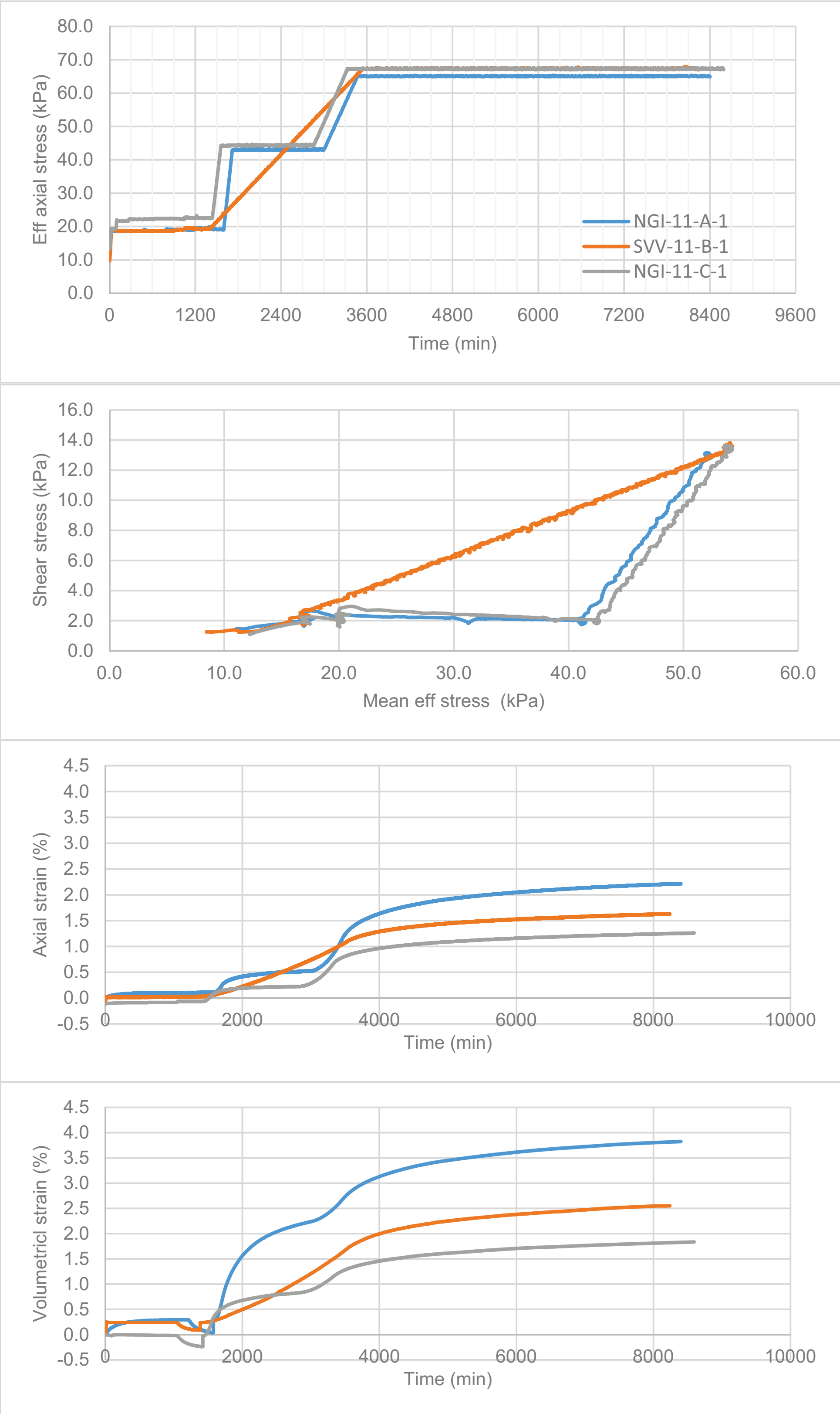
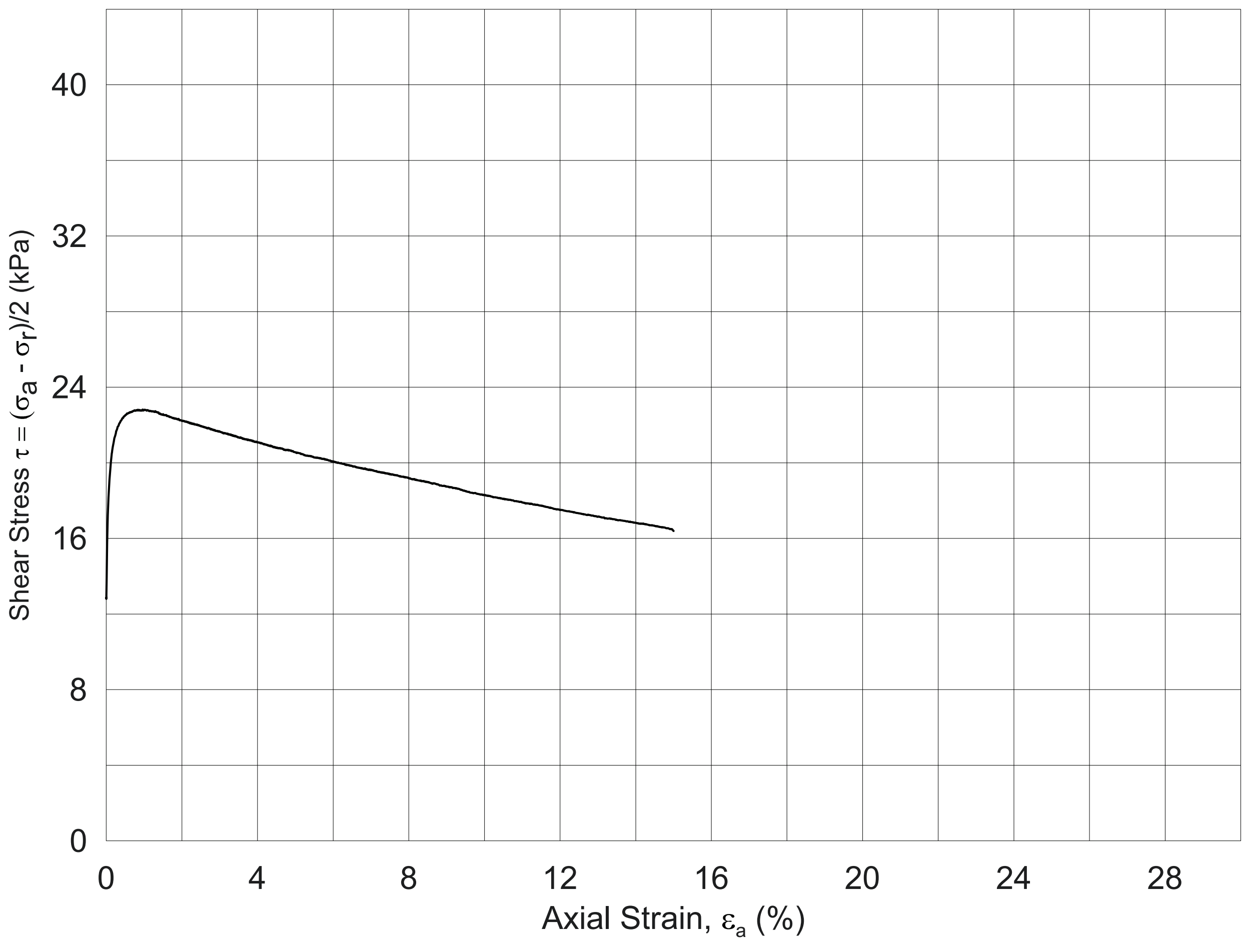
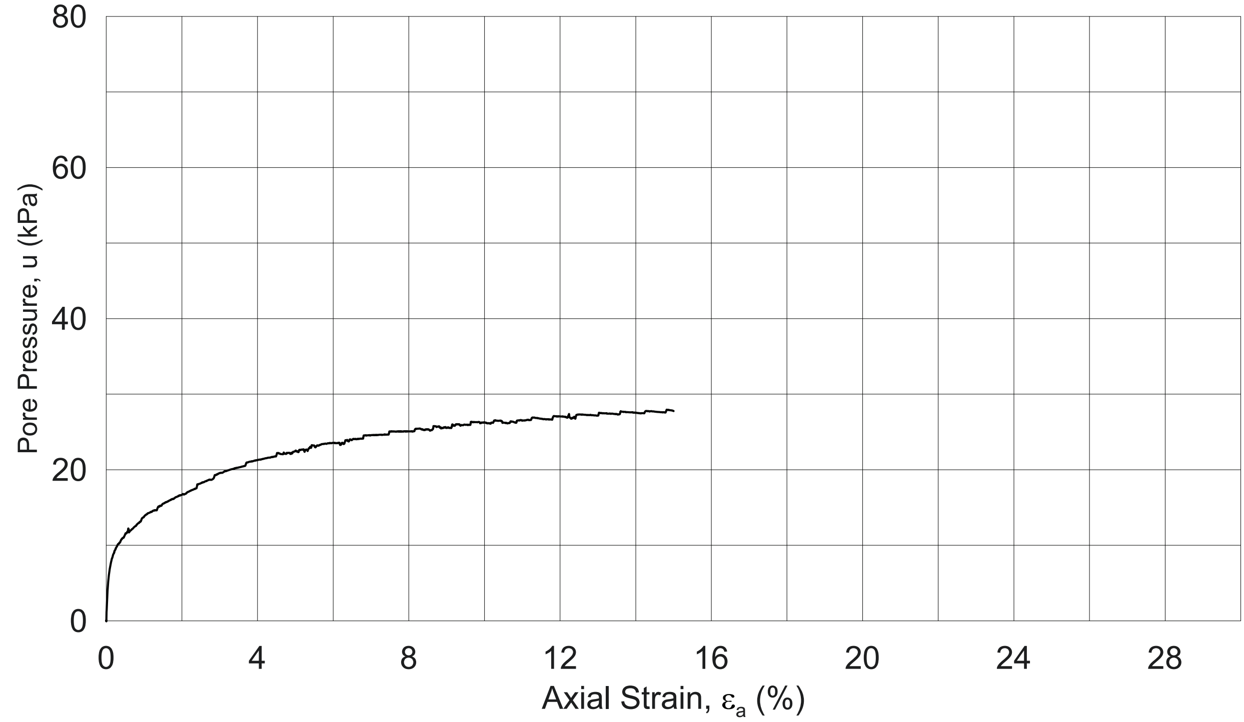


Table 1

Test Name	Cons. type	Depth bgf	Depth cyclinder	estim. p'0	Index-Initial			Index-Consolidation			Consolidation												Shearing						Description
					Wi	γi	Sri	Wc	γc	Src	σ'_{ac}	σ'_{rc}	τ'_c	p'_c	$\sigma'_{rc}/\sigma'_{ac}$	BP	Time	Time	ε'_{ac}	ε'_{vc}	ε'_{rc}	B-value	Av. rate	s_{uc}	s_{uc}/σ'_{ac}	ε_{af}	Δu_f	ϕ_u	
-	-	m	cm	kPa	%	kN/m^3	%	%	kN/m^3	%	kPa	kPa	kPa	kPa		kPa	hrs	days	%	%	%	%	%/hr	kPa	-	%	kPa	deg	-
ONSB41-11-A-1	NGI	10.18	0.18	65.3	59.65	16.44	99.6	56.11	16.69	99.6	65.2	39.1	13.0	52.2	0.60	200.3	140	5.8	2.22	3.83	0.83	95.8	1.39	22.8	0.35	0.962	13.7		Clay, Low strength, with some shell fragments
ONSB41-11-B-1	SVV	10.32	0.32	66.4	55.02	16.60	97.9	52.74	16.77	97.8	67.3	40.4	13.5	53.9	0.60	200.5	137	5.7	1.63	2.55	0.47	97.1	1.39	25.2	0.37	0.915	16.2		Clay, Low strength, with some shell fragments
ONSB41-11-B-1	NGI	10.60	0.60	67.5	53.79	17.28	103.7	52.20	17.42	103.8	67.1	40.3	13.4	53.7	0.60	200.0	143	6.0	1.26	1.84	0.29	98.1	1.38	26.3	0.39	1.243	18.1		Clay, Low strength, with some shell fragments

Time == total time from start of test (end of filter flusing) to start of shearing





Dato/rev.: 2014-12-23/01

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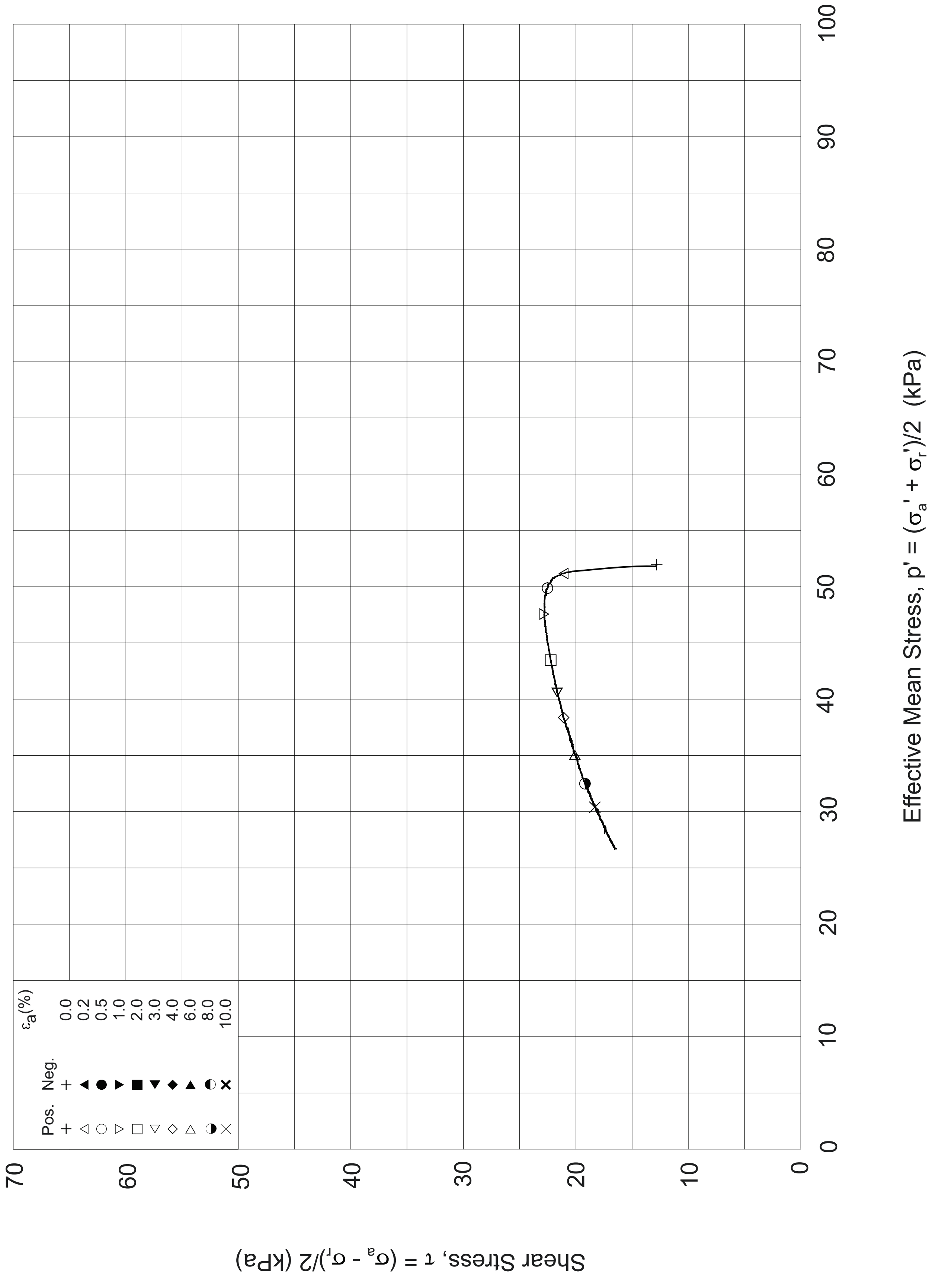
NGTS - Onsøy Test Site

Document No.	20160154-10					
Figure No.	GXX					
Date	2017-12-01	Drawn by/checked	PCa / MAS			

Triaxial test: **CAUC**

Boring:	**ONSB41**	Depth =	**10.18**	m	Consolidation stresses			
Tube:	**11**	po' =	**65.3**	kPa	(kPa)	max.	min.	final
Part:	**A**	w_i =	**59.6**	%	σ_{ac}' =	-	-	**65.2**
Test:	**1**	w_c =	**56.1**	%	σ_{rc}' =	-	-	**39.1**



Dato/rev.: 2014-12-23/01

BH_WFS2_H03_BATCH_E4_CIUC_01.Plot2.grf

NGTS - Onsøy Test Site

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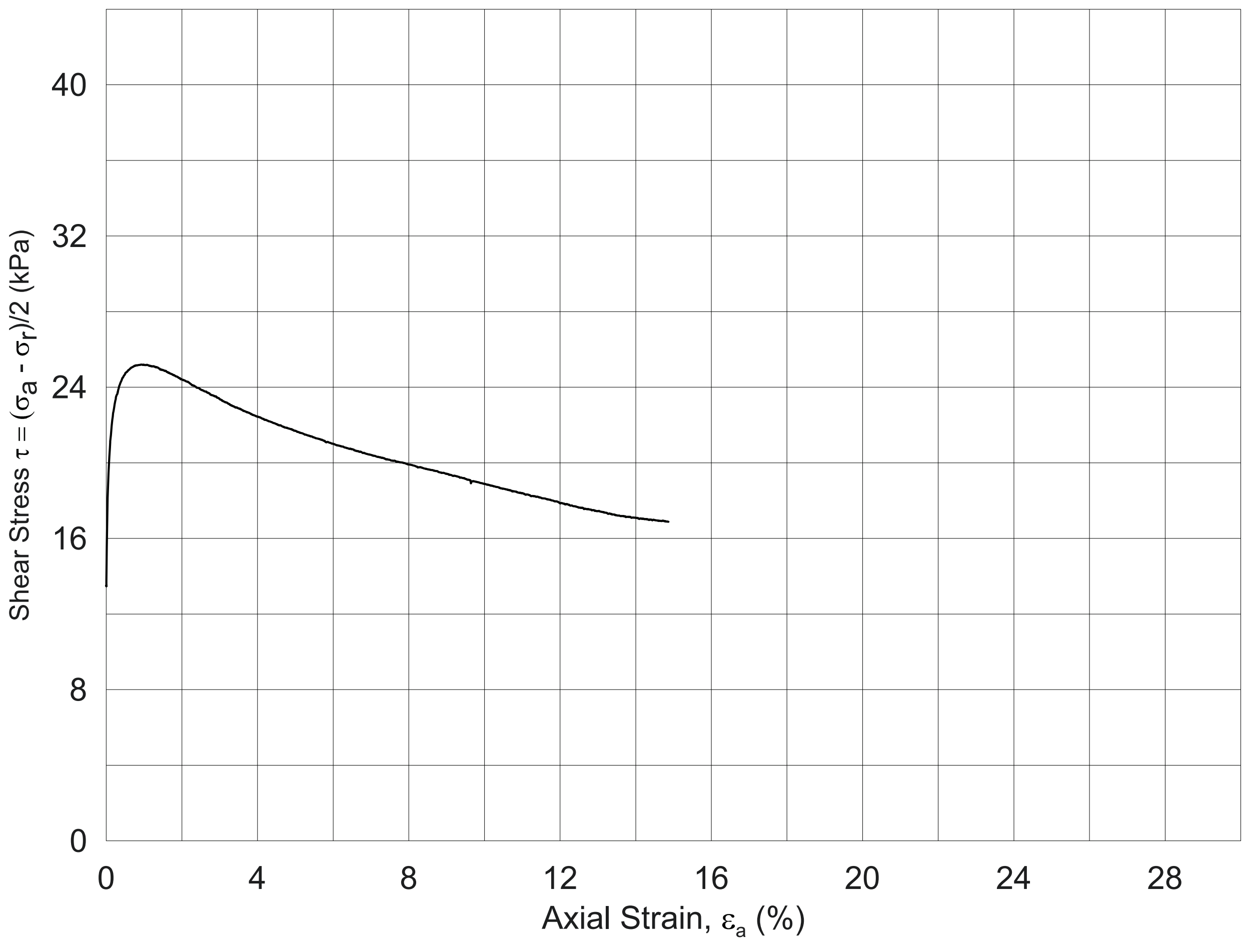
Triaxial test: **CAUC**

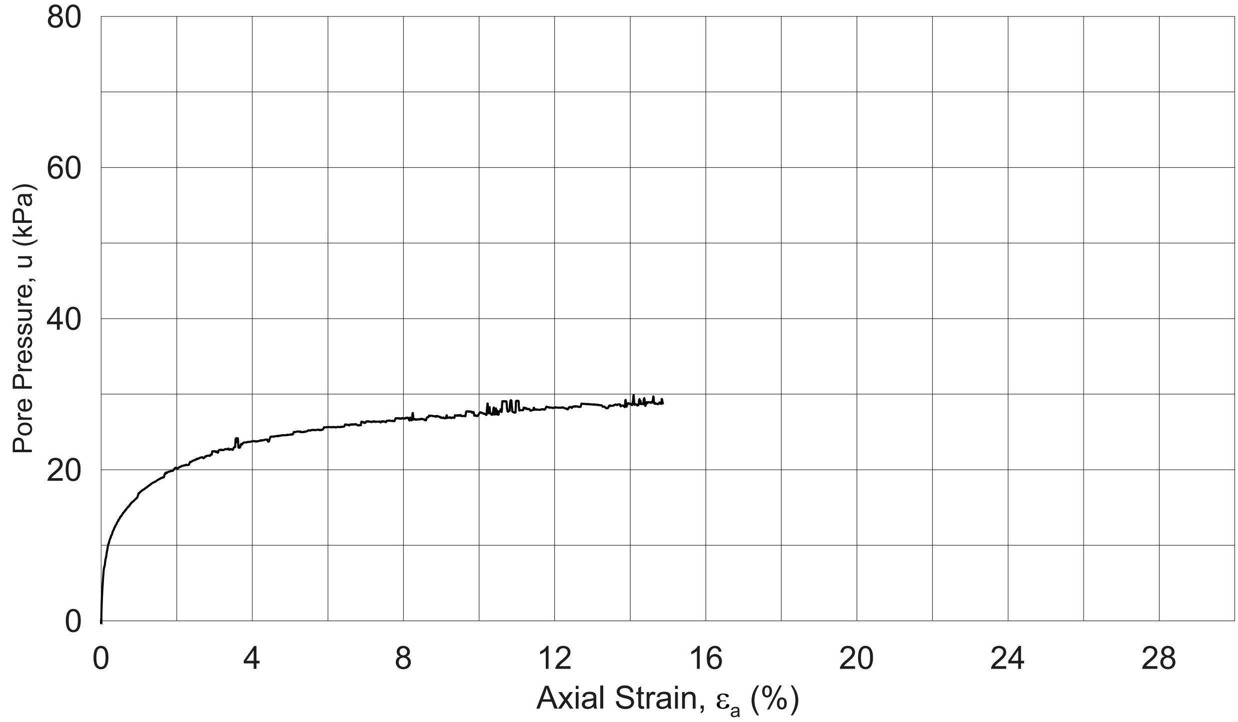
Figure No. GXX

Date 2017-12-01

Drawn by/checked PCa / MAS

Boring:	**ONSB41**	Depth =		**10.18**	m	Consolidation stresses					
Tube:	**11**	po'	=	**65.3**	kPa	(kPa)		max.	min.	final
Part:	**A**	w_i	=	**59.6**	%	σ_{ac}' =		-	-	**65.2**
Test:	**1**	w_c	=	**56.1**	%	σ_{rc}' =		-	-	**39.1**





Dato/rev.: 2014-12-23/01

BH_WFS2_H03_BATCH_E4_CIUC_01.Plot1.grf

NGTS - Onsøy Test Site

Document No. 20160154-10

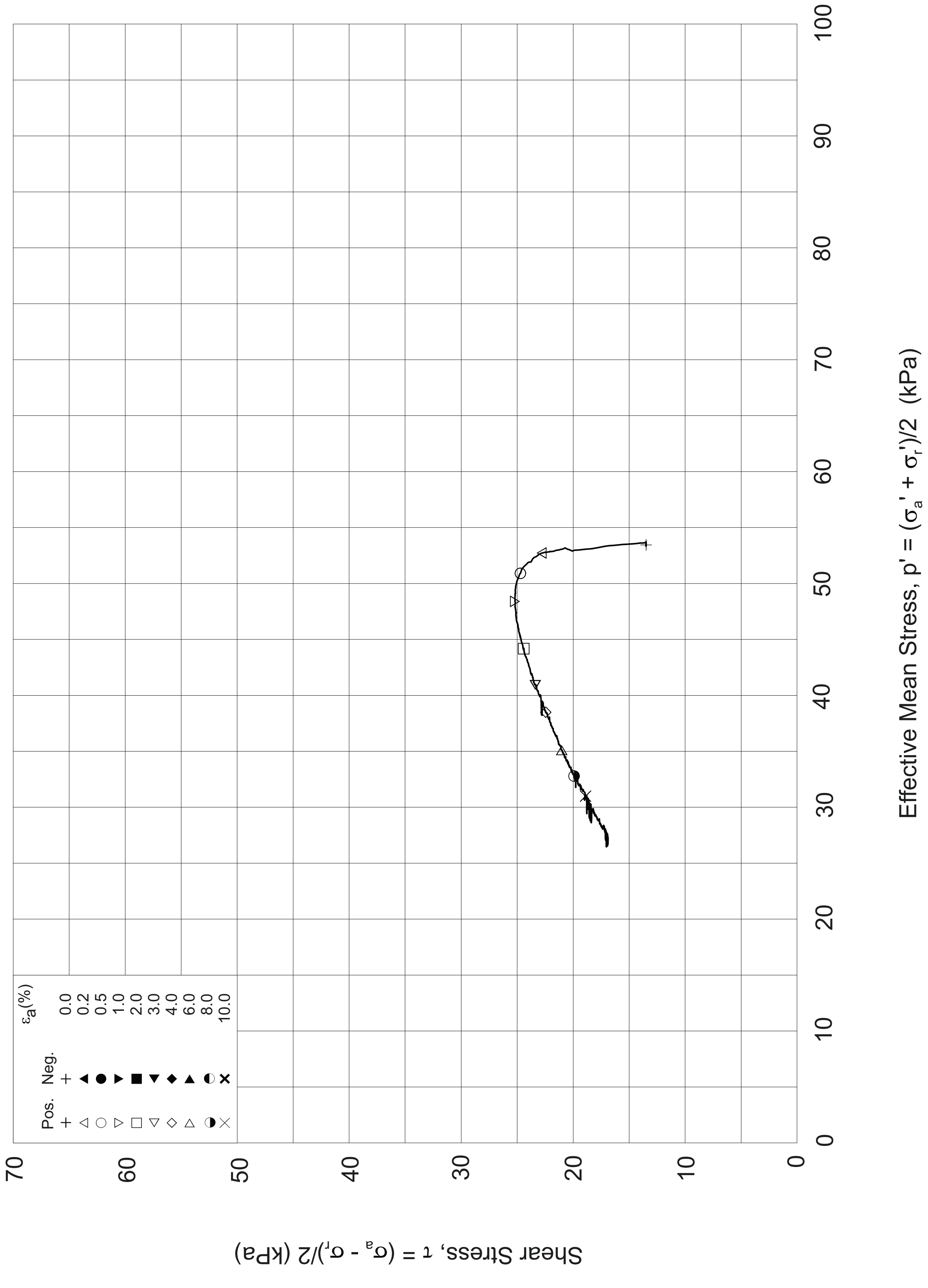
Figure No. GXX

Triaxial test: **CAUC**

Boring:	**ONSB41**	Depth =	**10.32**	m	Consolidation stresses					
Tube:	**11**	po' =	**66.4**	kPa	(kPa)	max.	min.	final		
Part:	**B**	w_i =	**55.0**	%	σ_{ac}' =	-	-	**67.3**		
Test:	**1**	w_c =	**52.74-**	%	σ_{rc}' =	-	-	**40.4**		

Date 2017-12-01

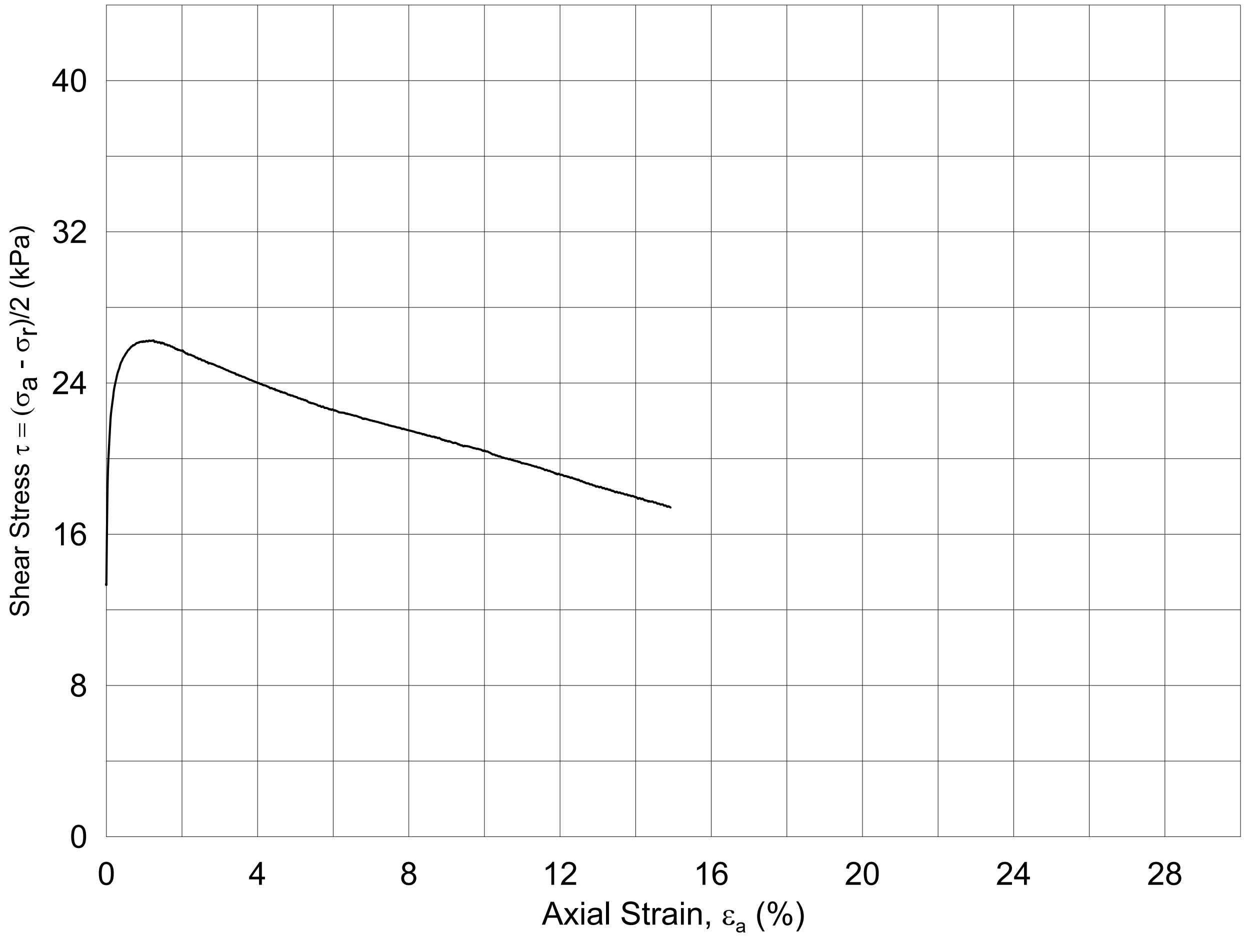
Drawn by/checked PCa / MAS

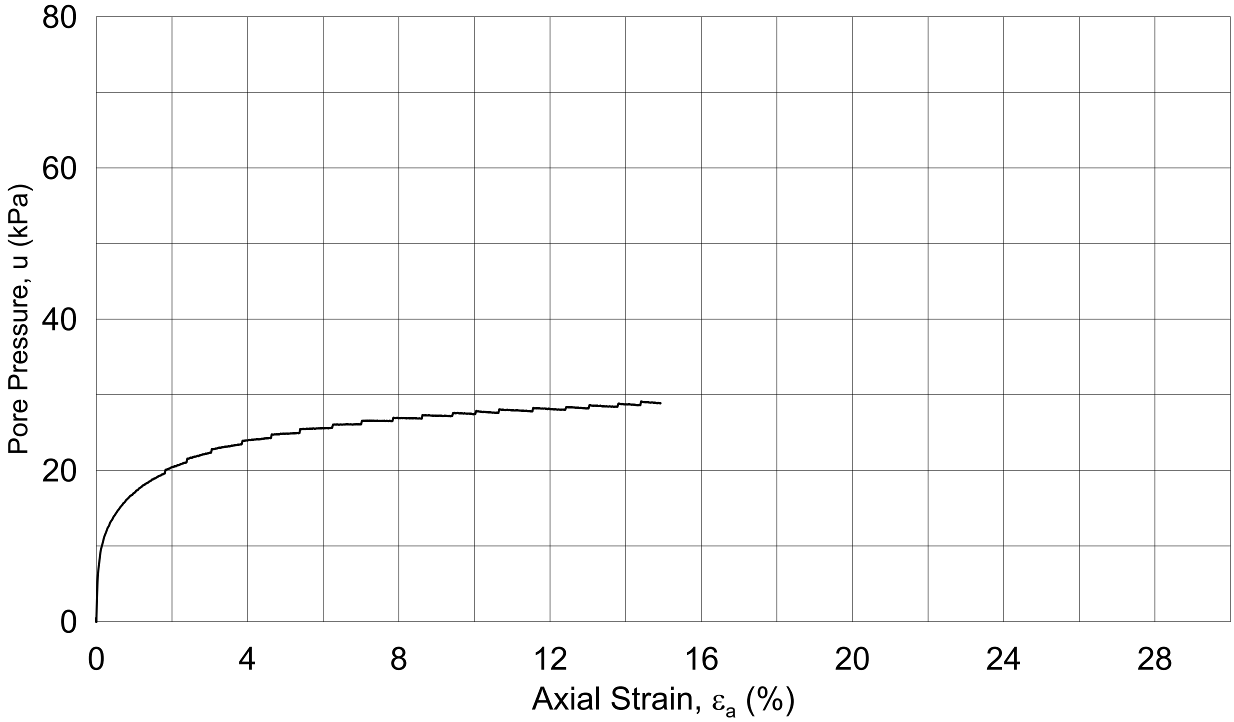


Dato/rev.: 2014-12-23/01

BH_WFS2_H03_BATCH_E4_CIUC_01.Plot2.grf

NGTS - Onsøy Test Site								Document No. 20160154-10
Triaxial test: **CAUC**								Figure No. GXX
Boring:	**ONSB41**	Depth =	**10.32**	m	Consolidation stresses			Date 2017-12-01 / Drawn by/checked PCa / GS
Tube:	**11**	po' =	**66.4**	kPa	(kPa)	max.	min.	final
Part:	**B**	w_i =	**55.0**	%	σ_{ac}' =	-	-	**67.3**
Test:	**1**	w_c =	**52.7**	%	σ_{rc}' =	-	-	**40.4**

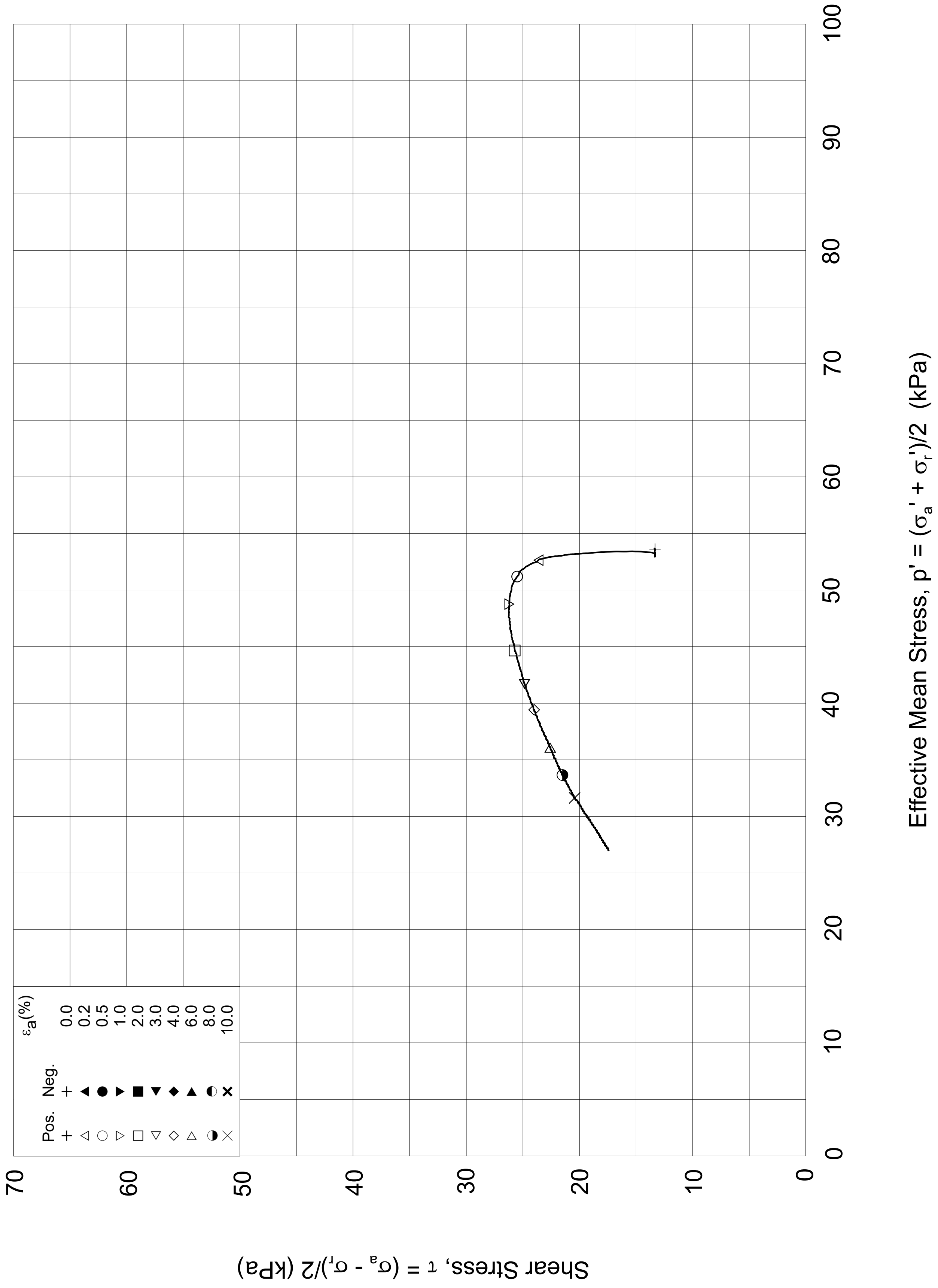




Dato/rev.: 2014-12-23/01

BH_WFS2_H03_BATCH_E4_CIUC_01.Plot1.grf

NGTS - Onsøy Test Site				Document No. 20160154-10
Triaxial test: **CAUC**				Figure No. GXX
Boring: **ONSB41**	Depth = **16.60** m	Consolidation stresses		Date 2018-02-12 / Drawn by/checked PCa / MAS
Tube: **11**	po' = **67.5** kPa	(kPa)	max. / min. / final	
Part: **C**	w_i = **53.79** %	σ_{ac}' =	- / - / **67.0**	
Test: **1**	w_c = **52.20** %	σ_{rc}' =	- / - / **40.3**	



Dato/rev.: 2014-12-23/01

BH_WFS2_H03_BATCH_E4_CIUC_01.Plot2.grf

NGTS - Onsøy Test Site										Document No. 20160154-10
Triaxial test: **CAUC**										Figure No. GXX
Boring:	**ONSB41**	Depth =	**10.60**	m	Consolidation stresses					Date 2018-02-12 / Drawn by/checked PCa / MAS
Tube:	**11**	po' =	**67.5**	kPa	(kPa)		max.	min.	final	
Part:	**C**	w_i =	**53.79**	%	σ_{ac}' =		-	-	**67.0**	
Test:	**1**	w_c =	**52.20**	%	σ_{rc}' =		-	-	**40.3**	

Kontroll- og referanseside/ Review and reference page

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OSLO
TRONDHEIM
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NORWEGIAN GEOTECHNICAL INSTITUTE
NGI.NO

Main office
PO Box 3930 Ullevaal St.
NO-0806 Oslo
Norway

Trondheim office
PO Box 5687 Sluppen
NO-7485 Trondheim
Norway

T (+47)22 02 30 00
F (+47)22 23 04 48
NGI@ngi.no

BIC NO. DNBANOKK
IBAN NO26 5096 0501 281
COMPANY NO
958 254 318MVA

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