

Arbeitsbericht NAB 20-08

**TBO Bülach-1-1:
Data Report**

**Dossier IX
Rock-mechanical and
Geomechanical Laboratory Testing**

February 2021

**National Cooperative
for the Disposal of
Radioactive Waste**

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P.O. Box 280
5430 Wettingen
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**TBO Bülach-1-1:
Data Report**

**Dossier IX
Rock-mechanical and
Geomechanical Laboratory Testing**

February 2021

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campaign, geomechanics, geomechanical laboratory testing,
triaxial tests, oedometric tests

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Nagra Arbeitsberichte NAB ("Working Reports") present the results of work in progress that have not necessarily been subject to a comprehensive review. They are intended to provide rapid dissemination of current information.

This NAB aims at reporting drilling results at an early stage. Additional borehole-specific data will be published elsewhere.

In the event of inconsistencies between dossiers of this NAB, the dossier addressing the specific topic takes priority. In the event of discrepancies between Nagra reports, the chronologically later report is generally considered to be correct. Data sets and interpretations laid out in this NAB may be revised in subsequent reports. The reasoning leading to these revisions will be detailed there.

This Dossier was prepared by a project team consisting of:

E. Crisci (formal analysis, investigation, writing)

S. Giger (project administration, conceptualisation, writing) and

L. Laloui (review, supervision)

The present report was prepared based on numerous contractor reports, to which many people from different laboratories contributed, namely B. Bohloli, M. Soldal, K. Halvorsen, J. Stenebråten, R. Stankovic, V. Vajdova, R. Ewy, A. Ferrari, M. Rosone, B. Lecampion, H. Baumgartner, C. Dietl and J. Szech.

Editorial work: P. Blaser and M. Unger.

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1 Introduction

1.1 Context

To provide input for site selection and the safety case for deep geological repositories for radioactive waste, Nagra has drilled a series of deep boreholes in Northern Switzerland. The aim of the drilling campaign is to characterise the deep underground of the three remaining siting regions located at the edge of the Northern Alpine Molasse Basin (Fig. 1-1).

In this report, we present the results from the Bülach-1-1 borehole.

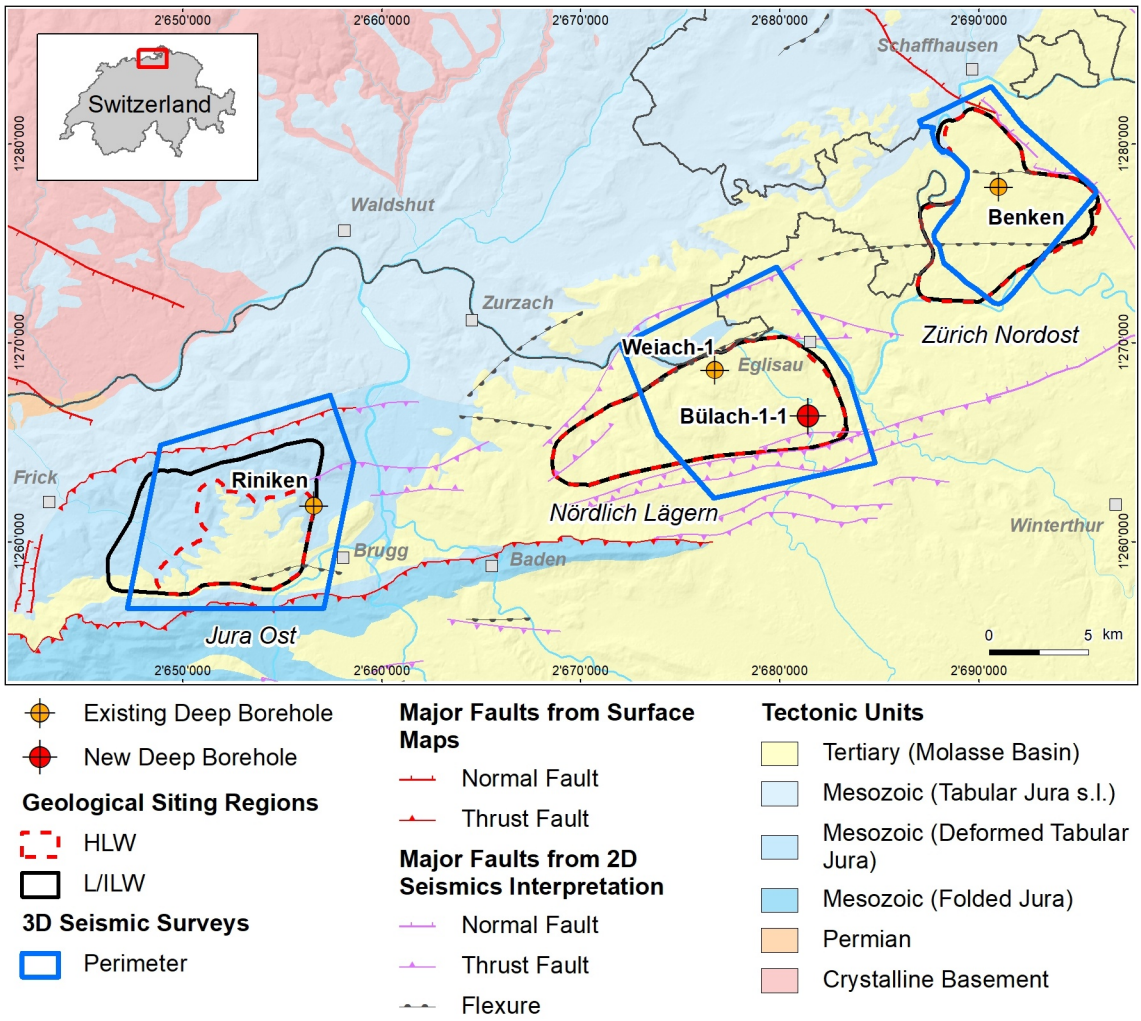


Fig. 1-1: Tectonic overview map with the three siting regions under investigation

1.2 Location and specifications of the borehole

The Bülach-1-1 (BUL1-1) exploratory borehole is the first deep borehole drilled within the framework of the TBO project. BUL1-1 is a vertical borehole. The drill site is located in the south-eastern part of the Nördlich Lägern siting region (Fig. 1-2). The borehole specifications are provided in Tab. 1-1. The lithostratigraphic profile and the casing scheme are shown in Fig. 1-3. Due to instabilities, the borehole was plugged back from 1'043.00 m to 918.70 m (*cf.* Dossier I). Resuming coring operations, a sidetrack was initiated with a kickoff point at 995.85 m. This sidetrack was labelled Bülach-1-1B (BUL1-1B). BUL1-1B reached the final depth of 1'370.19 m (measured depth MD core depth)¹. For easier communication and labelling, the name BUL1-1 is generally used for the complete vertical borehole unless stated otherwise. Fig. 1-3 shows the combined lithostratigraphic profile of BUL1-1.

Tab. 1-1: General information about the BUL1-1 borehole

Location	Bülach (Canton Zurich / ZH), Switzerland
Siting region	Nördlich Lägern
Coordinates	2'681'446.028 / 1'266'298.472
Elevation	389.62 m asl (top of rig cellar)
Borehole depth	1'370.19 m MD core depth
Drilling period	15 April 2019 – 27 November 2019 (spud – rig release)
Drilling company	Daldrup & Söhne AG
Drilling rig	Wirth B 152t
Drilling fluid	Water-based mud with varying amounts of Pure-Bore®, Pure-Bore® ULV, soda ash, sodium bicarbonate, xanthan gum

¹ *Measured depth (MD)* below top rig cellar along borehole trajectory. In all dossiers, depth refers to MD unless stated otherwise.

MD core depth refers to the depth marked on the cores. Note that petrophysical logs have not been shifted to core depth, hence MD log depth differs from MD core depth. The depth shift between core depth and log depth is documented in Dossier V, Appendix E-1.

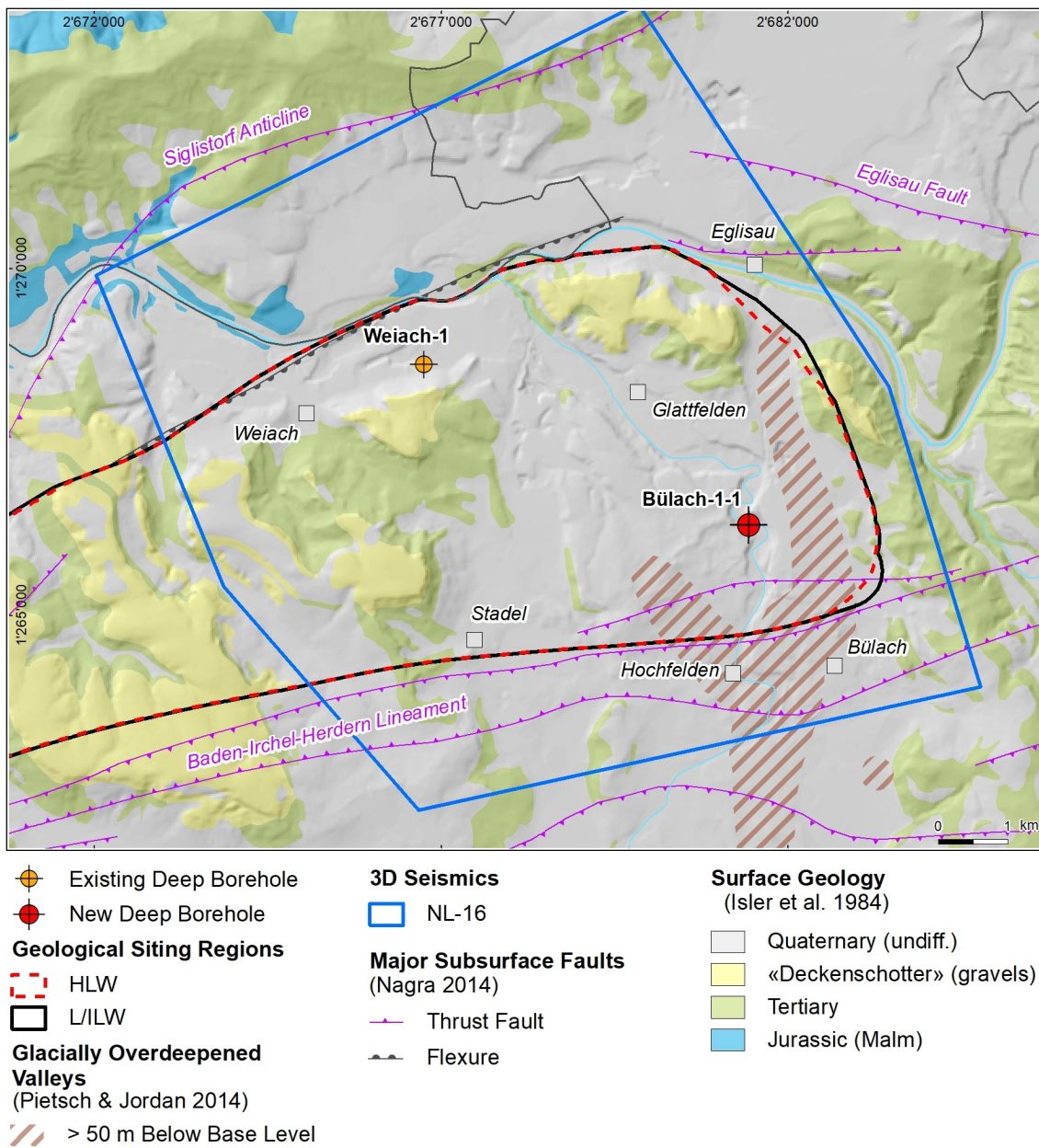


Fig. 1-2: Overview map of the investigation area in the Nördlich Lägern siting region with the location of the BUL1-1 borehole and the existing Weiach-1 borehole

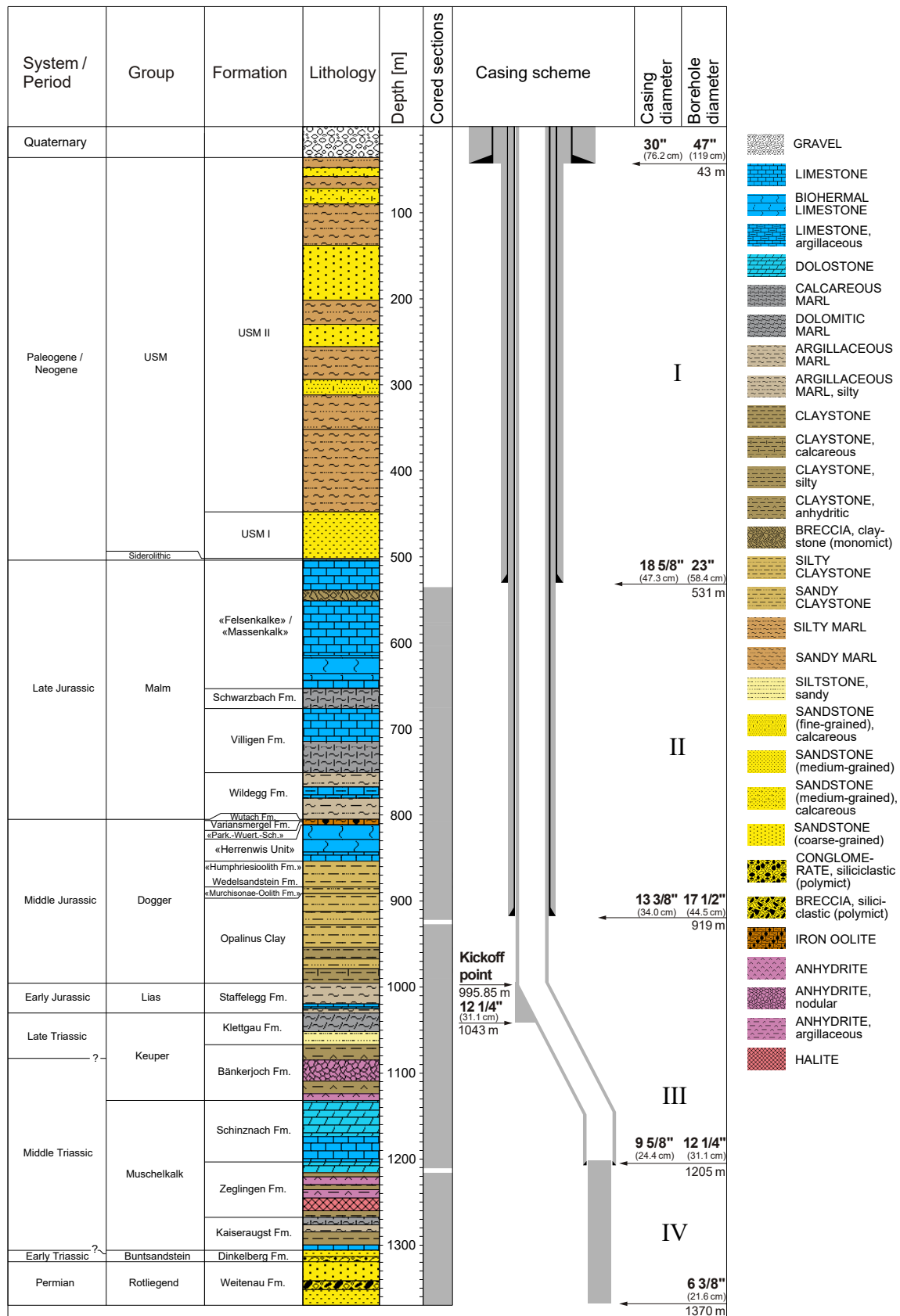


Fig. 1-3: Lithostratigraphic profile and casing scheme for the BUL1-1 borehole

1.3 Documentation structure for the BUL1-1 borehole

NAB 20-08 documents the majority of the investigations carried out in the BUL1-1 borehole, including laboratory investigations on core material. The NAB comprises a series of stand-alone dossiers addressing individual topics and a final dossier with a summary composite plot (Tab. 1-2).

This borehole documentation aims at early publication of the data collected in the BUL1-1 borehole. It includes the majority of the data available approximately one year after completion of the borehole. Some analyses are still ongoing (e.g. diffusion experiments, analysis of veins, hydrochemical interpretation of water samples) and will be published in separate reports.

The current borehole report will provide an important basis for the integration of datasets from different boreholes. The integration and interpretation of the results in the wider geological context will be documented later in separate geoscientific reports.

Tab. 1-2: List of dossiers included in NAB 20-08

Black marks the dossier at hand.

Dossier	Title	Authors
I	TBO Bülach-1-1: Drilling	M. Ammen & P.-J. Palten
II	TBO Bülach-1-1: Core Photography	D. Kaehr & M. Gysi
III	TBO Bülach-1-1: Lithostratigraphy	P. Jordan, H. Naef, P. Schürch, M. Schwarz, T. Ibele, R. Felber & M. Gysi
IV	TBO Bülach-1-1: Microfacies, Bio- and Chemostratigraphic Analysis	S. Wohlwend, H.R. Bläsi, S. Feist-Burkhardt, B. Hostettler, U. Menkveld-Gfeller, V. Dietze & G. Deplazes
V	TBO Bülach-1-1: Structural Geology	A. Ebert, L. Gregorczyk, E. Hägerstedt & M. Gysi
VI	TBO Bülach-1-1: Wireline Logging and Microhydraulic Fracturing	R. Garrard, J. Gonus, J. Desroches & E. Bailey
VII	TBO Bülach-1-1: Hydraulic Packer Testing	R. Schwarz, L. Schlickenrieder & T. Vogt
VIII	TBO Bülach-1-1: Rock Properties, Porewater Characterisation and Natural Tracer Profiles	M. Mazurek, L. Aschwanden, L. Camesi, T. Gimmi, A. Jenni, M. Kiczka, U. Mäder, D. Rufer, H.N. Waber, P. Wanner, P. Wersin & D. Traber
IX	TBO Bülach-1-1: Rock-mechanical and Geomechanical Laboratory Testing	E. Crisci, L. Laloui & S. Giger
X	TBO Bülach-1-1: Petrophysical Log Analysis	S. Marnat & J.K. Becker
	TBO Bülach-1-1: Summary Plot	Nagra

1.4 Structure of Dossier IX

The report is organised as follows:

- Chapter 2 presents the aims of the laboratory testing programs and introduces the laboratories involved.
- Chapter 3 provides an overview of core sampling, conditioning and storage methods, and also explains the non-destructive method used to evaluate core integrity prior to shipping to the laboratories.
- Chapter 4 introduces the testing methods used in the testing programmes and provides an overview of the executed tests per formation.
- Chapter 5 documents the results of the rock mechanical testing programme.
- Chapter 6 documents the triaxial and the oedometric test results of the geomechanical testing programme. Given the greater complexity of the testing procedure and the relevance of the test results for site evaluation, these test results are more rigorously documented than the rock mechanical test results.
- A short discussion on the representativeness of the results is provided in Chapter 7.

Furthermore, large documentation of the performed tests is reported in the appendix, and includes:

- A – Photo documentation of the rock mechanical testing programme
- B – XCT (medical X-ray tomography) cross-sections, with indication of the section used for geomechanical testing.
- C – Diagnostic plots of the triaxial tests (geomechanical testing programme)
- D – Mineralogical analysis (geomechanical testing programme)
- E – Triaxial test results (geomechanical testing programme)
- F – Test results in oedometric conditions (geomechanical testing programme)

2 Aims of the study and overview of the testing programme

2.1 Aims of the study and distinction of rock mechanical and geomechanical testing programmes

For site characterisation, rock mechanical and geomechanical properties are used for the following applications:

- a) Supporting data for the detailed analysis and interpretation of in situ micro-hydraulic fracturing tests
- b) Parametrise 3D geomechanical numerical models with (sub-)formation properties to assess the spatial variability of the stress tensor
- c) Design of underground repository access structures
- d) Hydro-mechanical properties of the host rock Opalinus Clay and its confining units
- e) Volumetric behaviour (consolidation and swelling) and hydraulic properties of the host rock Opalinus Clay and its confining units

To address points a) to c) «standard» rock mechanical tests were performed on core material in the stratigraphic interval Malm to Rotliegend, except for Opalinus Clay. «Standard» here means that the tests were performed according to national and international standards, notably the suggested methods of the International Society of Rock Mechanics (*cf.* Section 4.2). Standard tests were performed within the **"rock mechanical testing programme"**.

In contrast, laboratory testing under points d) and e) were conducted using specialised protocols (*cf.* Section 4.3) combining rock mechanical and soil mechanical aspects. Specifically, these protocols account for 1) the higher requirements of the low-permeability and fluid-sensitive Opalinus Clay and its argillaceous confining units to obtain robust test results, and 2) for the greater relevance of these units to evaluate the safety of a future repository. Tests performed using the specialised protocols are hereafter assigned to the **"geomechanical testing programme"**.

2.2 Overview of testing programme and contractors

2.2.1 Rock mechanical testing programme

The rock mechanical testing programme with core material of the BUL1-1 borehole comprised of the following testing methods:

- Fracture toughness using semi-circular bend specimen (FT)
- Indirect tensile strength (Brazilian)
- Uniaxial compressive strength (UCS)
- Triaxial compressive strength (TRX)

The following two laboratories were contracted:

- Geo-Energy Lab, Gaznat chair on GeoEnergy, Institute of Civil Engineering, EPFL (Switzerland). Main contact was Brice Lecampion.
- Gesteinslabor Dr. Eberhard Jahns (Germany). Main contacts were Carlo Dietl and Hansjörg Baumgartner.

2.2.2 Geomechanical testing programme

The geomechanical testing programme was specifically designed for Opalinus Clay and its confining units and comprised of the following testing methods with cores from the BUL1-1 borehole:

- Triaxial deformation tests with various stress paths (TRX)
- High-pressure oedometric testing (OED), including for some tests constant-head permeability phases
- Constant-head permeability tests in triaxial conditions (PERM)
- One-dimensional swelling tests (OS), carried out in oedometric conditions

Four laboratories were contracted for the geomechanical testing campaign:

- NGI (Norway), main contacts were Bahman Bohloli and Kristine Halvorsen
- RSTD (USA), main contacts were Rudy Stankovic and Russ Ewy
- Sintef (Norway), main contact was Jørn Stenebråten
- DI (Italy), main contact was Alessio Ferrari

Each laboratory is identified with a letter, used in the test results labelling (A = NGI, B = Sintef, C = RSTD, D = DI).

Specialised protocols were developed in recent years to evaluate hydro-mechanical properties of Opalinus Clay in both high-pressure oedometric tests (Favero 2017, Ferrari et al. 2016, Crisci et al. 2019) and triaxial deformation tests (Favero et al. 2018, Giger et al. 2018). In the case of triaxial testing, the testing procedures were further validated in a benchmarking study (Minardi et al. 2019, Minardi et al. 2020).

The one-dimensional swelling tests were also performed in the framework of the geomechanical testing programme but did not make use of advanced protocols.

3 Core sampling, storage and XCT scanning

3.1 Core sampling and conditioning

Cores used for geomechanical or rock mechanical analyses (hereafter referred to as GM cores) represent one of nine major categories sampled at the borehole. Each of these categories has special requirements to minimise potential effects on laboratory analyses, and hence special conditioning methods. Target exposure time to atmosphere was set to 20 minutes for all GM cores to allow geological description and documentation (incl. core scanner for photographic description). A complete archive of core photographs, and exposure time to atmosphere prior to embedding the cores in resin is made in Nagra's internal QA/QC documentation.



Fig. 3-1: Sampling and storage workflow of GM cores

(a) Inserting core directly into PVC tube (no plastic film). (b) Rack of PVC tubes during curing with clamp bar in core at the front row (left side). (c) Labelled PVC tubes with core and top spacer (used for clamping) embedded in epoxy resin. (d) Custom made core box used for transporting and archiving.

The cores were then immediately inserted in PVC tubes, and axially constrained with bar clamps (Fig. 3-1). The annulus was then filled with epoxy (Sikadur-52). The annulus between the core and the PVC tube was chosen very small (nominal 3.2 mm) to ensure no excessive temperature from curing of the epoxy develops². Further details of the conditioning procedure are provided in the field manual (Rufer 2019).

A total of 78 GM cores with a cumulative length of 34 metres were conditioned in this way. An additional 15 cores were sampled later at the core storage facility. This was done to enable rock mechanical testing to complement the in-situ testing programme (micro-hydraulic fracturing) where no conditioned core was secured, and only for hard rocks not sensitive to water exchange (drying).

3.2 Evaluation of core quality using XCT Scanning

To evaluate core integrity prior to shipping to the testing laboratories, all GM cores from Opalinus Clay and its confining units in the Upper Dogger and Lias (Staffelegg Formation) were analysed using medical X-ray computer-tomography (XCT). Details of the applied XCT method can be found in Keller & Giger (2019). The obtained image data had a voxel size of $0.25 \times 0.25 \times 0.4$ mm. By stacking individual scans, a virtual cross-section could be obtained for each core (Fig. 3-2). It is noted that the CT-number is a proxy for material density. This enabled the detection of small cracks and a physical representation of the variability of material properties. These cross-sections were the basis for the selection of sub-coring for the geomechanical testing programme (Chapters 5 and 6, and Appendix B).

² Maximum measured temperature in the laboratory was $< 27^{\circ}$ C.

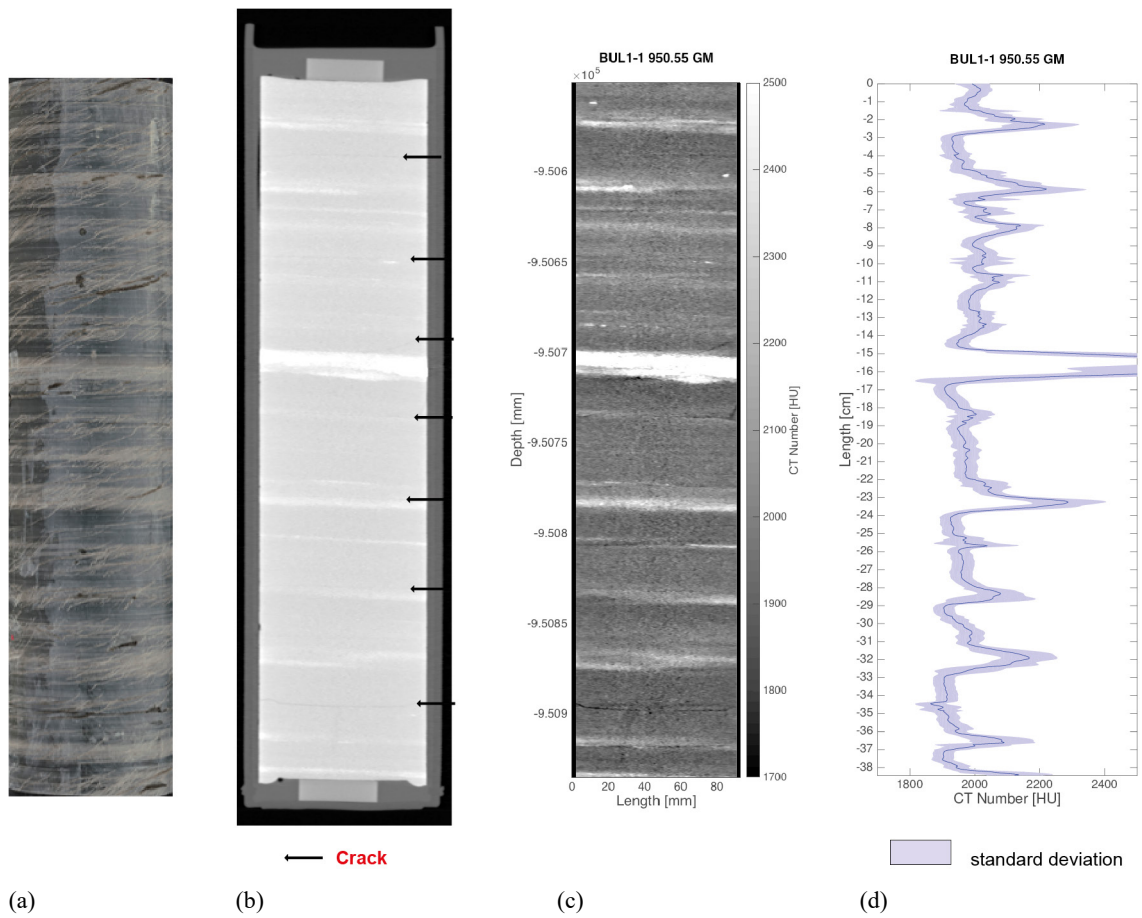


Fig. 3-2: Cross-section of XCT stack

(a) core picture; (b) entire core in PVC tube and resin (both dark grey) and embedded spacers (light grey – subhorizontal cracks are indicated by arrows); (c) optimised image for the core only; (d) vertical profile with CT number (Hounsfield Units).

4 Testing methods and overview of executed tests

4.1 Convention of sample geometry

Sedimentary geomaterials typically display transversely isotropic mechanical properties which are related to the orientation of the bedding. To study this anisotropic response, cylindrical specimens were tested in three orientations with respect to the bedding (Fig. 4-1).

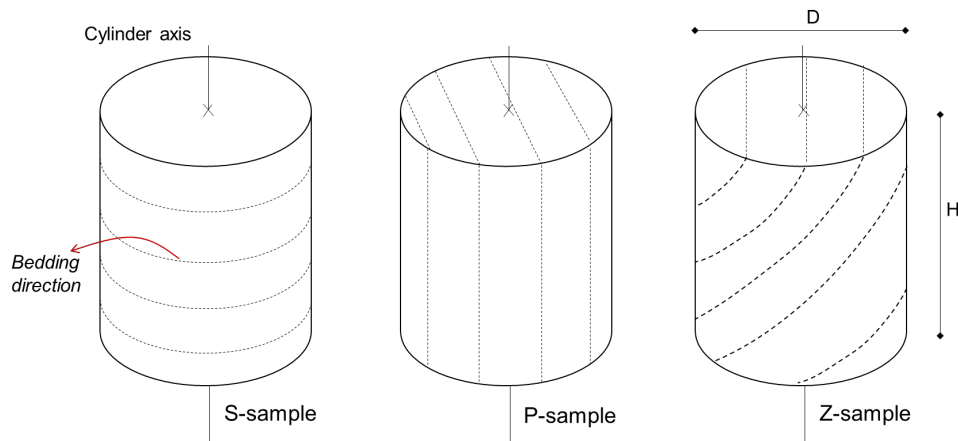


Fig. 4-1: Terminology of specimens in relation to bedding orientation

In this report, the following convention applies:

- S-samples: the cylinder axis is perpendicular to bedding direction
- P-samples: the cylinder axis is parallel to bedding direction
- Z-samples: the cylinder axis is at an angle with respect to bedding.

The convention applies to all kind of tests, however the ratio of diameter (D) to height (H) of the specimen is different. The ratio ranges from $D:H \approx 1/2$ for triaxial and uniaxial tests, to $D:H \approx 1.4$ in Brazilian tests and $D:H \approx 2.5$ for oedometric tests.

4.2 Rock mechanical testing programme – testing methods

4.2.1 Fracture toughness using semi-circular bend specimen

Semi-circular specimens were prepared according to the ISRM recommended specifications (Kuruppu et al. 2014). Specimens were cut with a blade and water at room temperature as a cutting fluid.

In the semi-circular notched specimen sketched in Fig. 4-2, a is the length of the notch, R the core radius, B the specimen thickness, and s the distance between the base supports for the test. P denotes the applied load.

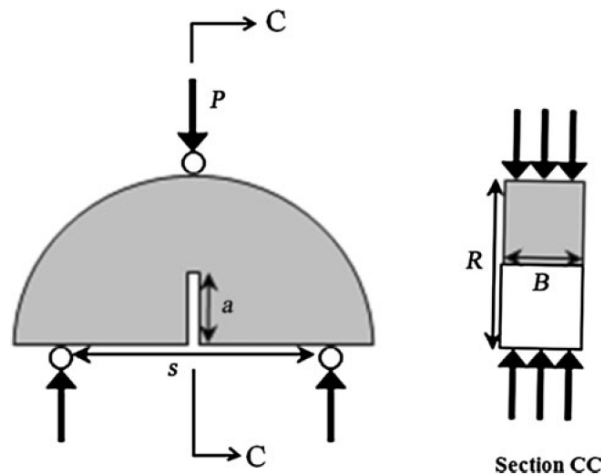


Fig. 4-2: Semi-circular bending test geometry
After Kuruppu et al. (2014).

4.2.2 Indirect tensile strength test (Brazilian)

The Brazilian test is a uniaxial compressive strength test aimed for the indirect determination of tensile strength, and tests were done according to ASTM standard D3967-16 (ASTM 2016).

Test specimens have a diameter of 30 mm and a width of 21 mm (thickness-diameter ratio of 0.70). Testing plugs were drilled either perpendicular to bedding (so-called "S" direction) or parallel to bedding (so-called "P" direction). The plugs were drilled with a diamond tipped hollow bit. Cooling during drilling and trimming was performed with the low viscous volatile mineral oil Ilocut EDM 180 (Castrol). Plug trimming to the required length was carried out with a diamond chop saw with slow constant velocity feeding.

During the test, cylindrical specimens are loaded perpendicular to the cylinder axis (Fig. 4-3). For clarification, it is noted that loading was always done perpendicular to the core axis. In case of the P-samples (Fig. 4-1) loading was parallel to bedding and the tensile force perpendicular to bedding. In case of the S-samples loading and tensile force are both parallel to bedding.

The constant loading rate during testing was 0.30 MPa/s. In case of the P-samples (*cf.* Section 4.1) the specimen was oriented such that loading was parallel to bedding, with the tensile force, consequently, acting perpendicular to bedding. In case of the S-samples loading and tensile force are both perpendicular to bedding. The Brazilian tests were carried out on a high precision digitally controlled electromechanical testing machine with a maximum load range of 100 kN (accuracy class 0.5).

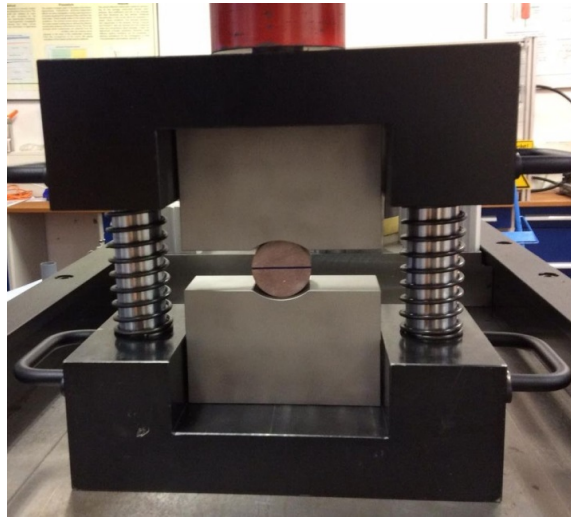


Fig. 4-3: Brazilian testing configuration with curved jaws
Plug diameter is 30 mm.

4.2.3 Unconfined compressive strength test

Unconfined compressive strength (UCS) tests were done according to ASTM standard D7012-14e1 (ASTM 2014).

Test specimens have a diameter of 25.4 mm and a length of at least 50.8 mm (length-diameter ratio of 2:1). Testing plugs were drilled either perpendicular to bedding (so-called "S" direction) or parallel to bedding (so-called "P" direction). The plugs were drilled with a diamond tipped hollow bit. Cooling during drilling and trimming was performed with the low viscous volatile mineral oil Ilocut EDM 180 (Castrol). Plug trimming to the required length was carried out with a diamond chop saw with slow constant velocity feeding. The accuracy in plane parallelism is in accordance with ASTM D4543-19 (2019).

The UCS tests were performed on a digitally controlled, servo-hydraulic testing machine with a maximum load range of 600 kN (accuracy class 1), and at a constant strain rate of 10^{-5} s^{-1} .

Axial deformation of the sample was measured with one LVDT displacement transducer directly at the sample to avoid the load frame deformation being included in the deformation results of the specimens. Radial deformation of the sample is measured diametric with a transverse extensometer (Fig. 4-4).

Throughout the report, the strains are calculated as the ratio of the variation in height (or diameter) to the initial height (or diameter).

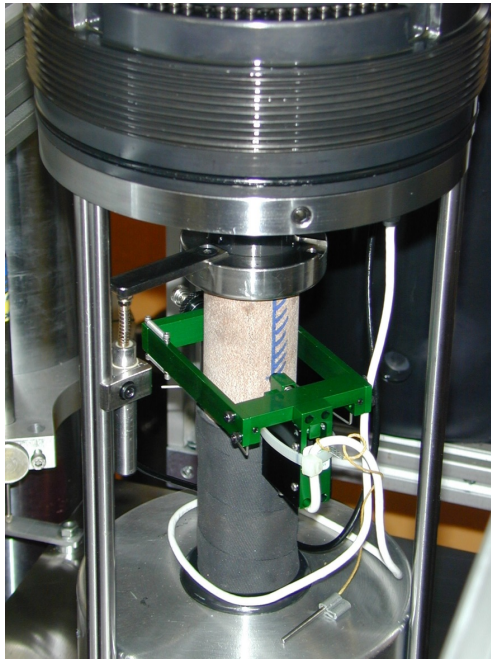


Fig. 4-4: UCS testing configuration

Note axial LVDT and transverse extensometer. Specimen diameter is 25.4 mm.

4.2.4 Triaxial deformation test

Triaxial deformation tests (TRX) tests were done according to ASTM standard D7012-14e1 (ASTM 2014) for single-stage procedure, but also included a hysteresis cycle (unload-reload) before and after peak strength (below).

Test specimens have the same dimensions as for the UCS tests (diameter 25.4 mm, length 50.8 mm) and were prepared in the same manner (*cf.* Section 4.2.3). The accuracy in plane parallelism is also in accordance with ASTM D4543-19 (ASTM 2019).

Axial (average of three LVDTs) and radial deformation of the sample (diametric with one single strain gauge) were measured 'in-vessel' to avoid the load frame deformation being included in the results of the deformation of the individual specimen (Fig. 4-5). The measurement range of the LVDTs is ± 5 mm; their sensitivity in the range between 9 and 60 mV; non-linearity is 0.15 % full scale. Three LVDTs were used to observe sample distortions that will influence the strength measurements in a non-acceptable manner. The differential axial load (strain gauge) also was measured 'in vessel' with a load cell mounted directly below the specimen to eliminate the influence of piston friction effects to the stress measurement. The load cell used for the first specimens is a 600 kN load cell of accuracy class 0.5.

The artificial brine was composed of demineralised water and 5 g/l NaCl. An exception was the anhydrite specimen in test BUL1-1B-1243TRX_S1 for which no fluid was used. The pistons are perforated to enable drained conditions. Side drains were not used. No spherical seats were used. All the important parameters were recorded automatically with different intervals, adjusted to expected speed of changes. No smoothing or filtering algorithms were applied.

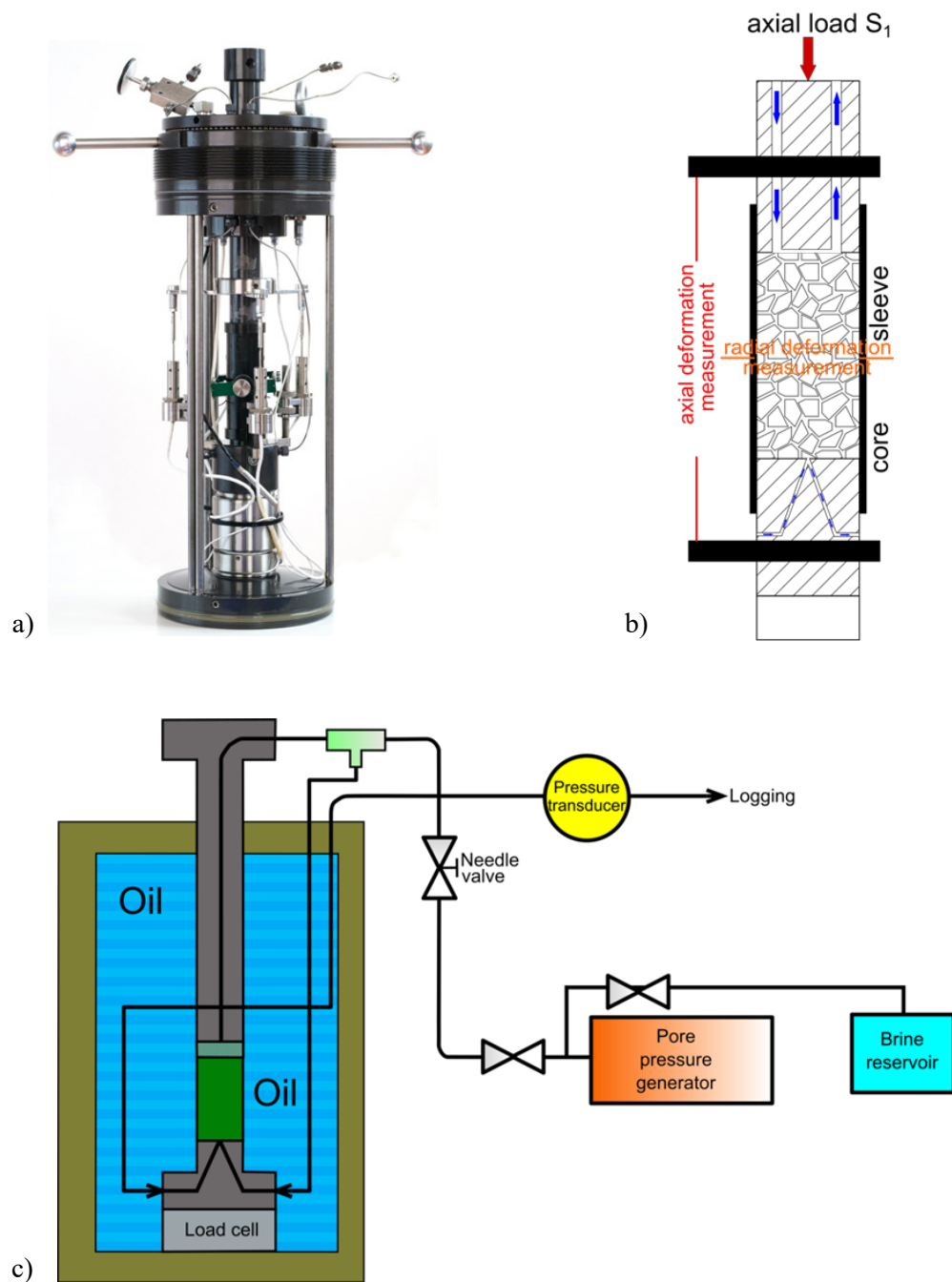


Fig. 4-5: Triaxial testing configuration used for rock mechanical testing

Specimen in membrane with axial and radial strain sensors, pore fluid line and ultrasonics (a), schematic of the specimen (b) and the pressure cell with pore fluid system (c).

During all phases of the tests, ultrasonic velocities, both P- and S-waves, were logged and monitored according to ISRM standards (Aydin 2014). The piezometer resonance frequency was 1 MHz.

The theoretically possible stress paths in a triaxial testing configuration with two independent controls (axial and radial stress) are illustrated in Fig. 4-6 in mean total stress (p) versus deviatoric stress (q) space, where:

$$p = \frac{\sigma_a + 2\sigma_r}{3} \quad (4-1)$$

$$q = \sigma_a - \sigma_r \quad (4-2)$$

where σ_a and σ_r are the total axial (direction of the cylinder axis) and radial (direction perpendicular to the cylinder axis) stress, respectively.

The upper part of the plane (compression part) summarises the stress paths that induce axial compression of the tested specimen. The axial compression can be induced by either increasing the axial stress σ_a (CTC path), reducing the radial stress σ_r (RTC path) or increasing σ_a and reducing σ_r at the same time (TC path) keeping the mean total stress constant. The stress paths that induce an axial extension of the specimen are illustrated below the p -axis in Fig. 4-6. The axial extension can be induced by either decreasing the axial stress (RTE path), increasing the radial stress (CTE), or decreasing σ_a and increasing σ_r at the same time (TE path) keeping the mean total stress constant. Therefore, when the term reduce is used to define the stress path, it means that one of the two stresses is decreased while the other is kept constant. On the other hand, when the term conventional is used to define the stress path, it means that one of the two stresses is increased and the other is kept constant. It is noted that the illustration in Fig. 4-6 is for total stress only, so the tests can be conducted in either drained or undrained conditions. For the triaxial tests of the rock mechanical testing programme, CTC and TC stress paths were executed.

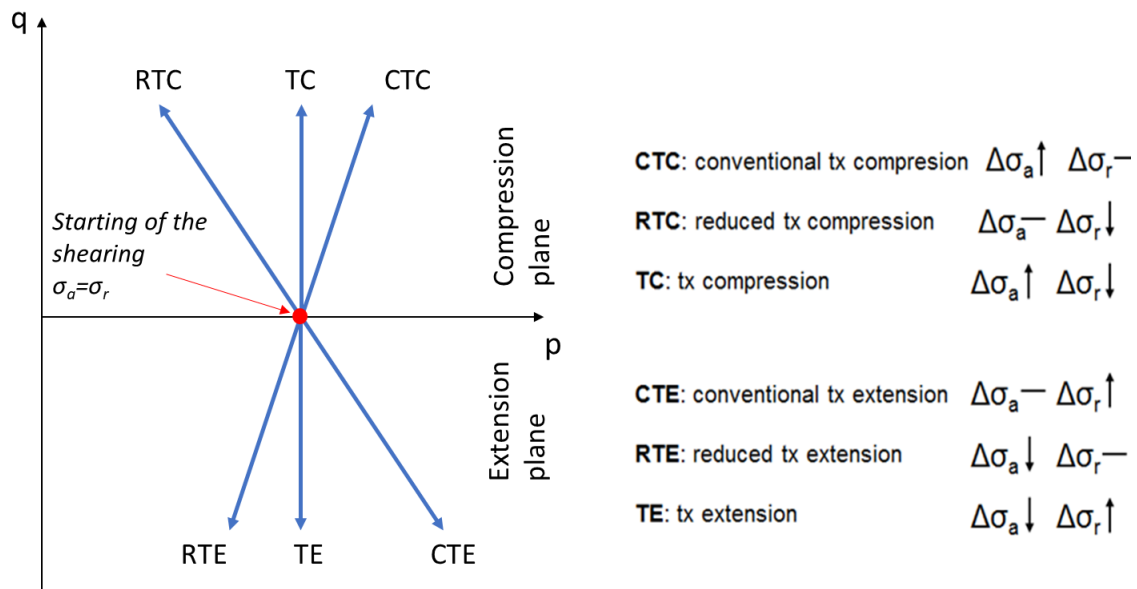


Fig. 4-6: Summary of possible stress paths that can be used to perform triaxial tests

On the right side, it is indicated which of the two stresses (axial σ_a and radial σ_r) is increased or decreased.

Test procedure

The testing procedure consisted of a maximum of four main phases:

1. Isotropic loading phase, during which the radial confining pressure and the axial load are increased simultaneously. The needle valve to the pore pressure generator is opened after the total stress has reached an equivalent value of 2 MPa. Both total mean stress (p) and fluid pressure (u_w) are then increased in concert to their respective target values: 10 to 15 MPa for p and 5 MPa for u_w .
2. *Selected tests only*: the pore-fluid-coupling parameter during isotropic loading (Skempton's B value) is evaluated in undrained conditions. During the Skempton's B test the change in pore pressure over time (Δu_w) is determined as a function of the change in isotropic pressure increase over time (Δp) (u_w is allowed to equilibrate at least for 20 minutes up to 12 – 24 hours):

$$B = \frac{\Delta u_w}{\Delta p} \quad (4-3)$$

In the present case three Skempton's B tests were carried out per sample with confining pressure increase Δp of 1 MPa.

3. *Selected tests only*: a consolidation phase is performed to determine the consolidation coefficient c_{vi} . An isotropic total stress increase (or decrease) is applied to the sample and the deformations in time are recorded and analysed. The deformation evolves according to the dissipation of the pore water over (or under) pressure induced by the load variation. Calculation of the consolidation coefficient c_{vi} (in mm²/h) is done according to the following equation (Head 1998):

$$c_{vi} = \frac{\pi D^2}{\lambda t_{100}} \quad (4-4)$$

with D as the plug diameter and t_{100} as the theoretical time to 100 % consolidation. The theoretical time t_{100} is found by extrapolating the linear portion of the volumetric strain versus square root of the experimental time curve (with time in h) to the final strain value. Lastly, λ is a constant depending on drainage boundary conditions.

Depending on the lithology this test took up to several days. A consolidation phase was only requested for samples that were estimated to be high in clay content, and therefore with potentially low permeability; for those tests, in order to allow proper drained or undrained conditions, strain rate was adjusted according to the consolidation coefficient estimation

4. The fourth testing phase is the shearing phase or triaxial test, applying a constant deviatoric stress or strain rate. The standard strain rate was 10^{-6} s^{-1} but was adjusted based on theoretical calculations (Head 1998) using the consolidation coefficient determined in phase 3. Two tests required a lower strain rate.

The shearing phase of the triaxial tests was preceded in selected cases by a Skempton's B test for verification of specimen saturation and, subsequently, by a determination of the consolidation coefficient c_v as base for strain rate calculation.

The strain rate of the single-stage triaxial tests ranges between 10^{-5} s^{-1} (only the dry test on anhydrite specimen BUL1-1B_1243TRX_S1) and 10^{-7} s^{-1} ; the majority of the tests were done at a strain rate of 10^{-6} s^{-1} . Strain rate was chosen based on lithological criteria and partly also on the results of the determination of the consolidation coefficient c_v (in two cases). Generally, clay-rich

rocks, such as clay- and siltstone or marl were tested at lower strain rates than limestone, sandstone and anhydrite.

4.3 Geomechanical testing programme – testing methods

4.3.1 Synthetic pore fluid

The artificial porewater (APW) used in the geomechanical tests is based on the recipe derived from the investigations in the Schlattingen borehole (Mäder in Wersin et al. 2013), and the composition is reported in Tab. 4-1. This recipe defines a porewater saturated with respect to calcite and dolomite under atmospheric CO₂ partial pressure (lab conditions).

Towards the end of the geomechanical testing programme, the porewater chemistry was also constrained for the BUL1-1 core material (*cf.* Dossier VIII). These investigations confirm the general Na-Cl water type, although with a higher salinity; e.g. chloride contents were found to be about twice as high in the Dogger and Liassic intervals compared to the APW (9 – 14 g/L vs. 6.7 g/L). In addition to saturation with carbonate minerals, the investigations suggest saturation with respect to sulphate minerals that can be present in trace amount in these rocks (celestite and eventually gypsum). But the APW was not changed for the final geomechanical tests, to be consistent with earlier tests. There was also a slight concern that higher mineralisation of the APW could lead to precipitation of saturated phases in the pore fluid lines. Finally, the higher ionic strength in the pore water found in the BUL1-1 core is not anticipated to have a significant effect on hydromechanical properties (Witteveen et al. 2013, Ewy 2014, Tuttolomondo 2021).

Tab. 4-1: Recipe used for the preparation of the APW.

Compound	mmol/kg _{H2O}	g/kg _{H2O}
NaCl	115.26	6.7356
NaHCO ₃	0.54	0.0456
CaCl ₂ 2H ₂ O	11.91	1.7510
KCl	2.55	0.1902
MgCl ₂ 6H ₂ O	9.17	1.8635
Na ₂ SO ₄	24.00	3.4089

4.3.2 Triaxial deformation test (specialised protocol)

To assess the mechanical properties of Opalinus Clay and confining units, dedicated testing protocols were established (Minardi et al. 2019).

The extraction of the specimens was carefully performed in order to minimise water content loss during the operation. All involved laboratories used a drilling machine. In particular, Lab B used a special core barrel configuration where an internal piston was used to stabilise the specimen during drilling by the application of a small axial force. All labs used hydrocarbons as cooling fluids during the drilling of the specimens.

Computed tomography of the cores after drilling were used to identify the most suitable sections for extracting test specimens (*cf.* Appendix A). The target zones were selected upon EPFL and Nagra's guidance.

The conventional testing procedure indicated by Nagra to carry out triaxial tests foresaw the following steps (Fig. 4-7a):

1. An initial isotropic total stress (0.5 – 1 MPa) is applied to the specimen to ensure contact with the axial piston. The saturation of the specimen is then performed at constant volume to minimise sample disturbance (i.e. fissure opening during re-saturation); axial and radial stresses are progressively increased to keep the specimen's volume constant while it is put in contact with fluid at low back pressure, lower than 100 kPa (*ss1*). Next, the fluid back-pressure is increased to 2 MPa (the total stresses are also increased to keep the effective stress constant) (*ss2*), to dissolve possible air remaining in the tubing or the material. This procedure allows the determination of the axial and radial swelling pressures. The confining stress is then set equal to the axial stress (*ss3*).
2. The assessment of the specimens' saturation is carried out by measuring the Skempton's B coefficient (B-check test). At least three isotropic undrained loadings are performed. More steps are applied when unsatisfactory values are obtained.
3. Drained consolidation of the specimen to the target consolidation stress is performed; consolidation effective stresses range between 4 and 17 MPa. The pore fluid pressure is kept constant to 8 – 10 MPa during consolidation. A heating phase was foreseen for selected samples and the rest of the test performed at high temperature.
4. The final stage is the shearing of the specimens at constant axial strain rate. An unloading reloading loop is performed, then the sample is sheared until the specimen's failure and achievement of the post-peak constant deviatoric stress, in undrained or drained conditions

This conventional testing procedure was adopted by Lab A and Lab B. A different procedure was used by Lab C and is referred to as «alternative procedure» (Fig. 4-7b). This alternative procedure foresees the equalisation of the specimen to different relative humidity values (between 92 % and 98 %) in desiccators before testing in order to achieve different saturation conditions. After that, the specimens were mounted in the rig, and an initial undrained loading was carried out until an increase of the pore pressure was detected. At this stage, the specimen was assumed to have achieved the fully saturated condition; B-check test was then performed in three steps. A pressure transducer was placed inside the vessel very close to the specimen to monitor the evolution of the pore fluid pressure during the application of the external mechanical stress. When this testing procedure is adopted, no artificial pore fluid is added to the specimen.

The laboratories involved in the triaxial testing programme with specialised protocols, prepared cylindrical specimens respecting the ratio $D:H \approx 0.5$. In particular the specimen nominal diameters are:

- Lab A: $D = 25$ mm
- Lab B: $D = 25.4$ mm
- Lab C: $D = 19.1$ mm

All the labs used both drainage systems, both radial and at the cylinder edges. The strain rates to apply during shearing were defined according to the drainage conditions and the specimen dimensions (i.e., drainage length).

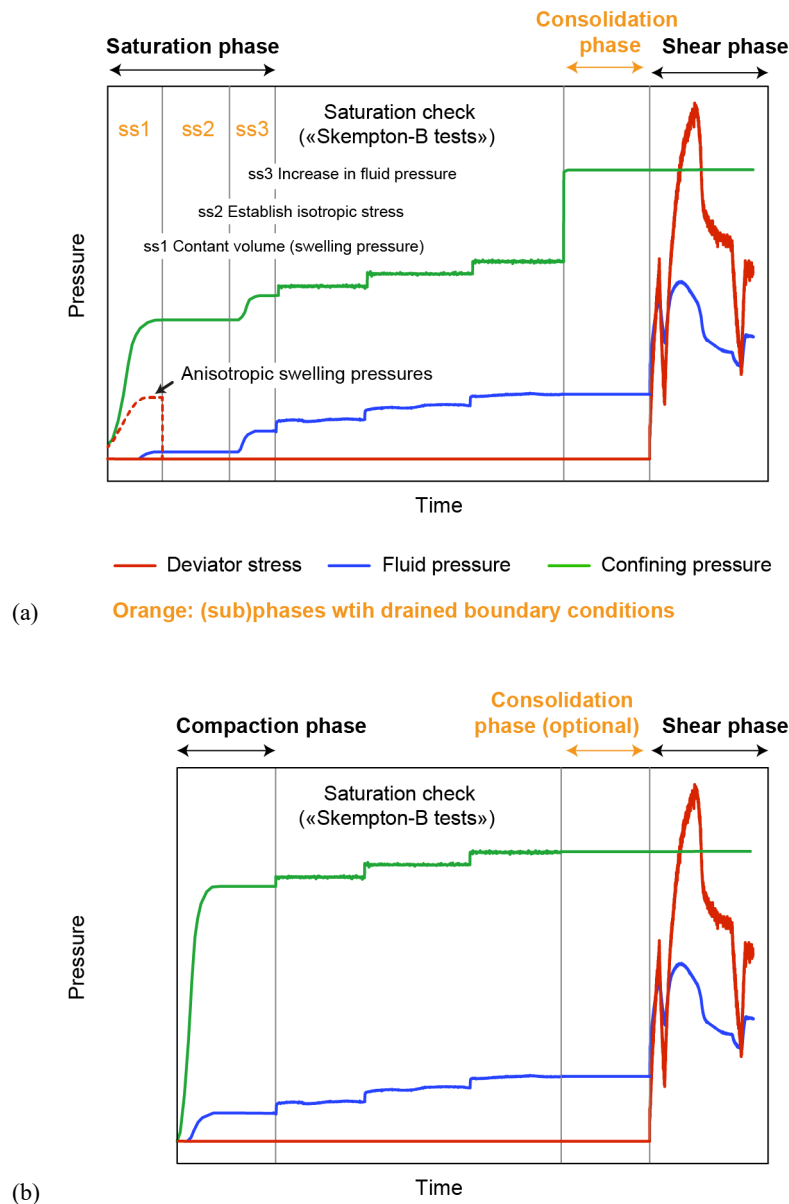


Fig. 4-7: Pressure-time sequences of triaxial deformation tests (specialised protocols)
 (a) conventional and (b) alternative testing procedures for undrained boundary conditions during the shear phase.

4.3.3 Oedometric test (high-pressure, specialised protocol)

Specimens for oedometric testing are of cylindrical shape, with Diameter (D) to height (H) ratio $D:H \geq 2.5$. The nominal dimensions of the specimens are:

- Lab A: D = 50.4 mm; H = 20.0 mm;
- Lab D: D = 35.0 mm; H = 12.5 mm;

The operators cut a pre-defined section of the core with a saw, through the PVC tube to minimise exposure of the geomaterial to the atmosphere. Then, the obtained slice is cut into a smaller piece

to be handle within a lathe machine. No air nor water is used during the sawing process. Planarity and parallelism of the bases are verified, and the dimensions of sawn samples are measured. In addition, Lab A covered the cut parts with a hydrocarbon oil and performed lathing at high humidity to limit the humidity gradient between the sample and the laboratory atmosphere. After the final specimen diameter was reached, the sample was wiped free of oil (using oil-only absorbent).

The specimen is then put into the oedometric ring, a rigid metallic ring to constrain the radial strains; silicon grease was placed on the internal surface of the ring to reduce the friction between the specimen and the metallic ring.

The specimen disks are loaded axially, and deformation is recorded with time.

Specimens are re-saturated within the apparatus in isochoric conditions. The axial deformations are kept constant by increasing the axial stress. Pore pressure is applied from the bottom side of the specimen at 50 kPa. When the swelling pressure generated by the saturation at isochoric conditions stabilises, pore pressure equal to 50 kPa is applied at the top side of the specimen. Displacements are monitored throughout this stage of the test. Once the axial stress has stabilised, the mechanical loading phase of the test can start.

The loading is performed in steps, starting from the axial swelling stress attained at the end of the resaturation phase. The incremental loading steps are performed by doubling the axial stress reached during the previous step. Each step lasted at least 24 hours. A few doubling steps (depending on the swelling stress) are performed to achieve axial stress of 10 – 15 MPa. For selected tests, a constant head permeability test is performed at this stage, increasing the pore pressure at the bottom side and measuring the fluid flow through the sample. The axial stress and pressure gradient are kept constant during the permeability test until steady-state conditions are achieved. Then, the bottom fluid pressure is reduced back to 50 kPa. Enough time is given for the displacement to stabilise, a minimum of 24 hours. An unloading/reloading phase follows. The specimen is then loaded in steps, up to a maximum stress of 60 MPa. Finally, an unloading phase is performed in steps to achieve axial stress of approximately 7 MPa.

After completion of the test, the oedometer cell is dismantled, and the specimen is dried in the oven to measure the water content.

4.3.4 Complementary tests

In this section, complementary tests performed within the context of the geomechanical testing campaign are described: one-dimensional swelling (OS) and constant head permeability tests (PERM).

Specimens for one-dimensional swelling tests are placed in the oedometric cell. The top cap of the cell is mounted, but no axial stress is applied, rather than the weight itself of the cap. In case the top cap weight was not sufficient to counterbalance the pore pressure applied, an equivalent load was allowed on top of the piston. The specimen is put in contact with pore fluid at low pressure (< 50 kPa) to saturate the specimen. In the direction perpendicular to the cylinder axis, the expansion is constrained, while in the direction parallel to the cylinder axis, expansion is allowed.

The axial displacement is continuously recorded throughout the test. When the axial expansion stabilises, the top of the specimen is connected to the pore fluid reservoir, and further expansion is evaluated. When deformation stabilises, the test is considered concluded. After completion of the test, the oedometer cell is dismantled, and the specimen is dried in the oven to measure the water content.

Constant-head permeability tests were performed on 6 of the specimens tested at predefined stages during one-dimensional compression tests, at constant vertical total stress. In addition, constant-head permeability tests were performed on 2 specimens, in the triaxial condition (constant radial and axial stress).

A pore pressure gradient between top and bottom side of the specimen was applied via two independent Pressure/Volume controllers, and the volumes of water flowing in and out of the specimen were measured. Once the flow reached a stable value (after a few days), this was used to compute the hydraulic conductivity adopting Darcy's law. After completion of the permeability phase, the oedometric test continued as described in the previous section.

4.4 Lithostratigraphy and test overview

4.4.1 Lithostratigraphy of tested core material

A brief summary is provided here on lithological characteristics of the formations of which cores were used in geomechanical testing. Further details on lithology can be found in Dossier III (TBO Bülach-1-1: Lithostratigraphy).

- **«Felsenkalke» / «Massenkalk»:** Bedded (Felsenkalke) to poorly or non-bedded limestones (Massenkalk). In the borehole BUL1-1, the transition is often unclear, repetitive and gradual.
- **Schwarzbach Formation:** Calcareous to argillaceous marls.
- **Villigen Formation:** Mostly calcareous, micritic limestones.
- **Wildegge Formation:** Calcareous marls. In the upper part, silty to fine, sandy marls, siltstones and fine-grained sandstones.
- **«Herrenwis Unit»:** A sedimentary succession consisting mainly of coral bioherms, reef detritus and crinoidal limestone. The upper section is characterised by coral bioherms. In the lower section, encrinite (arenitic limestone made up of echinoderm fragments, "Echinodermenspatkalk") prevails.
- **Wedelsandstein Formation:** Predominantly silty claystones to limestones. It can be subdivided into two shallowing upward cycles: an upper, more argillaceous cycle (855.53 to 869.70 m) can be distinguished from a lower one with siltstone and silty marls (871.50 to 883.30 m). They are separated by a more calcareous section with an erosional surface at the top (869.70 to 870.76 m).
- **Opalinus Clay:** Argillaceous siltstones and marls to claystones. The following subdivision was specified:
 - **891.70 – 912.55 m Sub-Unit 1: «Sub-unit with silty calcareous beds» (20.85 m):** Argillaceous siltstone (calcareous), in the upper part to silty to fine sandy claystone (calcareous) in the lower part. Both medium to dark grey, micaceous, with pyrite. From 904.2 to 912.6 m three, up to several decimetre thick calcareous sections in the form of nodular horizons or hardgrounds are characteristic.

- **912.55 – 927.77 m Sub-Unit 2: «Upper silty sub-unit» (15.22 m):** Monotonous succession of silty to fine sandy claystones (calcareous/ siderolithic) to silty to fine sandy marls, medium grey, with small lenses of siltstones to sandstones (fine-grained). The fine lamination is wavy due to bioturbation in some places.
- **927.77 – 978.80 m Sub-Unit 3: «Mixed clay-silt-carbonate sub-unit» (51.03 m):** Silty claystones dark grey to medium grey, micaceous, with few pyrite nodules, and thin lenses and lenticular nodules of siltstone. From 943 to 954 m significantly increased clay mineral content.
- **978.80 – 995.85 m Sub-Unit 4: «Clay-rich sub-unit» (17.05 m):** Claystones to silty claystones medium to dark grey, micaceous, with pyrite, laminated, very fossiliferous. Strongly bioturbated in the lower part.
- **Staffellegg Formation:** Heterogeneous succession, dominated by marl and claystone, some bituminous, and including various thin limestone horizons, some of them iron-oolitic. The tested interval at approximately 1'001 m belongs to the Rietheim Member and consists of argillaceous to silty and calcareous marl with abundant biotritus and pyrite. The tested interval at 1'016 m belongs to the Frick Members and shows affinities to the lowest part of the Opalinus Clay except for the ammonite fossils.
- **Klettgau Formation:** Heterogeneous succession of carbonates, marls and sandstones. The tested intervals at approximately 1'055 m belong to the Ergolz Member which consist of dolomitic marls and sandstones of channel and overbank sediments.
- **Bänkerjoch Formation:** Succession of anhydrite and marls or claystones.
- **Schinznach Formation:** Succession of dolomites and dolostones, partly dolomitised limestones and micritic limestones, and bituminous claystones.
- **Zeglingen Formation:** Alternation of thin layers (mm to a few cm) of anhydrites (argillaceous), claystone (anhydritic), dark grey, and dolostones (argillaceous), beige grey. Intercalated anhydrite with chicken-wire texture and a predominantly massive anhydrite section. The fine-layered sediment is deformed at several levels (e.g. box folds and chaotic "cauliflower" folding, roof like "folding" and thrusting).
- **Weitenau Formation:** Siliciclastic pre-Mesozoic sediments. Fanglomeratic cycles in general normally graded upward. Includes polymict breccias, argillaceous sandstones, fine-grained conglomerates and sandstones (coarse- to fine-grained), partly with pebbles.

4.4.2 Test overview

A total of 177 rock mechanical and geomechanical tests were conducted on a total of 39 cores from borehole BUL1-1 (Tab. 4-2). The number of tests in the geomechanical testing programme is smaller than in the rock mechanical testing programme. However, typical testing times in the geomechanical programme were 12 to 30 days for TRX+ tests and 20 to 35 days for OED+ tests. That is one order of magnitude longer than the typical TRX test in the rock mechanical testing programme.

Cores without proper initial conditioning (PVC = 0 in Tab. 4-2) were generally only used for lithologies not sensitive to water exchange, i.e. without any macroscopic indication of damage at the time of sampling in the storage facility.

Tab. 4-2: Overview of performed tests

The red border illustrates the clay-mineral rich interval of Opalinus Clay and confining units (focus of geomechanical testing programme). PVC indicates whether cores were conditioned in resin and core barrel on site (1) or later wrapped in aluminium foil off-site (0). TRX = triaxial test, UCS = unconfined compressive strength test, TS = indirect tensile strength test (Brazilian), FT = fracture toughness (semi-circular bend test), TRX+ = TRX with specialised protocols, OED+ = Oedometric test with specialised protocols (high-pressure); PERM = constant-head tests in OED+ tests (^a = exception in triaxial configuration), OS = one-dimensional swelling test.

Core-ID	Formation	Top	Length	Bottom	PVC	Rock mechanical testing programme				Geomechanical testing programme			
		[m]	[cm]	[m]		Standard rock mech. tests				Special protocols		Other	
						TRX	UCS	TS	FT	TRX+	OED+	PERM	OS
BUL1-1-558.14/106-GM	«Felsenkalke» / «Massenkalk»	558.14	106	559.20	0				8				
BUL1-1-559.20/50-GM		559.20	50	559.70	1	3	2	3					
BUL1-1-565.11/34-GM		565.11	34	565.45	1				4				
BUL1-1-577.94/57-GM		577.94	57	578.51	0				4				
BUL1-1-587.47/43-GM		587.47	43	587.90	1		2	2					
BUL1-1-642.34/40-GM		642.34	40	642.74	1		2	2					
BUL1-1-661.10/50-GM	Schwarzbach Fm.	661.10	50	661.60	1		2	2					
BUL1-1-665.18/40-GM		665.18	40	665.58	1	3	2	2					
BUL1-1-666.00/100-GM		666.00	100	667.00	0				6				
BUL1-1-668.00/100-GM		668.00	100	669.00	0				4				
BUL1-1-689.35/45-GM	Villigen Fm.	689.40	40	689.80	0		2	2					
BUL1-1-716.25/50-GM		716.25	50	716.75	0		3	1					
BUL1-1-735.00/50-GM		735.00	50	735.50	0		2	1					
BUL1-1-753.31/50-GM	Wildeggen Fm.	753.31	50	753.81	1				5				
BUL1-1-761.62/42-GM		761.62	42	762.04	1	3	2	2					
BUL1-1-816.60/50-GM	«Herrenwis Unit»	816.60	50	817.10	1	3	2	2					
BUL1-1-827.66/35-GM		827.66	35	828.01	1							2 ^a	
BUL1-1-846.47/49-GM		846.47	49	846.96	0	3	2	3					
BUL1-1-855.58/49 GM	Wedelsandstein Fm.	855.58	49	856.07	1					3			
BUL1-1-857.39/49 GM		857.39	49	857.88	1					2	2	1	
BUL1-1-859.28/31 GM		859.28	31	859.59	1						2		
BUL1-1-878.60/50 GM		878.60	50	879.10	1					2			
BUL1-1-892.50/38 GM	Opalinus Clay	892.50	50	893.00	1					4	1	1	
BUL1-1-896.45/49 GM		896.45	49	896.94	1					7			
BUL1-1-915.18/51-GM		915.18	51	915.69	1					8			
BUL1-1-935.84/39-GM		935.84	39	936.23	1						4	2	1
BUL1-1-940.89/43-GM		940.89	43	941.32	1					2			
BUL1-1-947.95/39-GM		947.95	39	948.34	1					3	1		
BUL1-1-950.55/40-GM		950.55	40	950.95	1					6			
BUL1-1-970.56/37-GM		970.56	37	970.93	1					2	1	1	4
BUL1-1-1001.14/43-GM	Staffellegg Fm.	1'001.14	43	1'001.57	1							1	2
BUL1-1-1015.77/50-GM		1'015.77	50	1'016.27	1					1	1		
BUL1-1B-1054.87/50-GM	Klettgau Fm.	1'054.87	50	1'055.37	1		2	1					
BUL1-1B-1099.77/33-GM	Bänkerjoch Fm.	1'099.77	33	1'100.10	1		3	1					
BUL1-1B-1172.02/39-GM	Schinznach Fm.	1'172.02	39	1'172.41	1		3	1					
BUL1-1B-1194.46/50-GM		1'194.46	50	1'194.96	1	1	2	1					
BUL1-1B-1243.15/31-GM	Zeglingen Fm.	1'243.15	31	1'243.46	0		2	2					
BUL1-1B-1329.00/50-GM	Weitenau Fm.	1'329.00	50	1'329.50	0		3	2					
BUL1-1B-1329.72/44-GM		1'329.72	44	1'330.16	0	1	1	2					
Total					28	17	39	32	31	40	12	8	7

5 Rock mechanical testing

5.1 Fracture toughness using semi-circular bend specimen

The fracture toughness is obtained from the peak load P_{max} recorded during such a semi-circular bending test:

$$K_{Ic} = P_{max} \frac{\sqrt{\pi a}}{2 R B} Y' \quad (5-1)$$

Y' is a dimensionless factor which depends on the distance between the two supports (s). It is defined as

$$Y' = -1.297 + 9.516 \left(\frac{s}{2R} \right) - \left(0.47 + 16.457 \frac{s}{2R} \right) \beta + \left(1.071 + 34.401 \frac{s}{2R} \right) \beta^2 \quad (5-2)$$

with $\beta = a/R$.

A typical test result is illustrated in Fig. 5-1.



Fig. 5-1: Example of test specimen after testing
The crack does not follow a parallel plane to the notch.

Tests were performed only in the Malm section at three distinct depth intervals.

Tab. 5-1: Test results from semi-circular bend tests

Last column depicts standard deviation

Formation	N samples	Depth range [m]	K _{Ic} [MPa.m ^{1/2}]	σ [MPa.m ^{1/2}]
«Felsenkalk» / «Massenkalk»	16	558 – 578	1.02	0.2
Villigen Formation	8	666 – 668	0.51	0.3
Wildegge Formation	3	753 – 754	0.38	0.1

5.2 Indirect tensile strength test (Brazilian)

A total of 32 Brazilian tests were performed (Tab. 5-2). The splitting tensile strength (MPa) is calculated according to the following formula (curved jaws):

$$s_z = \frac{1272 * F}{\pi * d * l} \quad (5-3)$$

with the compressive force F (in kN); the specimen diameter d (in mm) and the specimen length l (in mm). It is noted that the values of s_z are negative.

Tab. 5-2: Test results from indirect tensile strength tests

Test-ID	Specimen geometry	Average depth	Bulk density	Water content	Tensile strength
		[m]	[g/cm ³]	[wt.-%]	[MPa]
BUL1-1_559TS_S1	S	559.22	2.70	0.33	-12.00
BUL1-1_559TS_P1	P	559.27	2.70	0.29	-6.96
BUL1-1_559TS_P2	P	559.32	2.67	0.90	-3.50
BUL1-1_587TS_S1	S	587.62	2.57	2.33	-3.10
BUL1-1_587TS_P1	P	587.67	2.56	2.17	-5.13
BUL1-1_642TS_S1	S	642.53	2.70	0.22	-6.55
BUL1-1_642TS_P1	P	642.55	2.69	0.21	-7.15
BUL1-1_661TS_S1	S	661.22	2.64	1.79	-2.94
BUL1-1_661TS_P1	P	661.29	2.66	1.16	-3.67
BUL1-1_665TS_S1	S	665.24	2.65	1.66	-3.14
BUL1-1_665TS_P1	P	665.30	2.65	1.64	-2.42
BUL1-1_689TS_S1	S	689.67	2.69	0.10	-10.25
BUL1-1_689TS_P1	P	689.74	2.69	0.13	-7.34
BUL1-1-716TS_S1	S	716.32	2.67	0.21	-9.43
BUL1-1-735TS_S1	S	735.21	2.63	0.82	-8.52
BUL1-1_761TS_S1	S	761.67	2.62	2.14	-2.45
BUL1-1_761TS_P1	P	761.72	2.60	2.60	-2.46
BUL1-1_816TS_P	P	816.73	2.64	0.72	-5.28
BUL1-1_816TS_S	S	816.90	2.67	0.63	-4.01
BUL1-1_846TS_S1	S	846.59	2.69	0.23	-4.38
BUL1-1_846TS_S2	S	846.64	2.68	0.12	-5.67
BUL1-1_846TS_P1	P	846.75	2.68	0.23	-6.55
BUL1-1B_1054TS_S	S	1'055.00	2.40	4.55	-2.07
BUL1-1B_1099TS_S	S	1'099.99	2.81	0.86	-5.82
BUL1-1B_1172TS_S	S	1'172.13	2.62	2.41	-5.43
BUL1-1B_1194TS_S	S	1'194.84	2.69	0.12	-5.42
BUL1-1B-1243TS_S1	S	1'243.29	2.92	0.09	-8.04
BUL1-1B-1243TS_P1	P	1'243.36	2.93	0.09	-8.27
BUL1-1B-1329TS_S1	S	1'329.78	2.42	1.49	-2.33
BUL1-1B-1329TS_S2	S	1'330.08	2.46	0.94	-4.56
BUL1-1B-1331TS_S1	S	1'331.58	2.43	1.59	-3.26
BUL1-1B-1331TS_P1	P	1'331.65	2.46	1.30	-3.84

The test results yielded values from -12.00 MPa to -2.07 MPa for S-samples and -8.27 MPa to -2.42 MPa for P-samples. A graphical representation of the test results is provided for each formation and grouped by sample geometry in Fig. 5-4. Photographs of the specimens before and after testing can be found in Appendix A.

5.3 Unconfined compressive strength tests

A total of 39 UCS tests were performed (Tab. 5-3). A graphical representation of the test results is provided for each formation and grouped by sample geometry in Fig. 5-4. Photographs of the specimens before and after testing can be found in Appendix A.

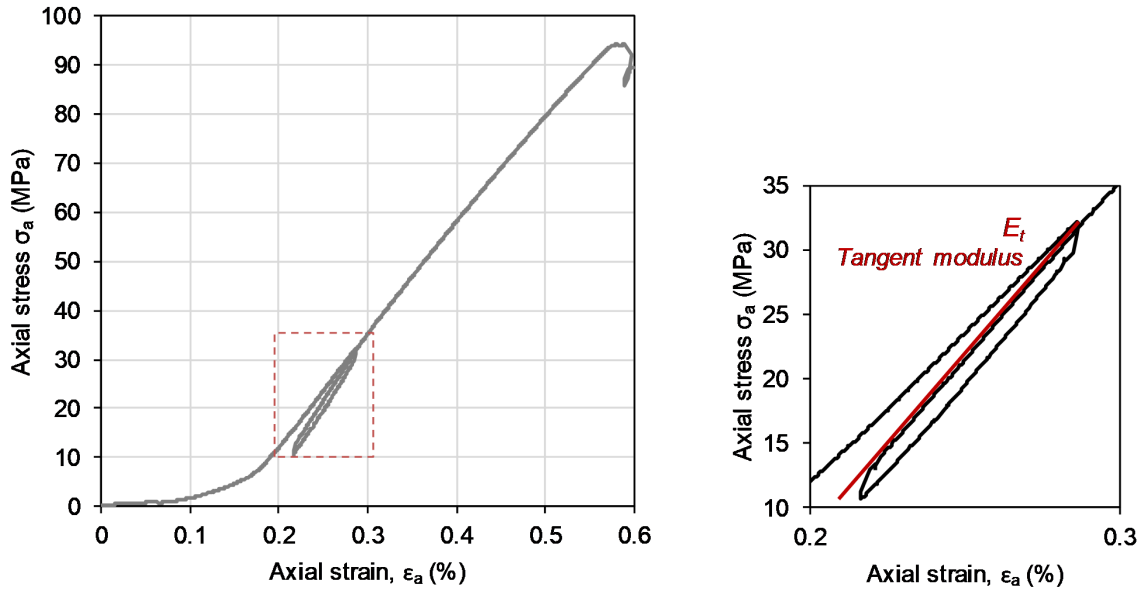


Fig. 5-2: Stress strain diagram for UCS test

Example from test BUL1-1_661UCS_S1, on the left zoom into the unload reload loop, and definition of tangent and secant Young's moduli.

At the linear portion of the axial load cycle, an unload-reload cycle was performed by reducing the axial stress to 1/3 of the actual load, and hysteresis was observed. The cycle was performed for determining the elastic properties of the rock. The Young's modulus E (in GPa) was determined as an average reload modulus using the slope of the rising portion of the hysteresis as the ratio between the axial load change $\Delta\sigma_a$ (in MPa) and the induced axial strain $\Delta\epsilon_a$ (dimensionless) according to the equation:

$$E = \frac{\Delta\sigma_a}{\Delta\epsilon_a} \quad (5-4)$$

The Poisson's ratio (ν) is calculated as the negative ratio between the delta in radial $\Delta\epsilon_r$ and axial strain $\Delta\epsilon_a$:

$$\nu = -\frac{\Delta\epsilon_r}{\Delta\epsilon_a} \quad (5-5)$$

An example of calculation of tangent E modulus is reported in Fig. 5-2.

Following the hysteresis, the axial load was again increased until peak strength was reached, and the sample was allowed to fail. The compressive peak strength was derived from the peak value of the axial load.

Tab. 5-3: Test results from UCS test

^a Only first load modulus, no unload-reload loop performed.

Test-ID	Specimen geometry	Average depth	Bulk density	Water content	Young's modulus	Poisson's ratio	Peak strength	Strain at peak strength
		[m]	[g/cm ³]	[wt.-%]	[GPa]	[-]	[MPa]	[%]
BUL1-1_559UCS_S1	S	559.27	2.69	0.27	32.92	0.13	223.7	0.75
BUL1-1_559UCS_P1	P	559.27	2.70	1.77	39.48	0.16	111.9	0.37
BUL1-1_587UCS_S1	S	587.70	2.55	2.45	11.93 ^a	0.30 ^a	24.3	0.26
BUL1-1_587UCS_P1	P	587.78	2.54	2.51	26.38	0.15	58.1	0.37
BUL1-1_642UCS_S1	S	642.60	2.69	0.30	32.95	0.15	213.6	0.75
BUL1-1_642UCS_P1	P	642.61	2.68	0.56	28.98	0.10	142.4	0.55
BUL1-1_661UCS_P1	P	661.34	2.67	1.15	19.92	0.15	119.2	0.59
BUL1-1_661UCS_S1	S	661.34	2.67	4.23	27.96	0.20	94.3	0.59
BUL1-1_665UCS_S1	S	665.28	2.65	1.72	20.28	0.21	84.7	0.74
BUL1-1_665UCS_P1	P	665.30	2.65	1.65	26.16	0.25	106.2	0.60
BUL1-1_689UCS_P1	P	689.51	2.70	0.19	44.27	0.19	216.8	0.53
BUL1-1_689UCS_S1	S	689.55	2.69	0.18	46.84	0.22	158.8	0.43
BUL1-1-716UCS_S1	S	716.39	2.68	0.17	43.63	0.20	209.5	0.59
BUL1-1-716UCS_S2	S	716.48	2.68	0.26	40.54	0.21	210.6	0.71
BUL1-1-716UCS_S3	S	716.64	2.64	1.62	25.60	0.18	114.0	0.74
BUL1-1-735UCS_S1	S	735.12	2.64	0.85	26.61	0.21	133.4	0.69
BUL1-1-735UCS_S2	S	735.39	2.61	1.24	19.55	0.19	106.1	0.83
BUL1-1_761UCS_P1	P	761.67	2.62	2.30	20.74	0.26	67.5	0.50
BUL1-1_761UCS_S1	S	761.84	2.59	3.17	11.43	0.31	48.5	0.90
BUL1-1_816UCS_P	P	816.64	2.67	0.74	9.77	0.36	14.8	0.32
BUL1-1_816UCS_S	S	816.85	2.70	0.23	17.07	0.04	48.0	0.28
BUL1-1_846UCS_S1	S	846.79	2.67	0.32	13.74	0.06	42.2	0.48
BUL1-1_846UCS_S2	S	846.85	2.68	0.11	26.51	0.14	111.1	0.56
BUL1-1B_1054UCS_S	S	1'054.91	2.44	4.28	4.02	0.10	30.1	0.81
BUL1-1B_1054UCS_P	P	1'055.00	2.41	4.34	6.45	0.14	29.1	0.53
BUL1-1B_1099UCS_P1	P	1'099.79	2.68	2.31	19.86	0.13	48.7	0.26
BUL1-1B_1099UCS_P2	P	1'099.85	2.80	1.12	11.20	0.10	33.2	0.34
BUL1-1B_1172UCS_S1	S	1'172.10	2.61	3.37	14.08	0.07	117.8	0.50
BUL1-1B_1172UCS_P	P	1'172.22	2.65	2.38	23.70	0.08	121.0	0.49
BUL1-1B_1172UCS_S2	S	1'172.27	2.61	2.19	19.32	0.81	68.6	0.40
BUL1-1B_1194UCS_S	S	1'194.54	2.69	0.13	32.65	0.13	106.3	0.36
BUL1-1B_1194UCS_P	P	1'194.56	2.70	0.08	30.35	0.13	116.6	0.47
BUL1-1B-1243UCS_S1	S	1'243.23	2.94	0.06	51.56	0.18	155.4	0.45
BUL1-1B-1243UCS_P1	P	1'243.41	2.94	0.08	48.75	0.19	154.5	0.44
BUL1-1B-1329UCS_S1	S	1'329.83	2.36	1.63	10.90	0.18	50.2	0.84
BUL1-1B-1329UCS_P1	P	1'329.84	2.36	1.44	11.53	0.22	39.4	0.63
BUL1-1B-1329UCS_S2	S	1'329.99	2.44	1.38	10.54	0.11	60.9	0.86
BUL1-1B-1331UCS_S1	S	1'331.51	2.45	1.34	14.87	0.18	60.1	0.87

The relatively large lithologic variability of tested specimens is reflected by the test results, ranging just over one order of magnitude both for Young's moduli and strength.

Notable is test 587UCS_S1, which failed at very low deviatoric stress (24 MPa) and no unload – reload loop could be performed. Elastic moduli for this test were therefore estimated from first loading.

5.4 Triaxial deformation tests

A total of 17 triaxial tests were performed. A graphical representation of the test results is provided for each formation and grouped by sample geometry in Fig. 5-4. Photographs of the specimens before and after testing can be found in Appendix A.

Basic properties and test conditions are provided in Tab. 5-4. With some exceptions, tests were conducted undrained with an initial fluid pressure of 5 MPa. The exceptions were one test (1243TRX_10S1, anhydrite) where no pore fluid pressure was applied, and two tests with significantly larger pore pressure (12 MPa) in an attempt to allow monitoring of fluid pressure in undrained conditions where large dilation occurs (see below).

A consolidation phase was performed for tests 665TRX_10S1 and 761TRX_10S1 to constrain the consolidation coefficient, which was used to examine the appropriate strain rate. In the same tests and additionally a third (559TRX_10S1) Skempton's B values were also constrained³. Confining pressure in all tests was either 10 or 15 MPa, except for 665TRX_10S1 (11.2 MPa). In two tests, the confining pressure was decreased during the test (see below). Ultrasonic velocities were recorded throughout the tests, and the tabulated values in Tab. 5-4 were constrained at the start of the shear phase. Signals were either too weak (761TRX_10S1) or unclear for the S-wave velocity (761TRX_15P1) in some tests. The range of values of confining stress and pore pressure were chosen to reflect approximately expected in situ state conditions, but at the same time it was decided not to change the values proportionally to depth of extraction to enable comparisons of test results between lithologies.

³ The tabulated values in Tab. 5-4 are not corrected for potential equipment compliance.

Tab. 5-4: Basic properties and triaxial test results from consolidation and saturation stage

^a Initial value before unloading (TC stress paths, cf. Tab. 5-5).

Test-ID	Specimen geometry	Average depth	Bulk density	Water content	Pore pressure	Confining pressure	Consolidation coefficient c_v	Skempton's B	P-wave velocity	S-wave velocity
		[m]	[g/cm ³]	[wt.-%]	[MPa]	[MPa]	[mm ² /h]	[-]	[m/s]	[m/s]
559TRX_10S1	S	559.41	2.69	1.02	9.5	10.0	-	0.65	6122	3096
559TRX_10P1	P	559.46	2.69	1.24	9.5	10.0	-	-	6150	3066
559TRX_15P1	P	559.46	2.69	1.40	14.0	15.0	-	-	6238	3162
665TRX_10P1	P	665.38	2.67	2.18	5.0	10.0	-	-	5081	2658
665TRX_10S1	S	665.40	2.66	2.32	6.2	11.2	500	0.49	4784	2559
665TRX_15P1	P	665.42	2.66	2.52	5.0	15.0	-	-	4947	2622
761TRX_10S1	S	761.84	2.59	4.08	5.0	10.0	80	0.79	-	-
761TRX_15P1	P	761.86	2.58	3.60	5.0	10.0	-	-	4167	-
816TRX_10P1	P	816.69	2.70	1.22	5.0	10.0	-	-	6190	3071
816TRX_10P2	P	816.69	2.66	1.73	12.0	13.0 ^a	-	-	5865	3020
816TRX_10S1	P	816.82	2.69	1.46	5.0	10.0	-	-	5679	2615
846TRX_15S1	S	846.85	2.70	1.19	5.0	15.0	-	-	6021	3010
846TRX_10P1	P	846.92	2.66	1.81	5.0	10.0	-	-	5779	2957
846TRX_15P2	P	846.92	2.66	1.51	12.0	13.0 ^a	-	-	5958	3014
1243TRX_10S1	S	1'243.23	2.93	0.07	0.0 ^b	10.0	-	-	6221	3292
1331TRX_15S1	S	1'331.51	2.43	3.29	5.0	15.0	-	-	3962	2289

Undrained testing in several tests proved challenging. In some tests the positive pore pressure build-up during the loading phase was significant. To avoid pore pressure exceeding the confining pressure, drained testing conditions were enabled when the pore pressure reached a level of 0.5 MPa below the confining pressure (Fig. 5-3, left side). This was implemented in four tests (Tab. 5-5). In other cases, the opposite effect occurred, and dilation was so strong that pore pressure was essentially consumed, i.e. Δu_w became negative and eventually matched the initial pore pressure of 5 MPa (Fig. 5-3, right side). In such cases the effective stress at failure is not really clear, although the test evolution suggests that peak strength coincides with the strain at which absolute pore pressure is essentially zero.

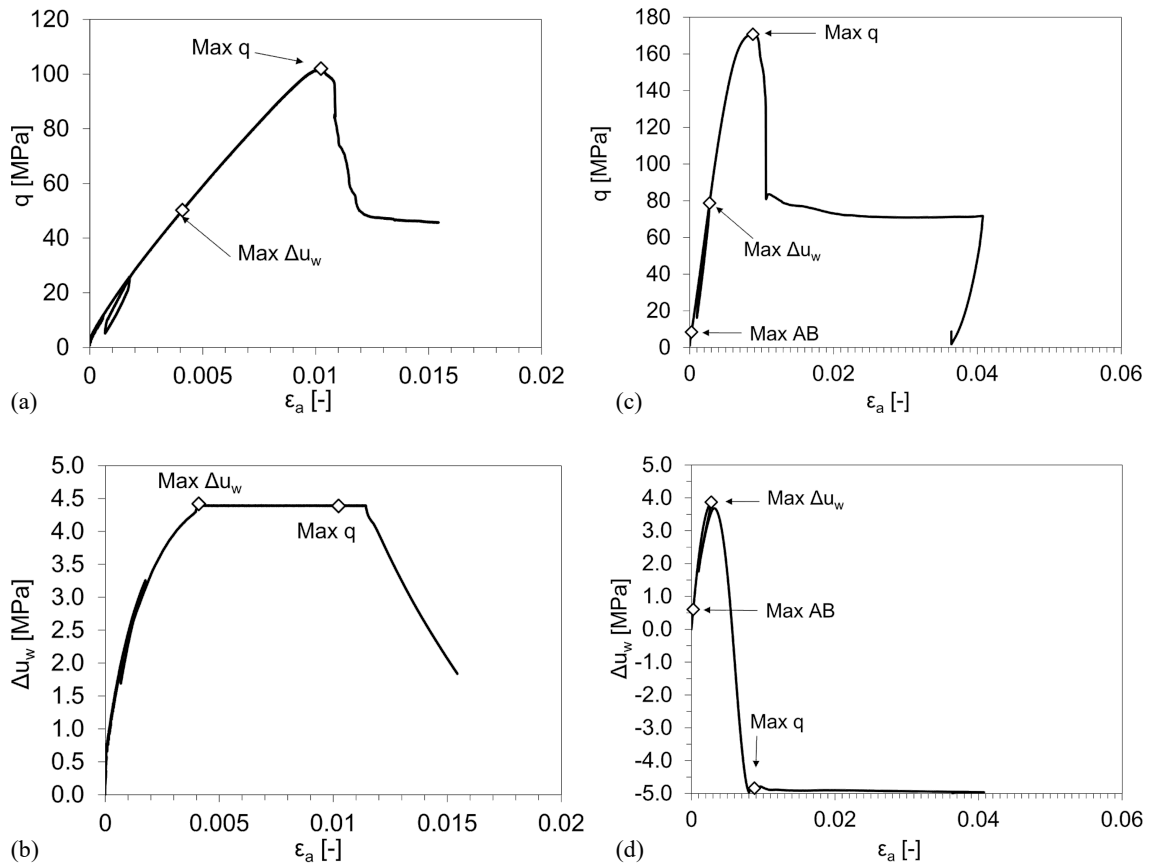


Fig. 5-3: Examples of problematic pore pressure evolution during undrained testing

(a), (b) Example test 665TRX_10S1, with deviatoric stress (q , (a)) and generated pore fluid pressure (Δu_w , (b)). Pore fluid pressure was held constant at 4.4 MPa to avoid fluid pressure exceeding the confining pressure. (c), (d) Example test 846TRX_15S1, large pore pressure drop due to strong dilation in undrained conditions.

Tab. 5-5: Triaxial test results from the shear phase

^aPore fluid pressure was kept 0.5 MPa below confining pressure. ^bPore pressure close to zero due to dilation. Peak strength refers to maximum deviatoric stress according to equation 4-2.

Test-ID	Stress path	Strain rate	Young's modulus	Poisson's ratio	Peak strength	Radial stress at peak	Pore pressure at peak	Strain at peak strength	Post-peak strength	Pore pressure at post-peak
		[1/s]	[GPa]	[-]	[MPa]	[MPa]	[MPa]	[%]	[MPa]	[MPa]
559TRX_10S1	CTC	1E-06	28.6	0.17	263.6	10.0	9.5 ^a	0.91	77.1	0.7
559TRX_10P1	CTC	1E-06	31.2	0.19	268.1	10.0	9.5 ^a	1.08	59.4	4.0
559TRX_15P1	CTC	1E-06	37.8	0.19	188.7	15.0	14.0 ^a	0.85	108.5	6.0
665TRX_10P1	CTC	1E-06	19.9	0.25	111.9	10.0	4.8	0.80	47.2	0.9
665TRX_10S1	CTC	1E-06	15.7	0.39	102.0	11.2	10.7 ^a	1.02	43.3	4.8
665TRX_15P1	CTC	1E-06	22.0	0.37	110.2	15.0	7.8	0.78	57.2	3.7
761TRX_10S1	CTC	1E-07	3.8	0.52	20.1	10.0	8.9	0.66	15.5	4.6
761TRX_15P1	CTC	1E-07	11.6	0.52	35.6	10.0	6.3	0.36	27.7	0.7
816TRX_10P1	CTC	1E-06	34.9	0.31	156.6 ^b	10.0	0.6	0.72	51.4	0.4
816TRX_10P2	TC	1E-06	-	-	65.4 ^b	3.2	0.5	0.48	50.6	0.0
816TRX_10S1	CTC	1E-06	24.7	0.32	109.8 ^b	10.0	0.4	0.78	59.9	0.1
846TRX_15S1	CTC	1E-06	33.1	0.25	170.7 ^b	15.0	0.2	0.90	71.1	0.1
846TRX_10P1	CTC	1E-06	28.3	0.40	143.8 ^b	10.0	0.4	0.83	59.8	0.1
846TRX_15P2	TC	1E-06	-	-	96.9 ^b	1.9	0.4	0.42	61.4	0.3
1243TRX_10S1	CTC	1E-05	42.5	0.21	183.4	10.0	0.0	0.68	80.6	0.0
1331TRX_15S1	CTC	1E-06	12.5	0.17	84.9	15.0	1.2	1.26	41.3	0.2

6 Geomechanical testing programme

In the BUL1-1 geomechanical testing campaign, 40 triaxial tests were performed. Three geometries (Section 5.1) were selected: S, P and Z samples. Z samples were prepared in order to obtain sample shearing along the bedding direction, i.e. the direction with potentially the minimum strength. The angle to bedding was decided based on the results from the previous testing campaign (Minardi et al. 2019) and first results on BUL1-1 testing campaign. 5 Z-samples with an angle of 60° between the bedding direction and the horizontal plane (i.e., 30° between bedding direction and the cylinder axis) were prepared.

Of the 40 triaxial tests, 38 were conducted at room temperature and 2 tests (S-samples) included a shearing phase at a temperature of 60° C. This temperature was selected as it is an upper boundary of expected in-situ temperatures of Opalinus Clay at the relevant potential repository depths. It is also consistent with the estimated temperature of 50° C constrained in the borehole BUL1-1. The results of the shearing phases at high temperature were compared to results on identical tests (extracted from the same core depth) performed at room temperature to evaluate the possible impact of temperature on the material response.

Seven triaxial tests were subjected to *hybrid* drainage conditions: for these tests, an initial load/unload loop was performed in order to constrain reversible response of the sample in drained (or undrained) condition; then, the boundary conditions were changed to drained (or undrained) and shearing continued until constant post-peak deviatoric stress was achieved. This technique allows to constrain both drained and undrained reversible response on one sample, with a considerable gain in terms of time, use of material, and comparability of the results.

In addition, two constant-head permeability tests were performed in triaxial conditions, while 12 one-dimensional compression, 7 one-dimensional swelling and 6 constant-head permeability tests were performed in oedometric conditions.

In the following sections, the test results of the experimental campaign are presented. First, the main geotechnical and intrinsic properties are reported (Section 6.1). Then, in Sections 6.2 and 6.3, the results of tests in triaxial and oedometric conditions are reported.

6.1 Geotechnical and intrinsic properties of tested cores

6.1.1 Water content and native activity

The laboratories involved in the testing campaign performed characterisation of the tested specimens. The water content of the specimens before and after testing was determined, and the results are reported in Fig. 6-1.

For the triaxial tests, the first phase of each test consisted in imposing saturated conditions to the sample, either (i) by injecting water in isochoric conditions, measuring the generated swelling pressure (conventional testing procedure), or (ii) by isotropically compressing the sample until positive pore pressure is developed (alternative testing procedure).

When the first procedure is used, the water content before and after the tests generally differs, and the difference may primarily be attributed to re-saturation in the rig. This is the case for Lab A and B results. Using the second procedure (lab C), no fluid is added to the sample. Therefore, the water content before and after the test is expected to be the same. Volumetric deformation during testing, when drained conditions are allowed, also alters the water content of the sample. Addi-

tional sources of discrepancy between the water content before and after tests can be attributed to the handling of the sample when dismantling the test (e.g., loss of solid mass while removing the sample from the sleeve).

The water content and the Native Activity of the material were also measured on trimming pieces from a part of the section from which testing specimens were extracted. The Native Activity of a specimen corresponds to the value of relative humidity in equilibrium with the specimen native state, with no variation of the initial water content⁴.

The measurements were done by placing a piece of freshly cut material within a closed container and measuring the resulting (relative) humidity generated within the container (Ewy 2015) by the use of psychrometer. While Lab A and C used intact pieces, Lab B crushed the freshly cut pieces in smaller parts before taking the measurement.

Results are reported in Fig. 6-2. All the samples present high Native Activity, above 70 %, with peaks at 85 – 90 %, showing excellent preservation conditions of the cores. Lab A measurements showed systematically lower Native Activity than Lab B, which may be attributed to the use for the measurement of trimming from the rig of the core, more exposed to environmental changes, and to moisture loss.

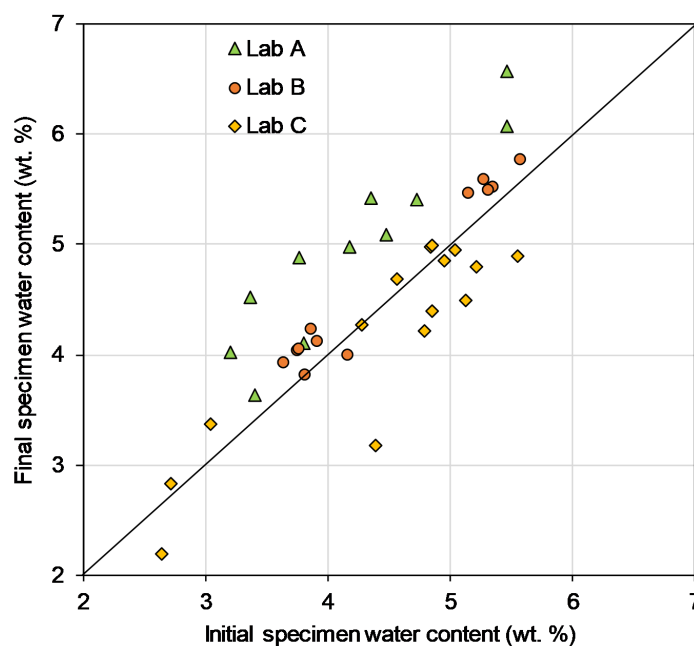


Fig. 6-1: Initial versus final water content of triaxial specimens

⁴ In a specimen in contact with controlled relative humidity, if water content increases, then that relative humidity value is greater than the Native Activity value of the shale. Conversely, if there is a water content decrease, then that relative humidity value is less than the Native Activity.

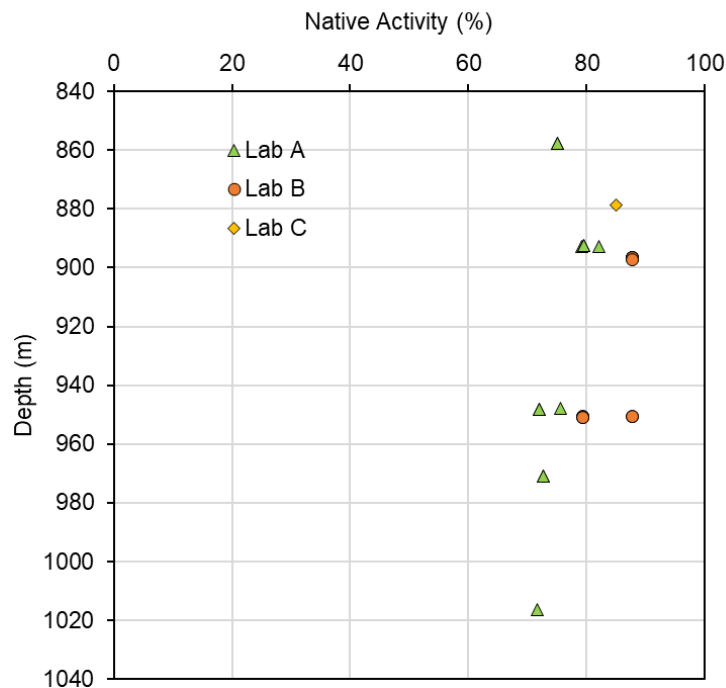


Fig. 6-2: Native activity of trimming versus depth of collection
Results grouped by laboratory.

6.1.2 Atterberg limits, plasticity index, grain size distribution

Lab A and D, in the context of the oedometric testing campaign, performed full geotechnical characterisation of the tested cores. In particular, Atterberg limits, plasticity index and grain size distribution were determined.

Atterberg limits were determined according to ASTM D4318-17 (ASTM 2017a); the material was manually crushed with a rubber-tipped pestle. Only the portion with grain dimension lower than 0.425 mm was used. Lab A used the ASTM One-Point method to determine the liquid limit. The one-point uses only one measurement (with the Casagrande cup), and a correcting factor defined by the standard.

Grain size distributions were determined according to ad-hoc procedures developed by the laboratories, according to Nagra's indications. The main aspects of the two procedures are here summarised:

- Lab D adopted a procedure implemented during previous projects involving Opalinus Clay and which is based on the ASTM D6913-17 (ASTM 2017b) and D7928-17 (ASTM 2017c); a gentle pre-crushing was performed manually with a rubber-tipped pestle, to separate the aggregates avoiding the breakage of individual particles. Before the test the material was submerged in distilled water for 24 hours and sieved using water. Grain size distribution was determined by wet sieving for particles larger than 0.075 mm, and by sedimentation (by the use of a hydrometer) for the smaller particles.
- Lab A measured the grain size distribution using a hydrometer, according to ASTM D7928-17 (ASTM 2017c). A trimming piece ~ 50 g was pestled into smaller bits. The bits were then placed in a beaker with 125 ml of a dispersing agent and left for 3 days. This method was chosen instead of pulverisation to preserve the grains. After 3 days, the material was mixed using a shaker and then transferred into a cylinder, filled up with distilled water. The cylinder

was then turned end-over-end to be mixed again and then covered overnight, to obtain stable temperature. After 24 hours, the cylinder was turned end-over-end 60 times in approximately 1 minute to ensure that the material was thoroughly mixed. The hydrometer was placed in the cylinder, and readings of both hydrometer and temperature were taken in intervals. When all the readings were performed, the material was wet-sieved with 75 μm . Afterwards, the remaining material was dry-sieved in order to determine the grain size of the material $> 75 \mu\text{m}$. The readings from the hydrometer and the sieve-results were then combined in a grain size distribution chart.

Atterberg limits and granulometric fractions are summarised in Tab. 6-1. The results on the Atterberg limits show consistency between the laboratories. Concerning the granulometric fractions, it is noted that the procedure adopted by lab A yielded low clay granulometric fractions, in comparison with the results from lab D. The results from both laboratories show low clay fraction with respect to the clay mineralogical content expected for the tested sections (Appendix D). This is attributed to an only partial disaggregation of the clay aggregates.

Tab. 6-1: Atterberg limits and grain size fractions for the tested cores

Core ID	Lab	Core interval	w_L	w_P	PI	Gravel	Sand	Silt	Clay
		[cm]	[-]	[-]	[-]	[%]	[%]	[%]	[%]
BUL1-1 859.28/31	D	6 – 11	0.40	0.19	0.21	0.1	4.3	69.3	26.3
BUL1-1-935.84/39	D	8– 11	0.30	0.21	0.09	4.5	15.7	56.0	23.8
BUL1-1-935.84/39	D	0 – 4	0.27	0.18	0.09	2.4	8.5	61.8	27.3
BUL1-1-1001.14/43	D	20 – 23	0.26	0.17	0.09	1.9	5.9	64.6	27.6
BUL1-1 892.50/38	A	8 – 10	0.27	0.19	0.08	0	24.0	67.5	8.5
BUL1-1 857.39/49	A	2 – 5	0.29	0.20	0.09	0	38.0	56.0	6.0
BUL1-1 857.39/49	A	19 – 24	0.31	0.20	0.11	0	36.0	57.0	7.0
BUL1-1 1015.77/50	A	34 – 37	0.27	0.18	0.09	0	36.0	41.0	5.0

6.2 Triaxial tests with specialised protocols

An overview of the performed tests is presented in Tab. 6-2. Each test is labelled with an ID indicating the main features of the test.

For example, the triaxial test ID *BUL1-1_896_60S5CTCU_T_B* indicates the borehole name (BUL1-1), the core top depth (896.6 m), the sample geometry (S), effective confinement before shearing (5 MPa), the stress path (CTCU), an indication of test performed at high temperature (T), and of the laboratory that performed the test (B).

It is reported in Tab. 6-2 for each tested sample:

- Sample ID, as above described
- Core depth [m]
- Geological formation, as presented in Section 4.4.1
- Sample geometry
- Total confining stress, p_0 [MPa] at the beginning of the shearing phase,

- Pore pressure [MPa] at the beginning of the shearing phase,
- Effective confining stress p'_0 [MPa] at the beginning of the shearing phase,
- Stress path, as presented in Fig. 4-6
- Indication of shearing conditions for hybrid (H), high temperature (T) tests or shearing phase conducted in drained conditions (d)
- Initial void ratio, computed using a reference solid density of 2.70 g/cm³
- Initial porosity, computed using a reference solid density of 2.70 g/cm³
- Initial and final water content

Tab. 6-2: Conducted triaxial tests

Sample ID, shearing phase characteristics, specimen initial (and final) conditions

Sample ID	Specimen location and geometry			Shearing phase characteristics					Specimen initial conditions			
	Core depth [m]	Formation	Sample Geometry	Total confining Stress, p_0 [MPa]	Pore pressure, u_{w0} [MPa]	Effective confining stress p'_0 [MPa]	Stress Path	Hybrid (H), High temperature (T), Drained shearing (d)	Initial void ratio, e_0^* [-]	Initial porosity, n_0 [-]	Initial water content, w_0 [%]	Final water content, w_f [%]
BUL1_1_855_89S5CTCU_C	855.58	Wedelsandstein	S	14	8.8	5.2	CTCU		0.072	0.067	2.71	2.83
BUL1_1_855_91S11TC_C	855.58		S	20	9	11	TC drained	d	0.076	0.071	3.04	3.37
BUL1_1_855_91S14CTCU_C	855.58		S	22	7.9	14.1	CTCU		0.070	0.066	2.64	2.19
BUL1_1_857_49P5CTCU_A	857.49		P	14	9	5	CTC + CTCU	H	0.106	0.096	3.77	4.88
BUL1_1_857_49S5CTCU_A	857.49		S	14	9	5	CTCU		0.110	0.099	3.98	4.87
BUL1_1_878_67S4CTCU_C	878.6		S	14	9.9	4.1	CTCU		0.126	0.112	5.21	4.79
BUL1_1_878_6S13_CTCU_C	878.6		S	22	9.4	12.6	CTCU		0.117	0.104	4.79	4.22
BUL1_1_892_53S13CTCU_A	892.53		Opalinus Clay	S	22	9	13	CTCU		0.090	0.083	3.40
BUL1_1_892_53S5CTCU_A	892.53	S		14	9	5	CTCU		0.080	0.074	3.21	4.02
BUL1_1_892_71S5CTCU_A	892.71	S		14	9	5	CTCU		0.108	0.097	3.80	4.10
BUL1_1_892_84S13CTCU_A	892.5	S		22	9	13	CTCU		0.123	0.110	4.35	5.42
BUL1_1_896_60S13CTCU_B	896.45	S		22	9	13	CTCU		0.099	0.090	3.91	4.11
BUL1_1_896_60S13CTCUT_B	896.45	S		22	9	13	CTCU	T	0.100	0.091	3.81	3.81
BUL1_1_896_60S5CTCU_B	896.45	S		14	9	5	CTCU		0.095	0.087	3.77	4.05
BUL1_1_896_60S5CTCUT_B	896.45	S		14	9	5	CTCU	T	0.092	0.085	3.65	3.93
BUL1_1_896_66P11CTCU_B	896.45	P		20	9	11	CTCU		0.099	0.090	4.17	3.99
BUL1_1_896_66P5CTCU_B	896.65	P		14	9	5	CTCU		0.096	0.088	3.75	4.04
BUL1_1_896_80S5TCU_B	896.8	S		12	7	5	TCU		0.104	0.094	3.86	4.23
BUL1_1_915_32_P11CTCU_C	915.18	P		20	8.5	11.5	CTC + CTCU	H	0.117	0.105	4.57	4.68
BUL1_1_915_32Z7CTCU_C	915.18	Z		17	9.4	7	CTCU		0.109	0.099	4.85	5.00
BUL1_1_915_33P5CTCU_C	915.18	P		14	9.5	4.5	CTCU		0.113	0.101	4.39	3.18
BUL1_1_915_33S10TC_C	915.18	S		17.9	9	8.9	TC drained	d	0.123	0.110	5.04	4.95
BUL1_1_915_33S12CTCU_C	915.18	S	22	9.6	12.4	CTCU		0.124	0.110	4.96	4.85	

Tab. 6-2: (continued)

Sample ID	Specimen location and geometry		Shearing phase characteristics						Specimen initial conditions			
	Core depth [m]	Formation	Sample Geometry	Total confining Stress, p_0 [MPa]	Pore pressure, u_{s0} (MPa)	Effective confining stress p'_0 [MPa]	Stress Path	Hybrid (H), High temperature (T), Drained shearing (d)	Initial void ratio, e_0^* [-]	Initial porosity, n_0 [-]	Initial water content, w_0 [%]	Final water content, w_f [%]
BUL1_1_915_33S5CTCU_C	915.18	Opalinus Clay (continued)	S	13.9	8.7	5.2	CTCU + CTC	H,d	0.118	0.106	4.84	4.97
BUL1_1_915_38Z13CTCU_C	915.18		Z	22	9.4	12.6	CTCU		0.112	0.101	4.28	4.28
BUL1_1_915_38Z5CTCU_C15	915.18		Z	14	9.6	4.4	CTCU		0.124	0.111	5.55	4.90
BUL1_1_941_20S10CTCU_C	940.89		S	20	9.5	10.5	CTC + CTCU	H	0.133	0.117	4.86	4.40
BUL1_1_941_2S17CTCU_C	940.89		S	25.9	9.2	16.7	CTCU		0.136	0.120	5.13	4.49
BUL1_1_948_01P13CTC_U_A	948.01		P	22	9	13	CTC + CTCU	H	0.127	0.113	4.72	5.41
BUL1_1_948_08P8CTCU_A	948.08		P	17	9	8	CTCU		0.156	0.135	5.46	6.08
BUL1_1_948_08P9TC_A	948.08		P	18	9	9	TC drained	d	0.153	0.133	5.46	6.57
BUL1_1_950_62S13CTCU_B	950.55		S	22	9	13	CTCU		0.122	0.109	5.28	5.59
BUL1_1_950_62S5CTC_B	950.62		S	14	9	5	CTCU + CTC	H,d	0.122	0.109	5.57	5.76
BUL1_1_950_62S9CTCU_B	950.62		S	18	9	9	CTCU		0.126	0.112	5.36	5.52
BUL1_1_950_71P5CTC_U_B	950.55		P	14	9	5	CTC + CTCU	H	0.130	0.115	5.15	5.46
BUL1_1_950_86_Z13CTCU_B	950.55		Z	22	9	13	CTCU		0.133	0.117	5.32	5.49
BUL1_1_950_86Z8CTCU_B	950.55		Z	17	9	8	CTCU		0.126	0.112	5.43	5.49
BUL1_1_970_78S13CTCU_A	970.78		S	22	9	13	CTCU		0.118	0.106	4.47	5.09
BUL1_1_970_78S5CTCU_A	970.78		S	14	9	5	CTCU		0.111	0.100	4.18	4.98
BUL1_1_1016_02S11CTCU_A	1'016.02	Staffegg	S	20	9	11	CTCU		0.075	0.070	3.36	4.52

6.2.1 Saturation and consolidation phase

6.2.1.1 Swelling pressure

In the conventional triaxial procedure (adopted by Lab A and B), the sample saturation was conducted imposing isochoric condition, and monitoring the evolution of the axial and radial stress. The effective stress reached at the end of the saturation phase is called swelling pressure, and the results (grouped by S- and P-geometries, respectively) are reported in Fig. 6-3 versus the sample initial water content (before saturation). For samples with S geometry, the radial direction corresponds the bedding direction, while the axial is perpendicular to it. Swelling pressure was found to be higher in the axial direction, and for both axial and radial direction, this is higher for samples with high water content (corresponding also to high expected clay-mineral content).

On the other hand, in P sample, the axial direction corresponds to the bedding direction, while the radial stress is obtained by maintaining close to zero the average of the deformation in the two radial directions, perpendicular and parallel to bedding.

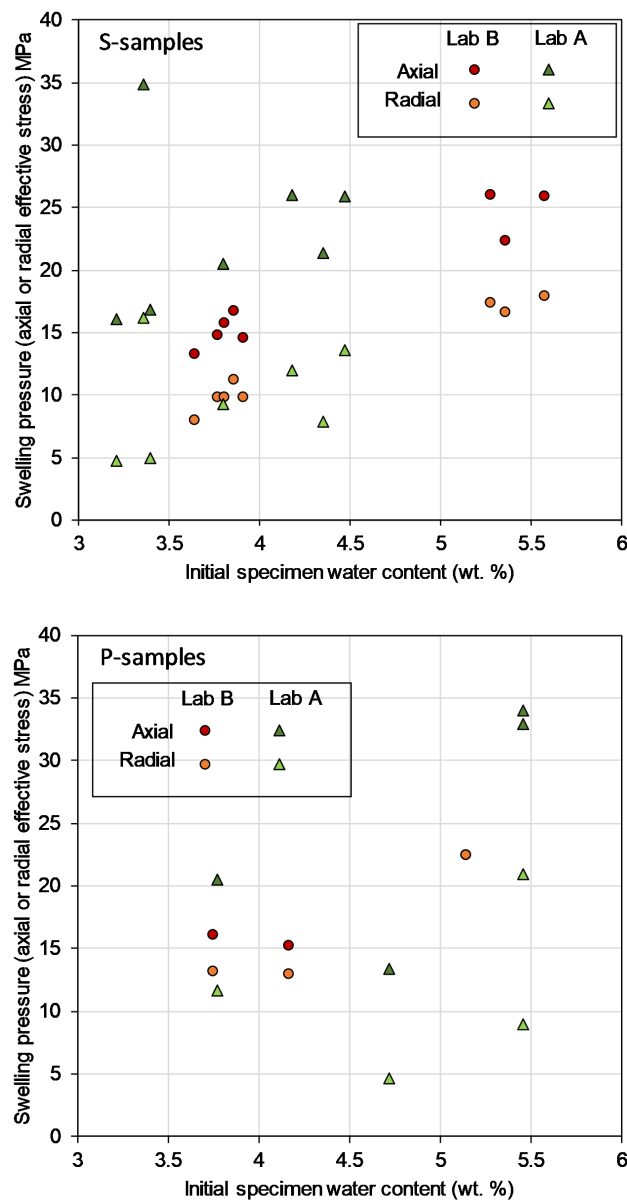


Fig. 6-3: Radial and axial swelling pressure versus initial specimen water content

6.2.1.2 B check tests

In each test, the sample saturation was verified, performing Skempton's B checks. B values were computed over multiple steps. If the difference in B values among the steps was less than 10 %, the average of the measurements was considered as the B coefficient and associated with the average effective confinement level at which B segments were performed. The values are reported in Fig. 6-4 for all results.

The compressibility of the apparatus can affect the accuracy of the measured B values. Wissa (1969) proposed a correction to the B value to consider the compressibility of the dead volume of water (i.e., the water in the drainage system) and the compressibility of the system (tubes and transducers). However, several researchers (e.g., Ghabezloo & Sulem 2010) show that the compressibility of modern types of steel tubes and transducers has a negligible impact on the B mea-

surements compared to the dead water volume impact. Therefore, the results of the testing campaign have been analysed considering the dead water volume in each apparatus, and B-values results ($B_{correct}$) were computed as (Wissa 1969, Favero et al. 2018):

$$B_{correct} = \frac{1}{\frac{1}{B_{obs}} - \frac{V_L}{V} - \frac{K}{K_f(1 - K/K_s)}} \quad (6-1)$$

where B_{obs} is the measured B value, V_L is the volume of fluid in the pore-water lines, V is the volume of the specimen, K is the bulk modulus of the specimen, K_f bulk modulus of the pore fluid, and K_s is the bulk modulus of the solid phase. Bulk and solid moduli $K = 6$ MPa and $K_s = 50$ MPa were considered for the corrections. The correction was applied for results of laboratories A and B, while lab C apparatus considers a close to zero dead volume; therefore, no correction was applied. It has to be noted that the dead volume in apparatus from lab A (2'200 mm³) is about three times larger than that of lab B (700 mm³). The results for the measured and corrected B values are reported in Fig. 6-4. A slight B value decrease with stress increase was detected. An average value of 0.82 was calculated and marked as a dotted line in the graph. In two cases, the correction would predict B values slightly higher than one, that may be due to the assumptions on K and K_s . In one test (lab B), a B fluid pressure tended to increase without stabilisation (and yielded pore pressure increment higher than the applied mean stress variation). The error was due to membrane indentation in the radial filter mesh and fixed after the test.

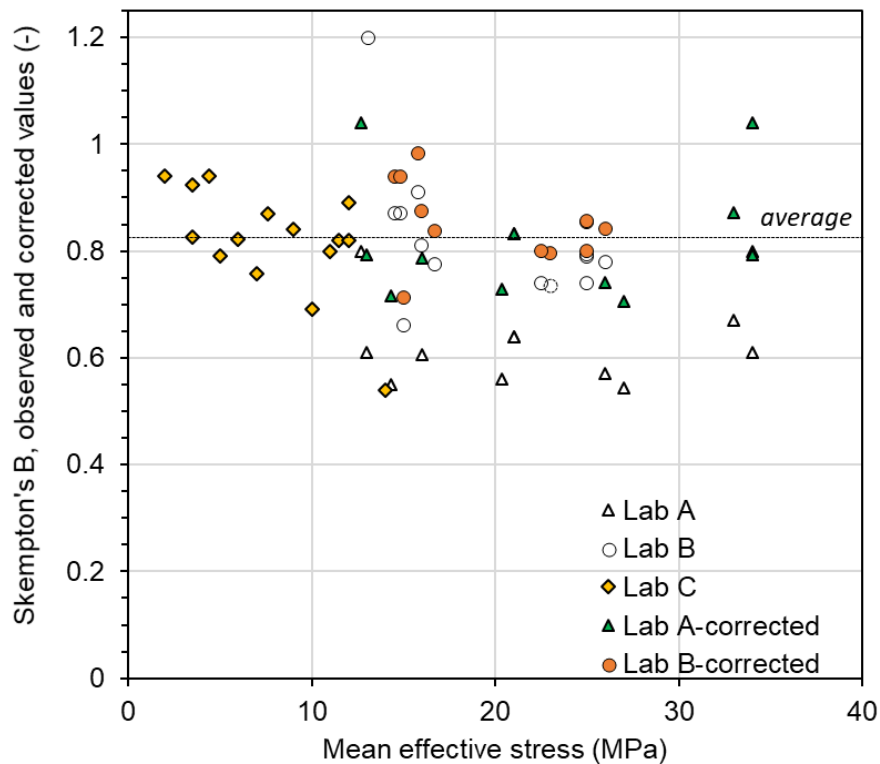


Fig. 6-4: Average Skempton's B values

B values versus the corresponding effective mean stress; all data from BUL1-1 (Opalinus Clay and confining units): Observed and corrected values accounting for dead water volume.

6.2.1.3 Consolidation phase

Drained consolidation phases were performed to achieve the target effective confining stress for shearing, and allow for determining the consolidation coefficient, according to Head (1998), and the strain rate to adopt to allow drained or undrained conditions. The consolidation coefficient for all tested samples is reported in Fig. 6-5 versus the mean effective stress at the end of the consolidation phase.

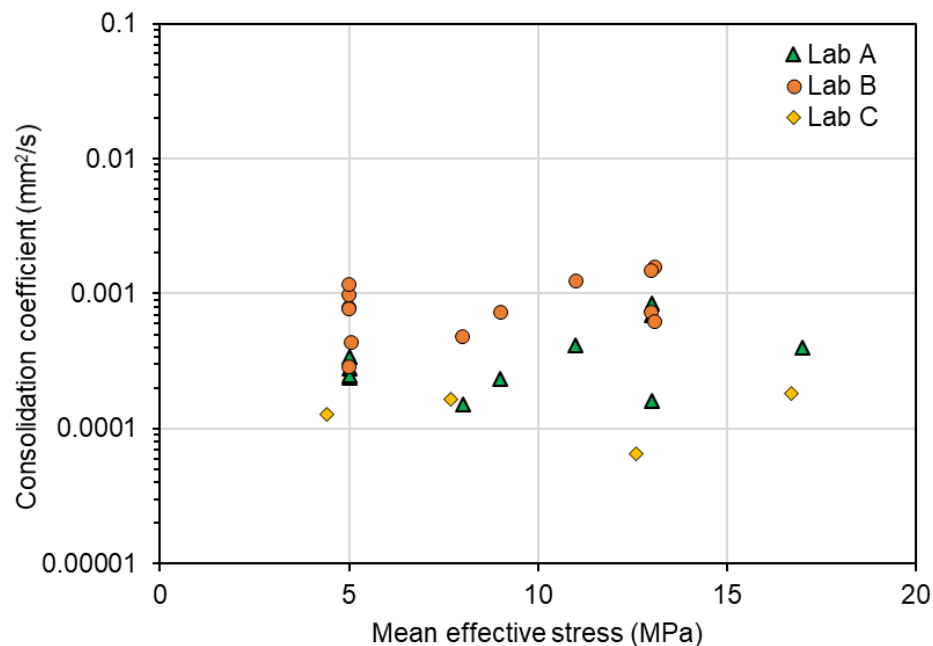


Fig. 6-5: Consolidation coefficient for Opalinus Clay samples (all geometries)

6.2.2 Heating phase

Two tests (BUL1_1_896_60S13CTCUT_B, BUL1_1_896_60S5CTCUT_B) included a heating phase up to 60 °C, performed at the end of the consolidation phase, before shearing. The heating was performed in two steps (15 – 20 °C each), with a heating rate of 1 °C/h. At the end of each phase, the temperature was halted for a few hours to assess the evolution of the pore pressure and the strains. The evolution of the temperature and volumetric strain is reported in time in Fig. 6-6.

Both samples experienced a small volumetric ($\ll 1\%$) expansion during heating. On the other hand, during the phases in which the temperature was halted, the sample at higher confinement (Fig. 6-6, right side), showed deformation in compression, that may be associated to the dissipation of overpressure generated during the heating.

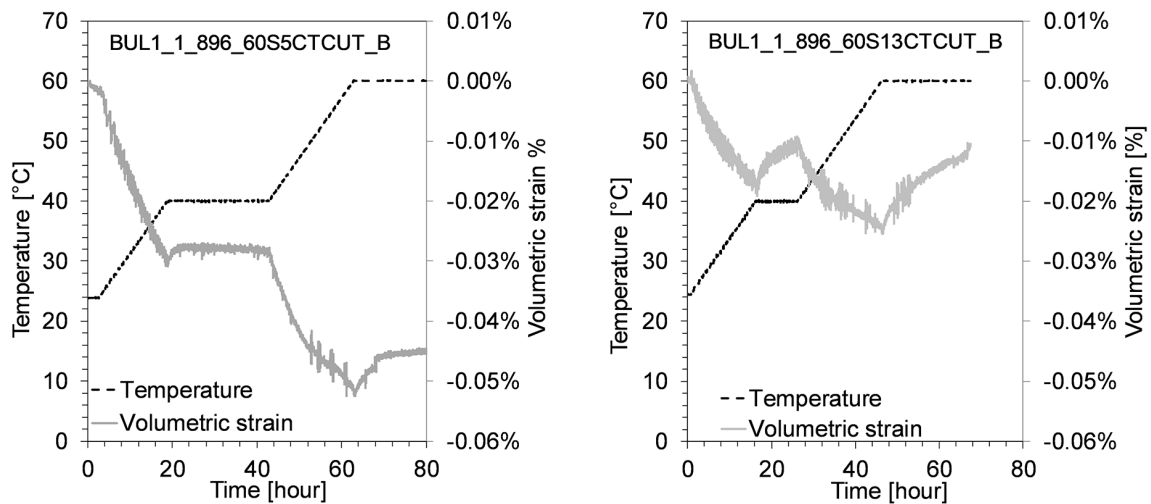


Fig. 6-6: Heating phase of two triaxial tests

6.2.3 Shearing phase

This section provides the results of the shearing phase of tests. The stress paths are reported in the $q - p'$ plane, where q is the deviatoric stress (Eq. 4-1) and p' is the mean effective stress, obtained subtracting to the total effective stress (Eq. 4-2) the pore water pressure u_w . Moreover, the maximum values of q , Δu_w , and AB during the shearing are also illustrated on the different graphs, where Δu_w is the variation of pore pressure from the start of the shearing, AB is the ratio between the Δu_w and Δq .

6.2.3.1 Stress paths of Opalinus Clay subunits

This section focuses on the stress paths obtained on Opalinus Clay samples during the shearing phase of the tests. The behaviour of the tested samples is characterised by a pronounced non-linearity before the peak stress and softening response after the peak. A similar evolution of the deviatoric stress (q) and pore water pressure (u_w) with axial effective strain is observed among tests (see Appendix B).

The effective stress paths obtained on Opalinus Clay samples are here grouped by the lithological sub-units (Fig. 6-7), as presented in Section 4.4.1. The two samples from Subunit 1 tested at high temperature are excluded from the following plots and discussed in the next section. No tests were conducted with core from Subunit 4.

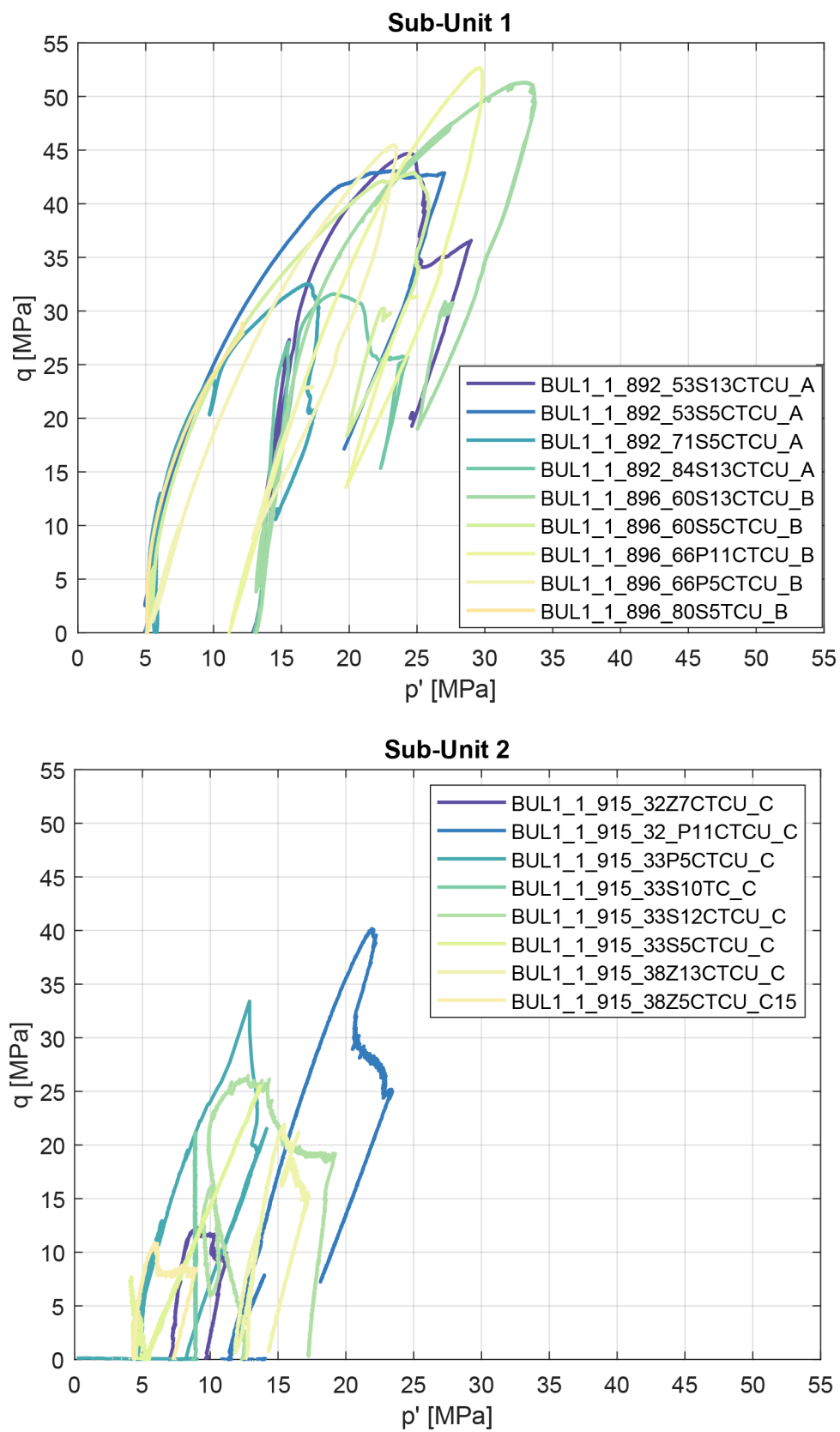


Fig. 6-7: Effective stress paths of tested samples grouped by lithological (sub) unit

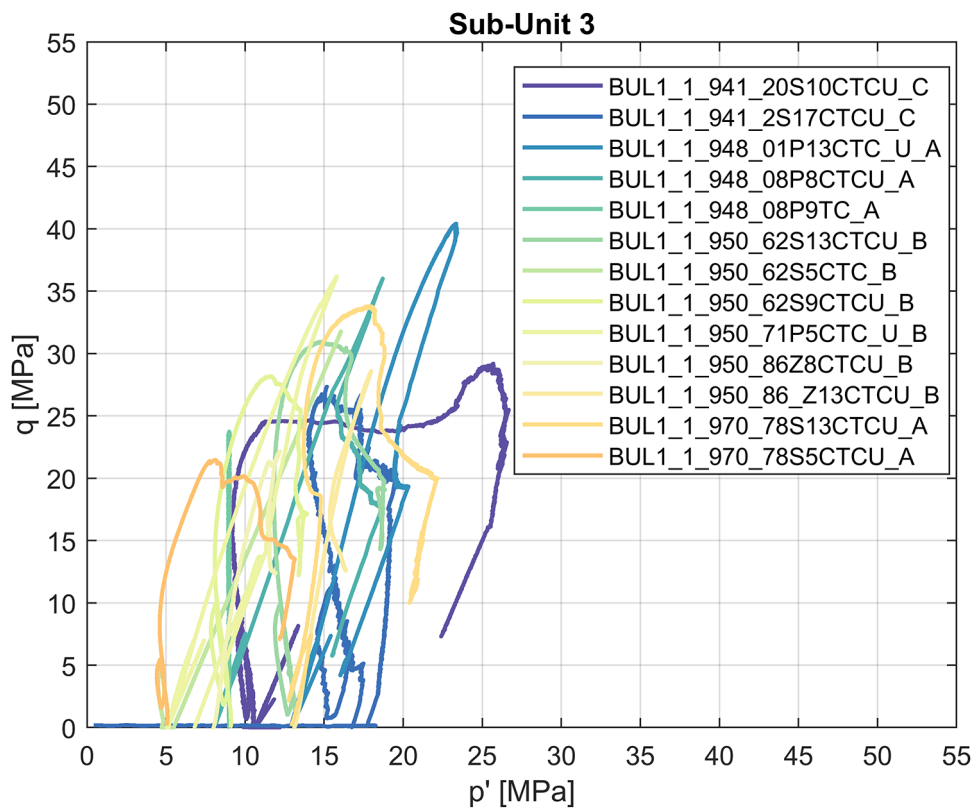


Fig. 6-7: (continued)

6.2.3.2 Temperature effect on the effective stress path during shearing phase

Two pairs of tests were performed on the same section of the borehole: for the first, samples were sheared at room temperature. For the second, samples were heated according to the procedure presented before and sheared at constant temperature (60° C). The stress paths are reported in Fig. 6-8. The stress paths of the high and low-temperature tests show comparable trends. No significant impact of the temperature was found on the Opalinus Clay response during shearing. Accordingly, the remaining testing programme was performed at room temperature.

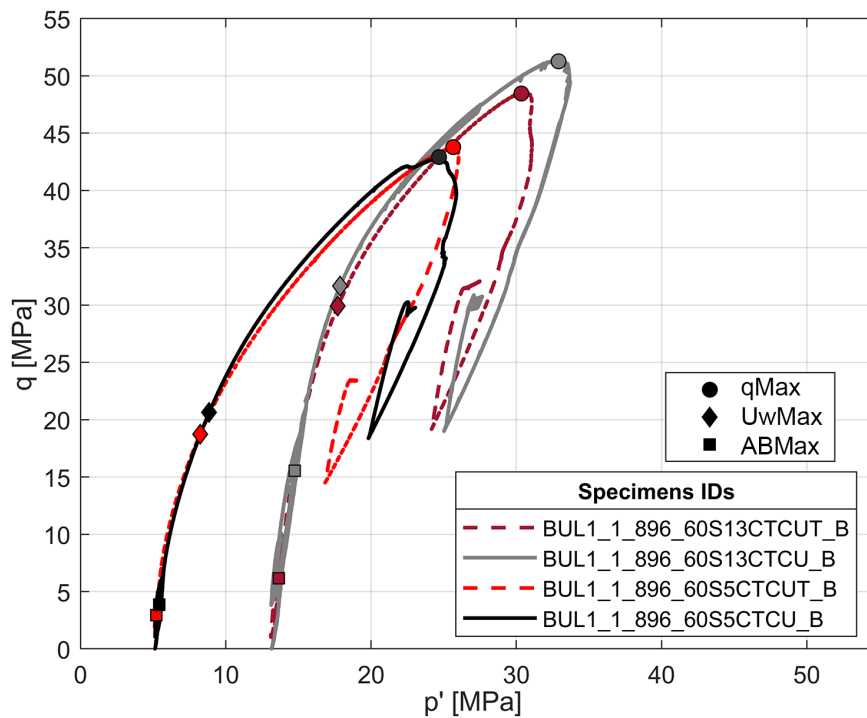


Fig. 6-8: Stress paths for samples sheared at ambient (room) and high (60° C) temperature

Samples are extracted from the same section/depth into the Opalinus Clay:

B04: BUL1_1_896_60S13CTCUT_B; B02: BUL1_1_896_45S5CTCU_B

B03: BUL1_1_896_60S5CTCUT_B; B01: BUL1_1_896_60S13CTCU_B

6.2.3.3 Elastic phase

The analysis of the elastic parameters is presented in this section. Unloading/reloading cycles were performed to estimate the elastic response of the material during an unloading phase. As requested in the testing procedure, the cycles were carried out during the shearing of the specimens before the achievement of the peak stress.

During unloading and reloading, shales present a hysteretic and non-linear behaviour. Depending on the strain amplitude of the unloading-reloading loop, the secant moduli may vary considerably. The strains are calculated as the ratio of the variation in height (or diameter) to the initial height (or diameter). To eliminate the dependency of the moduli on the amplitude of the unloading loop, the elastic modulus for all analysed tests was computed as the secant modulus during unloading for a pre-fixed strain range, 0.02 – 0.04 % from the beginning of the unloading phase (Fig. 6-9). The strain calculation refers to the initial sample height. The strain amplitude was selected as common to all the analysed test results.

Small-strain moduli were also evaluated from the stress cycles (Fig. 6-9) according to the procedure presented in Giger et al. (2018).

An example of elastic moduli evaluation is reported in Fig. 6-9, with reference to test BUL1_1_950_62S9CTCU_B. The top graph in Fig. 6-9 shows the evolution of the secant moduli with the axial strain range used for their computation, for the unloading phase. A clear non-linear decrease of the modulus is observed with the axial strain, which reflects the non-linearity of the material's response. This feature highlights the importance of the axial strain range adopted to

perform unloading-reloading cycles. Different ranges lead to different moduli, making a comparison among the tests difficult and misleading. Generally, larger strain ranges lead to lower moduli (Minardi et al. 2019).

A summary plot comparing small strain moduli and secant moduli is provided in Fig. 6-10. It is highlighted that small strain moduli are always higher than the secant moduli computed over the common strain range. Secant moduli from P and S samples are provided in Fig. 6-11, sorted by sample final water content. In the case of P-tests, higher values of the undrained elastic moduli with respect to S-tests are observed.

The elastic moduli were also computed on the drained loops in hybrid tests, adopting the same procedure as in the undrained ones. Results are reported in Fig. 6-12 (also, Appendix E). In terms of radial response, the relationship between axial and radial deformations during the unloading phase of the cycles performed before the achievement of the peak stress was used ($-\Delta\epsilon_r/\Delta\epsilon_a$) for the assessment of the drained Poisson's ratio (ν) and the undrained equivalent. In the undrained conditions, deviations from the theoretical value of 0.5 typically obtained from the linear isotropic elasticity are due to the anisotropic features of the material and possible poroelastic effects. The computed parameters for both drained and undrained unloading loops are also reported in Fig. 6-12, divided by sample geometry (S and P) and along with the corresponding elastic moduli.

For P-samples, the radial sensors detected the expected anisotropic response, with higher deformation in the direction perpendicular, compared to the direction parallel to the bedding plane. As a consequence, the Poisson's ratio computed using the strain perpendicular to bedding are higher than those parallel to bedding (values in Appendix E). This agrees with previous findings. It is worth to mention that the difference appears to drop for increasing confining effective stress.

Ultrasonic velocities of P (V_p) and S (V_s) waves were also measured by Lab A, B, and C. In particular, the Lab A and Lab C measured the velocity of both P and S waves in the axial direction (V_p and V_s). Lab B measured the velocity of P waves in the axial (V_p) and radial (V_{ph}) directions. These configurations allowed the estimation of the wave velocities considering the anisotropic structure of the tested specimens (bedding orientation).

Results are shown in Fig. 6-13. It is worth noting that the reported wave velocities by Lab C were obtained at a deviatoric stress level of a few MPa (data in Appendix E), while Lab A and B measured wave velocity at the end of the consolidation phase. However, the effect of the difference in deviatoric load among laboratories seems negligible.

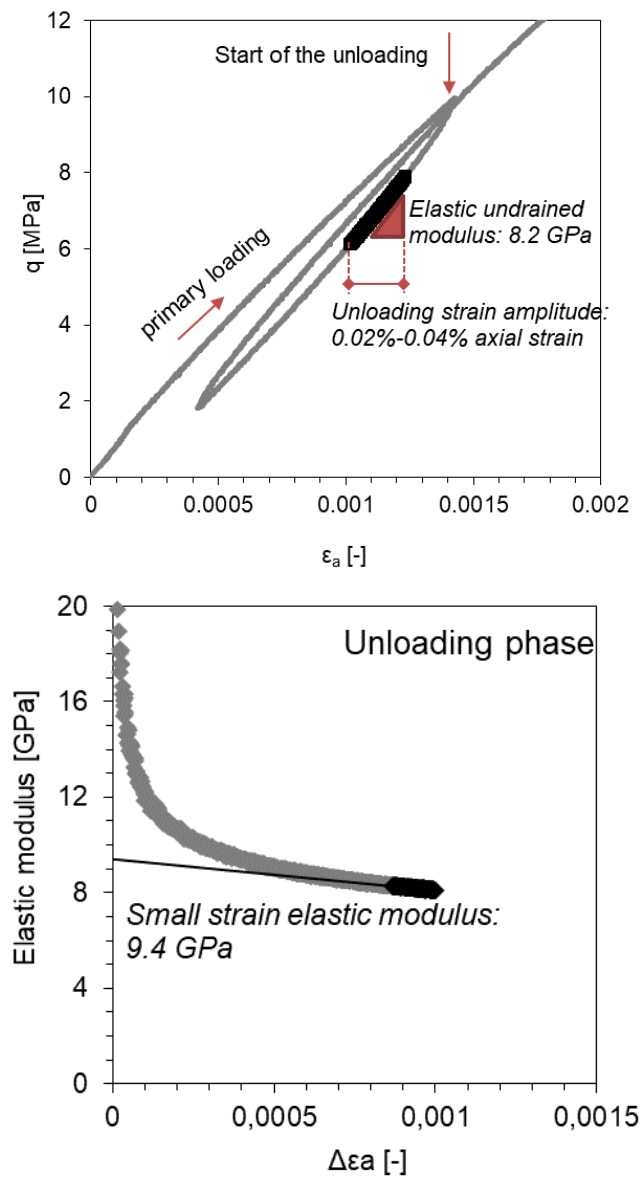


Fig. 6-9: Unloading-reloading cycle and small strain elastic modulus in triaxial tests

Determination of the elastic undrained moduli (E_u) during the unloading phase, for a pre-determined strain amplitude (top); small strain elastic modulus obtained on the unloading phase (bottom) (test BUL1_1_950_62S9CTCU_B).

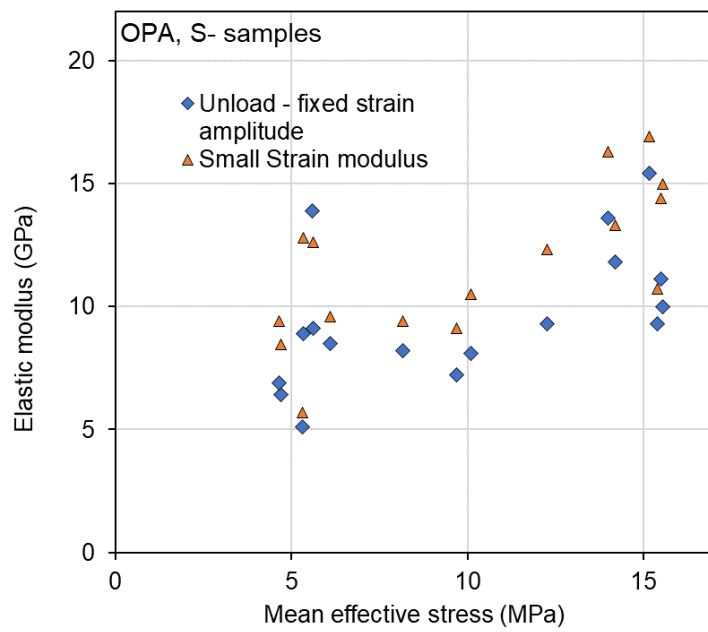


Fig. 6-10: Elastic undrained moduli calculated over an unloading (UL) path: modulus of a fixed strain interval and small strain extrapolation

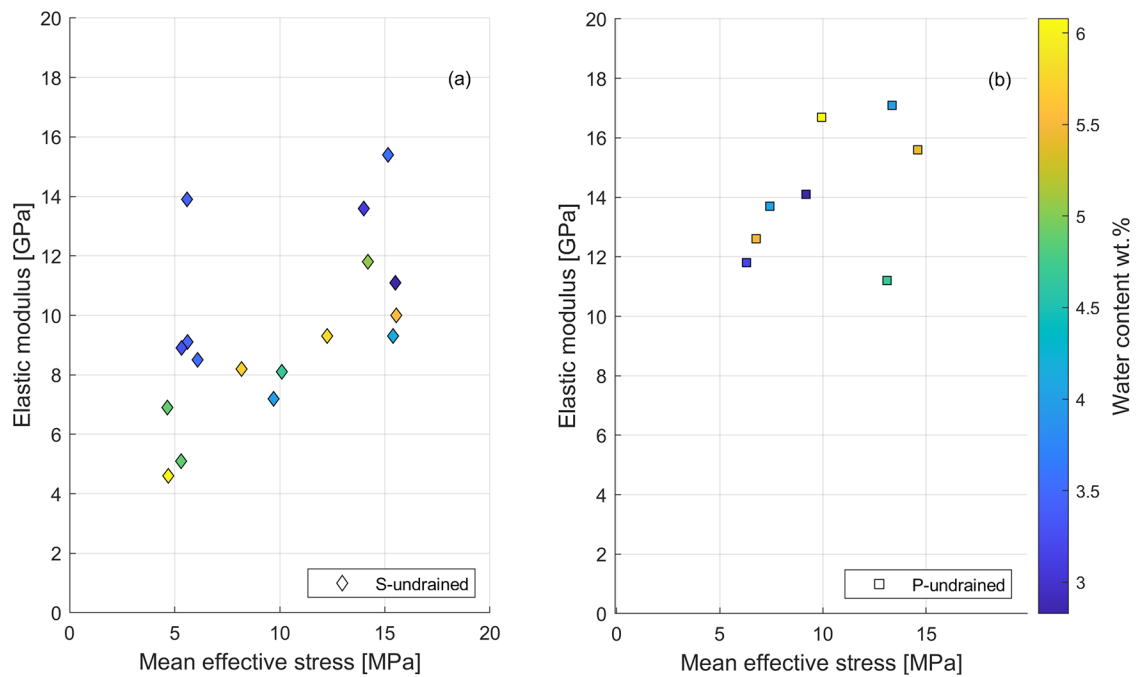


Fig. 6-11: Elastic undrained moduli, sorted by specimen final water content

Moduli are calculated over a fixed strain interval during unloading (UL) path: P and S geometries, sorted by water content.

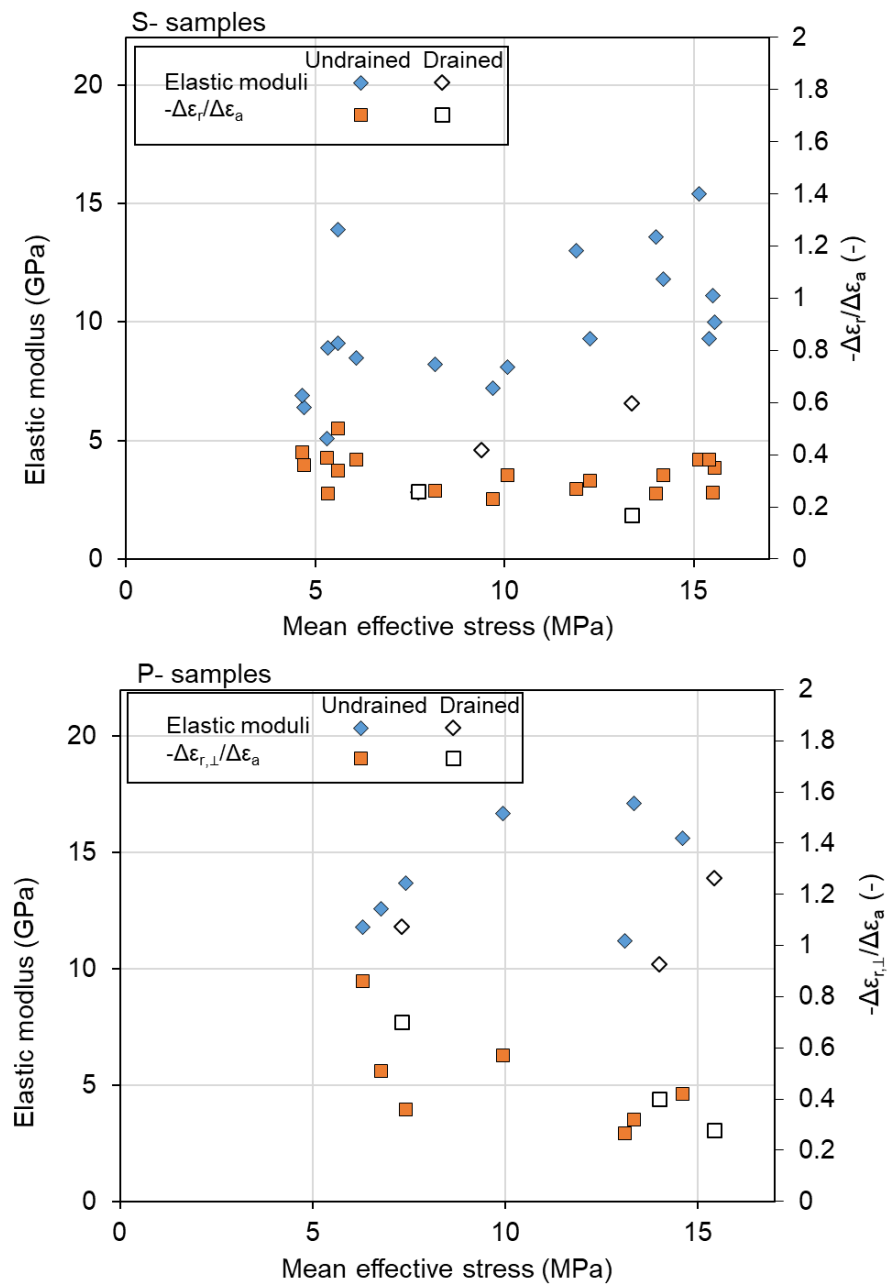


Fig. 6-12: Elastic drained and undrained properties

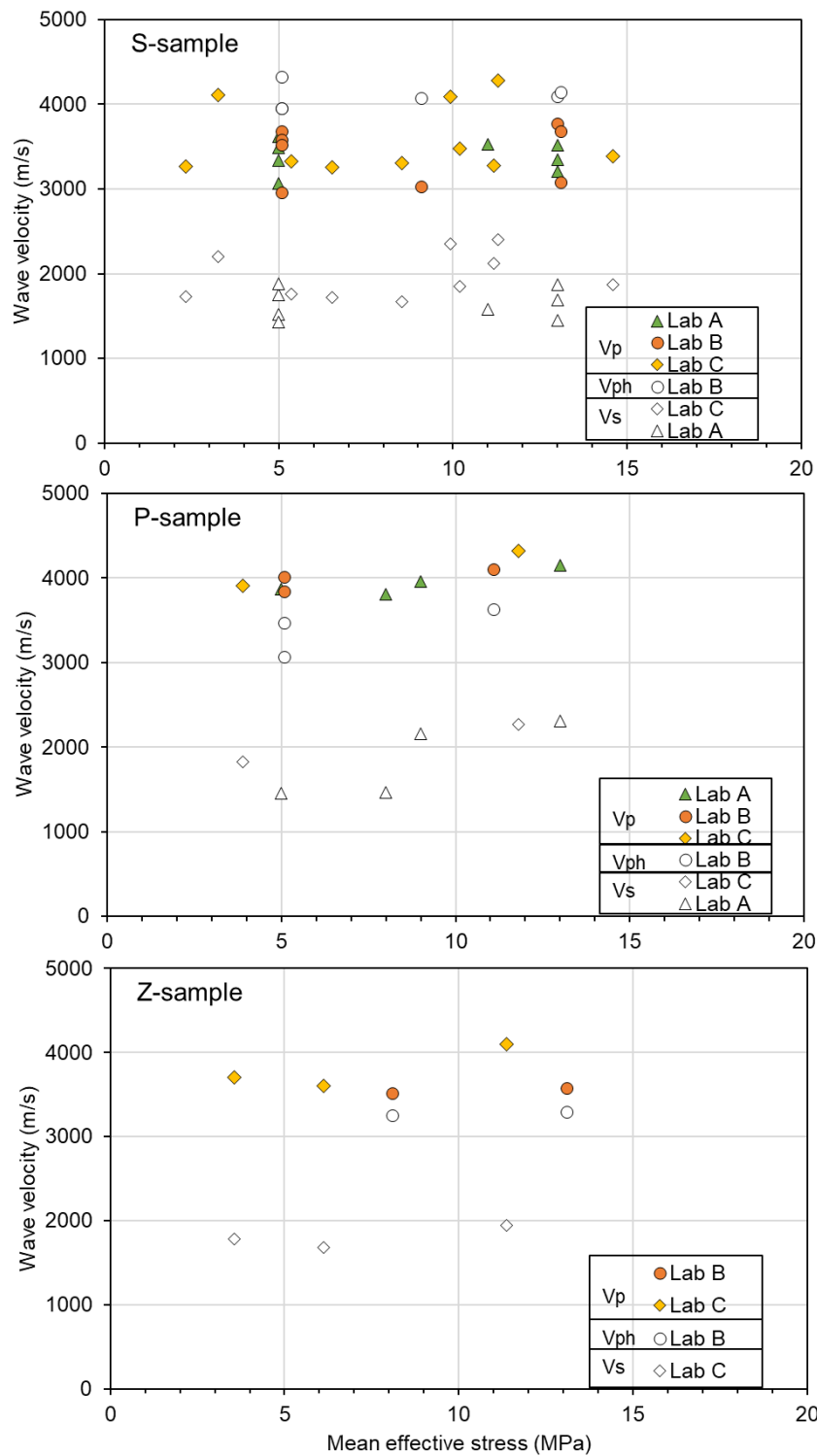


Fig. 6-13: Wave velocities at the beginning of the shearing phase

Results are divided by sample geometry. Lab C measured wave velocities after starting shearing, at a few MPa of deviatoric stress (data in Appendix E).

6.2.3.4 Pore pressure evolution

In Fig. 6-14, the stress levels ($p' - q$), at which the maximum value of the AB parameter and the maximum pore pressure u_w is reached, are plotted, along with the peak shear strength for all test results (data in Appendix B). q is the deviatoric stress, p' is the mean effective stress, Δu_w is the variation of pore fluid pressure from the beginning of the shearing, AB is the ratio between the Δu_w and q . The results are reported in terms of sample geometry.

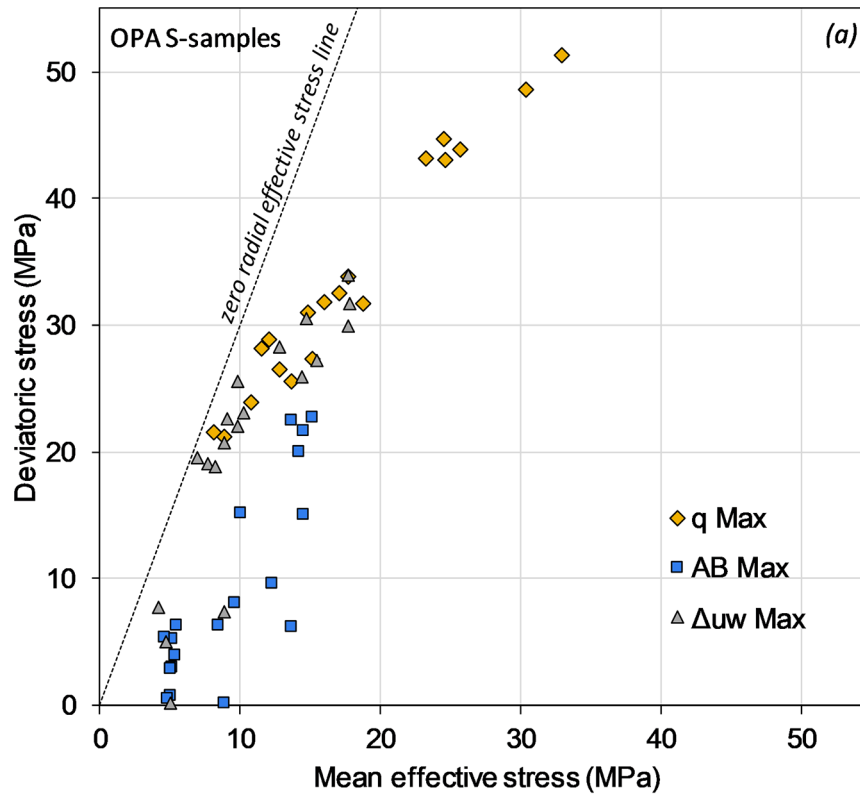


Fig. 6-14: Stress level at which the maximum deviatoric stress, the maximum value of pore pressure and the maximum value of the AB parameter are achieved

OPA results, all geometries (a) S-samples, (b) P and (c) Z samples.

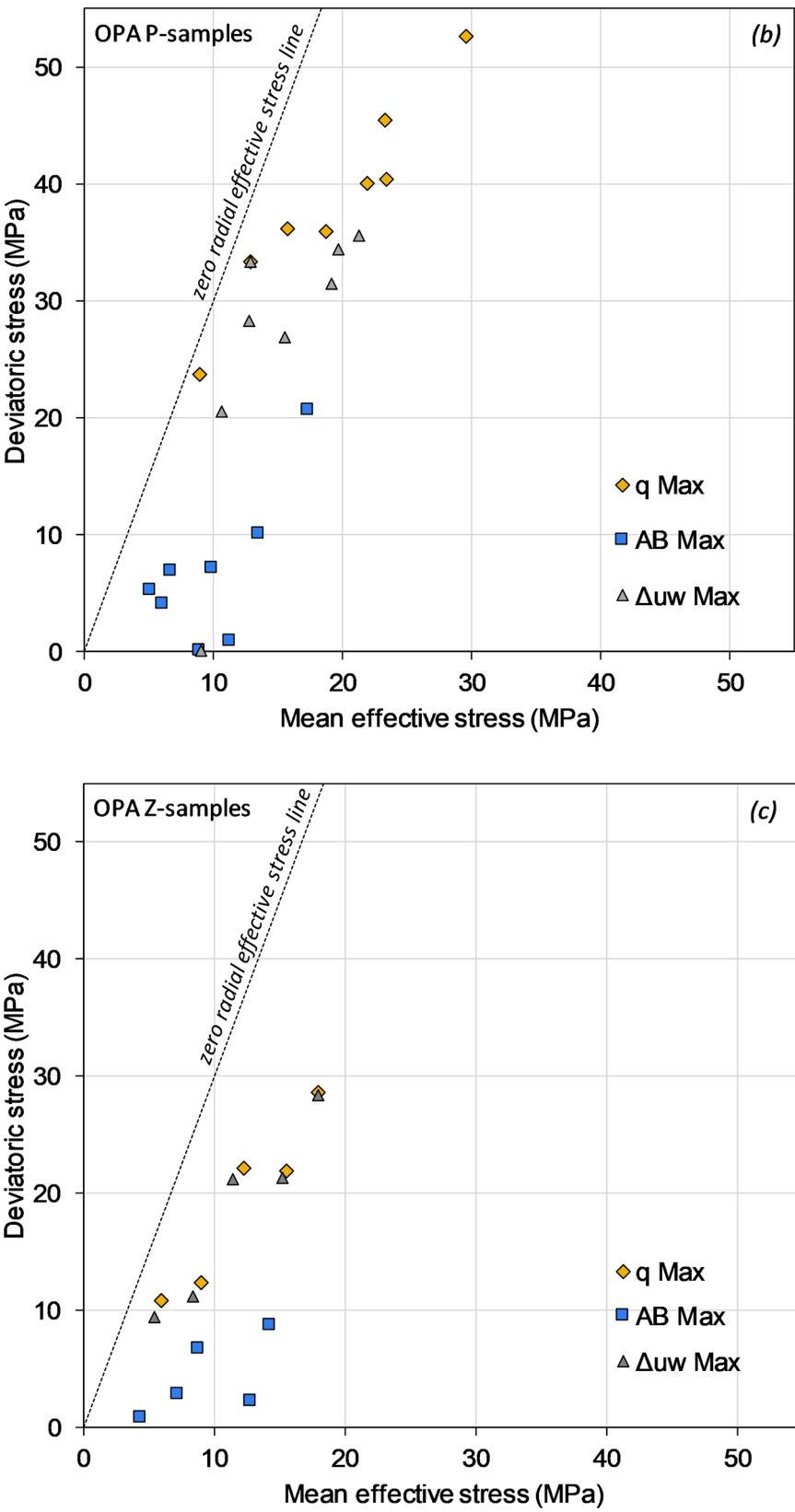


Fig. 6-14: (continued)

6.2.3.5 Shear strength

For all tested specimens, during shearing, a peak in deviatoric stress was achieved, before softening occurred and the achievement of constant deviatoric stress. The peak value is below reported, as the *peak shear strength*, while the constant deviatoric stress achieved while continuing shearing (axial strain up to 2 – 3 %) is hereafter mentioned as *post-peak shear strength*.

In Fig. 6-15, peak (a) and post-peak shear strength (b) are reported in the $p' - q$ plane. Only Opalinus Clay results are reported and grouped by sample geometry. The dotted lines highlight the pair of $p' - q$ values, at which radial effective stress equals zero and would correspond to the same boundary conditions of a UCS test. The stress states on the left side of this line cannot be achieved (negative radial effective stress) in a triaxial apparatus.

In a few cases, the pore pressure building up during shearing in undrained conditions reached the confining stress, bringing the radial effective stress close to zero.

In Fig. 6-16 the peak shear strength results are analysed sorted by the water content (Fig. 6-16a) of the specimens (obtained at the end of the test) and by clay mineral content (Fig. 6-16b), when available. The plot includes the results on the S-sample geometry of the Opalinus Clay formation and the confining units (namely, the Wedelsandstein Formation and the Staffelegg Formation). Within the Opalinus Clay formation, samples with low water content also showed a low clay content and yielded higher strength values. This agrees with the previous finding on Opalinus Clay (Minardi et al. 2019). The results are reported also in Fig. 6-17 in terms of radial versus axial effective stress.

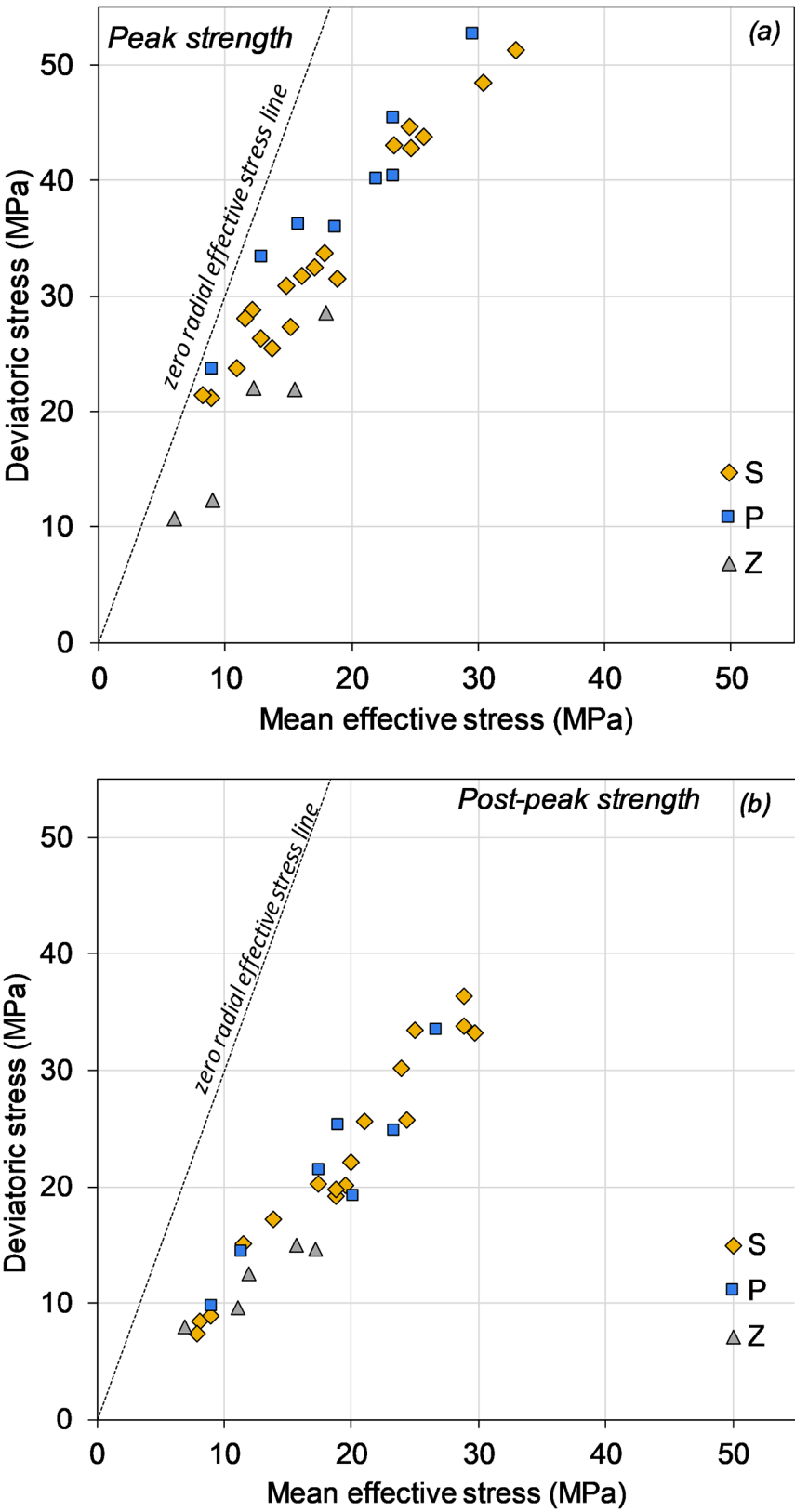


Fig. 6-15: Peak (a) and post-peak (b) shear strength of Opalinus Clay BUL1-1 samples
Results divided by geometry.

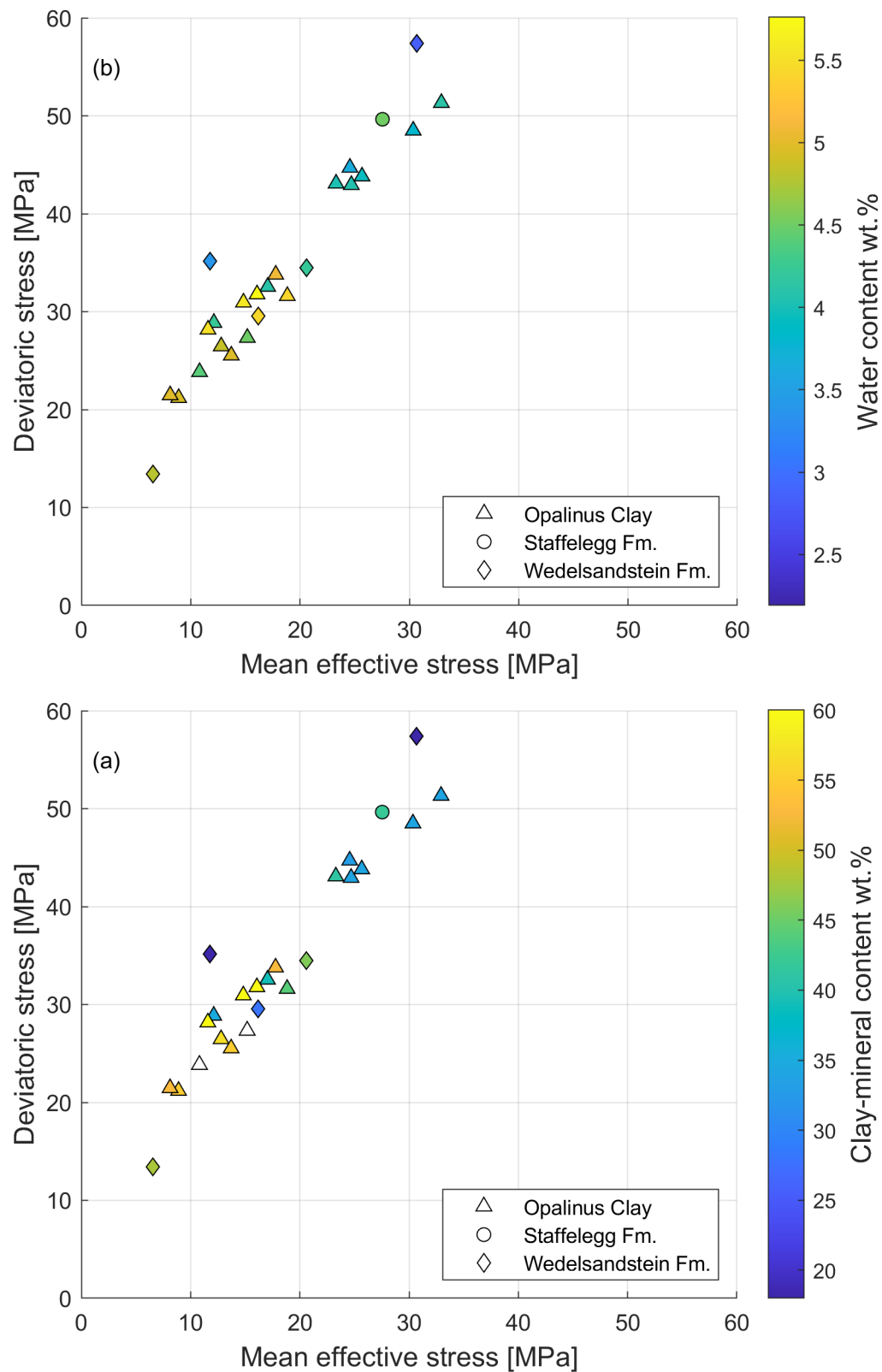


Fig. 6-16: Peak shear strength of samples of Opalinus Clay and confining units

Results in mean effective versus deviatoric stress, sorted by (a) water content and (b) clay mineral content, when available. Only S-samples are reported.

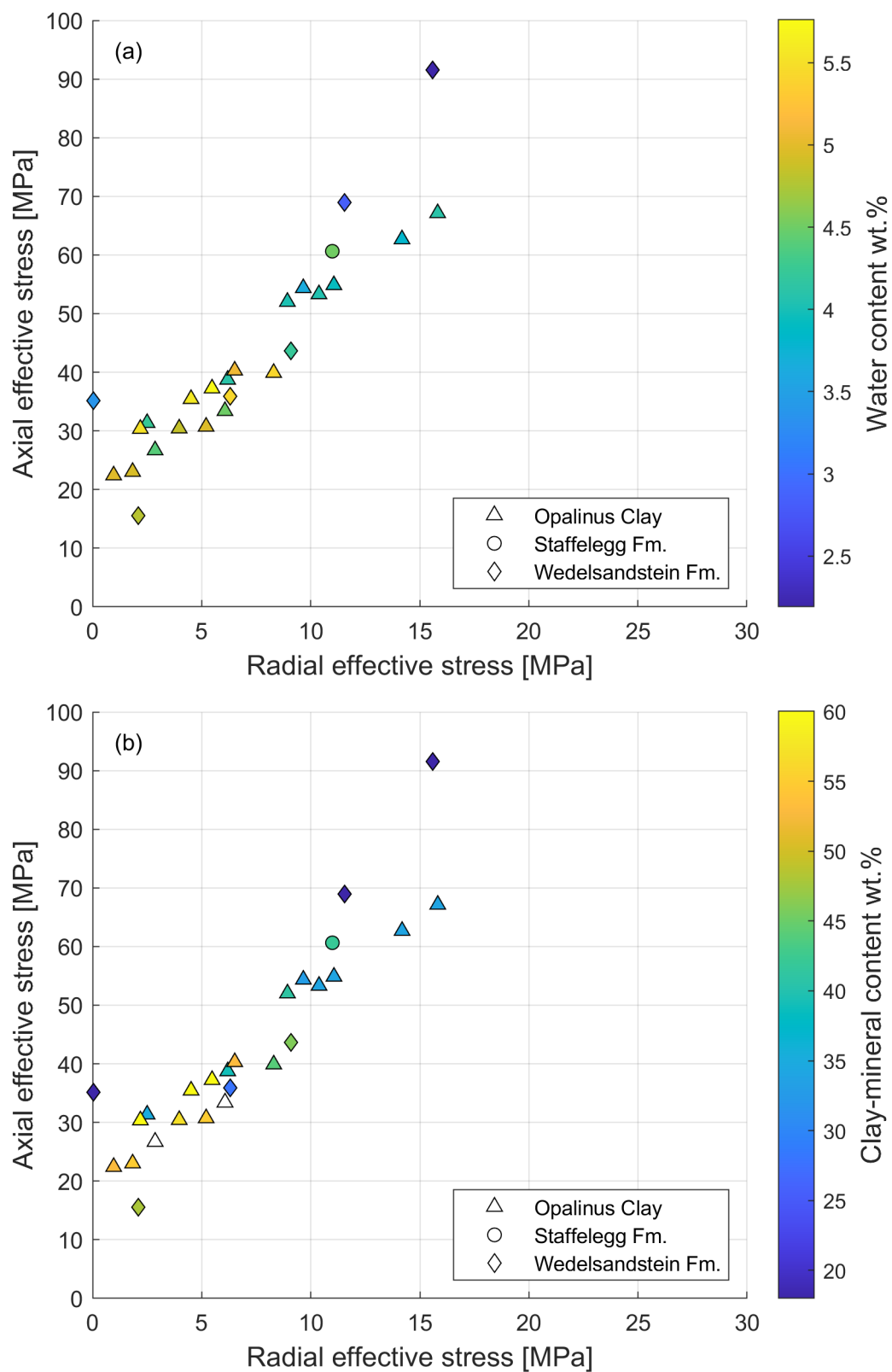


Fig. 6-17: Peak shear strength of samples of Opalinus Clay and confining units

Results in radial versus axial effective stress, sorted by (a) water content and (b) clay mineral content, when available. Only S-samples are reported.

6.2.3.6 Analysis of the Z-samples results

The results on samples with Z geometry are reported in Fig. 6-18. In the axial strain versus deviatoric stress graph, test BUL1_1_915_32Z7CTCU_C showed a two peaks response during shearing; the maximum shear strength was reached at axial strain 0.008, about twice the axial strain at the peak of the other tests with the same geometry. The peak deviatoric strength results lower compared to the other Z-test results, which can be an indication of a pre-damaged sample. This geometry was selected in order to obtain the shearing band in the direction sub-parallel to the bedding. Therefore, any fissuring along the bedding would directly impact the shear band formation, i.e., the peak shear strength achieved.

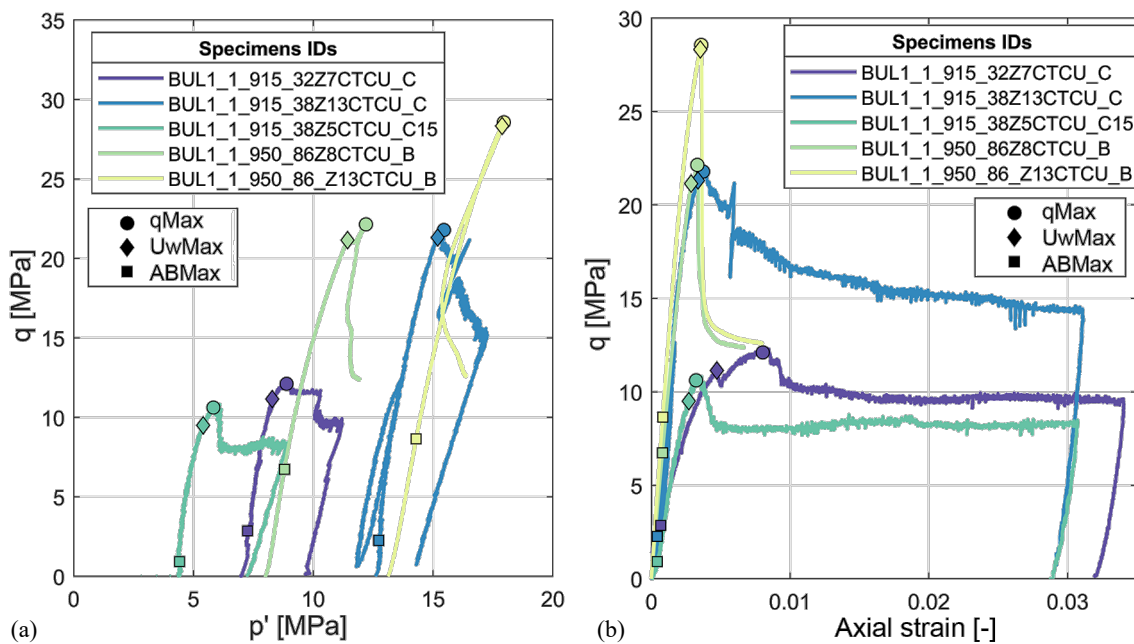


Fig. 6-18: Deviatoric stress (a) and axial strain (b) during shearing phase of Z-geometry samples

6.3 Oedometric tests, one-dimensional swelling and complementary permeability tests

This section includes the results obtained on:

- High pressure oedometric tests (OED)
- One-dimensional swelling test (OS)
- Constant head permeability tests (PERM)

OED, OS tests were obtained in oedometric (lateral strain prevented) conditions. For 6 samples, during oedometric tests, a constant head permeability phase was performed. Two additional PERM tests were performed in a triaxial apparatus, in isotropic stress conditions.

6.3.1 Overview of the performed tests and initial specimen conditions

The tests in oedometric conditions were labelled by indicating the letter of the laboratory that performed the test (e.g., A), a sequential number (01) and the type of test (OED = oedometric, OS = one-dimensional swelling, PERM = constant head permeability test). An overview of the performed tests is provided in Tab. 6-3.

Tab. 6-3: Tests performed in oedometric conditions and complementary permeability tests

ID	Depth [m]	Laboratory	Sample geometry	Formation
D01OED_BUL1_1	859.33	D	S	Wedelsandstein Fm.
D02OED_BUL1_1	859.32	D	P	Wedelsandstein Fm.
D03OED_BUL1_1	935.96	D	S	Opalinus Clay
D04OED_BUL1_1	935.87	D	P	Opalinus Clay
D05OED_BUL1_1	935.96	D	P	Opalinus Clay
D06OED_BUL1_1	936.20	D	S	Opalinus Clay
D07OED_BUL1_1	1'001.37	D	P	Staffelegg Fm
D01OS_BUL1_1	936.09	D	S	Opalinus Clay
D02OS_BUL1_1	1'001.29	D	S	Staffelegg Fm
D03OS_BUL1_1	1'001.29	D	P	Staffelegg Fm
A01OS_BUL1_1	970.67	A	S	Opalinus Clay
A02OS_BUL1_1	970.68	A	P	Opalinus Clay
A03OS_BUL1_1	970.75	A	S	Opalinus Clay
A04OS_BUL1_1	970.75	A	P	Opalinus Clay
A01PERM_BUL1_1	827.77	A	S	«Herrenwis Unit»
A02PERM_BUL1_1	827.77	A	S	«Herrenwis Unit»
A01OED_BUL1_1	892.50	A	S	Opalinus Clay
A02OED_BUL1_1	857.39	A	S	Wedelsandstein Fm.
A03OED_BUL1_1	857.39	A	P	Wedelsandstein Fm.
A04OED_BUL1_1	1'015.77	A	P	Staffelegg Fm.
A05OED_BUL1_1	970.56	A	P	Opalinus Clay

6.3.2 Oedometric test results

The settlement over time curve for each loading increment is analysed adopting an extension to consolidation theory tailored on shales response (Ferrari et al. 2016). The solution combines a modified form of the classical Biot theory (Biot 1941), to account for the behaviour of shales under oedometric conditions, and an extended one-dimensional consolidation theory, to consider the poroelastic behaviour of shales and time-dependent loading conditions. For each loading step, the method allows for the identification of the primary consolidation settlement, related to the water overpressure dissipation, the secondary compression coefficient, the hydraulic conductivity.

As an example, the results of one oedometric test is reported in Fig. 6-19, in terms of oedometric curves. The graph presents the void ratio (e) obtained at the end of the primary consolidation, versus the vertical effective stress (σ'_v). The initial void ratio (e_0), the swelling pressure attained at the end of the resaturation phase (S_p), the yield stress ($\sigma'_{v,y}$) obtained through the Casagrande's construction, the compressibility (C_c) and the unloading (C_s) indexes are marked in the figure. The results from the performed tests are summarised in Tab. 6-4. Corresponding plots are reported in Appendix F. Tab. 6-4 also includes the effective stress level at which the constant head permeability test was performed, where applicable.

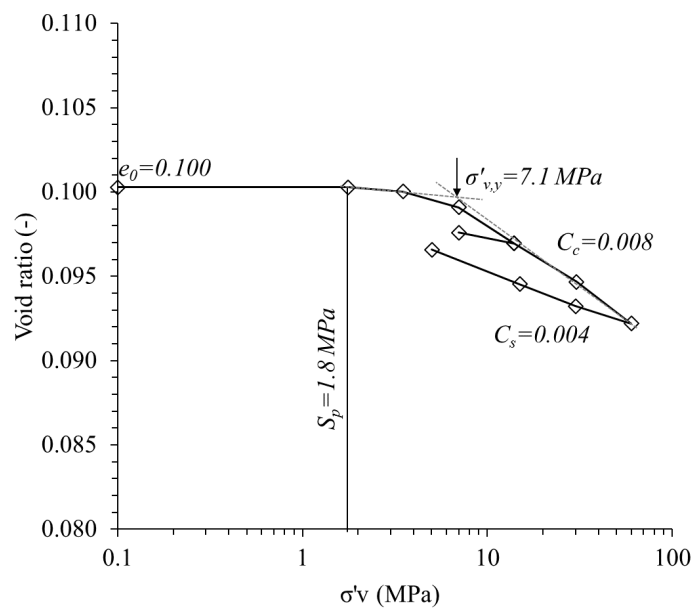


Fig. 6-19: Oedometric curve in void ratio versus vertical effective stress plane
Results from A01OED_BUL11.

Tab. 6-4: Oedometric and hydraulic conductivity test results

Nagra ID	Geo-metry	Specimen depth	e_0	C_c	S_p	$\sigma'_{v,y}$	C_s	K test at: σ'_v (or p')
		[m]	[-]	[-]	[MPa]	[MPa]	[-]	[MPa]
D01OED_BUL1_1	S	859.29	0.145	0.02	3.8	15.8	0.007	-
D02OED_BUL1_1	P	859.29	0.12	0.016	1.2	12.5	0.006	-
D03OED_BUL1_1	S	935.96	0.082	0.017	10.3	30.3	0.007	-
D04OED_BUL1_1	P	935.87	0.118	0.010	11.8	25.4	0.003	14
D05OED_BUL1_1	P	935.96	0.076	0.009	1.4	14.4	0.002	-
D06OED_BUL1_1	S	936.18	0.157	0.024	6.9	17	0.012	14
D07OED_BUL1_1	P	1'001.37	0.121	0.009	14.3	27.9	0.002	20
A01OED_BUL1_1	S	892.57	0.1	0.008	1.8	7.1	0.004	10
A02OED_BUL1_1	S	857.39	0.116	0.008	1.36	5.5	0.004	15
A03OED_BUL1_1	P	857.39	0.103	0.004	1.88	12	0.002	-
A04OED_BUL1_1	P	1'015.77	0.113	0.006	0.1	4.9	0.002	-
A05OED_BUL1_1	P	970.56	0.106	0.004	0.18	10.1	0.002	15
A01PERM_BUL1_1	S	827.9	0.014	-	-	-	-	$p' = 10$
A02PERM_BUL1_1	S	827.9	-	-	-	-	-	$p' = 10$

A wide range of swelling pressures was measured in oedometric conditions, ranging from fractions of MPa (typically for P geometries) up to about 10 MPa. It is noted that those values are generally lower than those obtained in triaxial conditions. This is attributed to a more precise strain control available in the triaxial setups.

6.3.3 Hydraulic conductivity: direct and indirect measurements

From the consolidation theory application, hydraulic conductivity at each loading step can be derived. In the following Fig. 6-20, the hydraulic conductivity of P and S samples obtained from consolidation theory are reported. The ranges of values obtained upon primary loading are represented as the average plus bars: the plot considers the values obtained for a range of vertical effective stress comparable to what is expected in situ (5 – 20 MPa). The results are combined with constant head permeability measurements and labelled with the values of vertical effective stress (for oedometric conditions) or mean effective stress (for triaxial conditions) at which the test was performed. Test A01PERM_BUL11 did not yield to a stable flow through the sample; therefore, hydraulic conductivity could not be estimated for this test.

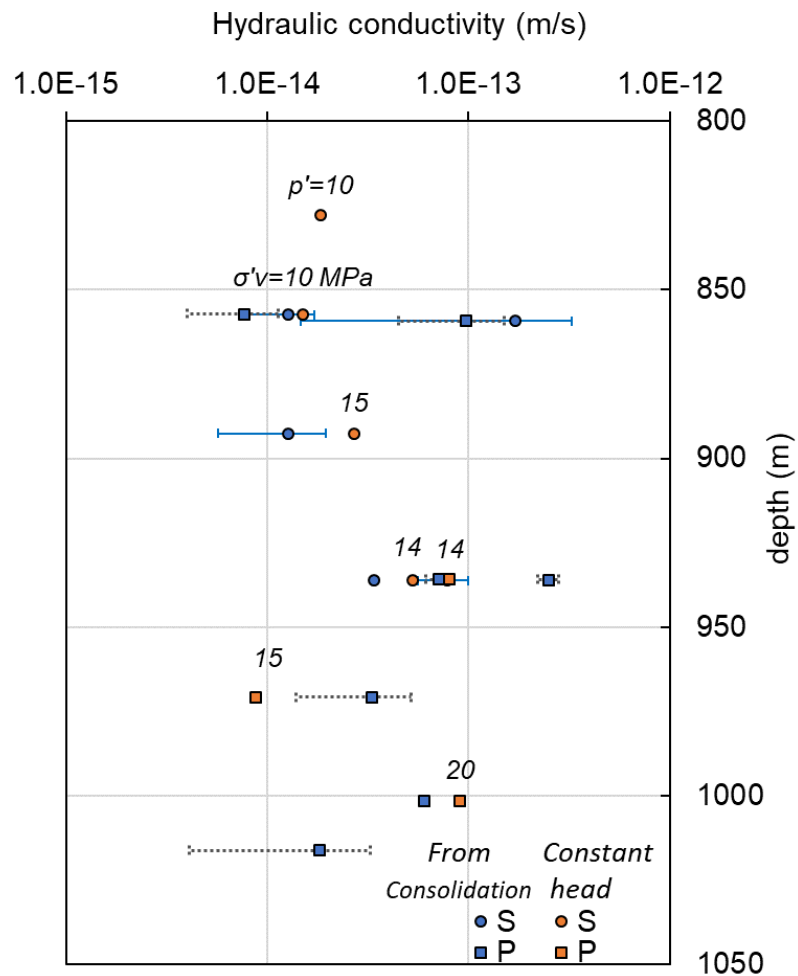


Fig. 6-20: Hydraulic conductivity versus depth

Results obtained from the analysis consolidation test and constant-head tests in oedometric and triaxial conditions.

6.3.4 One-dimensional swelling tests

The results of the one-dimensional swelling tests are reported in this section. An example of the evolution of the axial strain in time, while providing pore fluid allowing for one-dimensional swelling is reported in Fig. 6-21. The results are summarised in Tab. 6-5 (plots in Appendix F). The table reports the swelling pressure ε_{swell} attained once the deformation stabilised in absolute value (the minus sign is dropped for the sake of simplicity). Pore pressure u_w adopted during the test is also reported.

Lab D performed three tests, injecting pressure at 50 kPa and applying constant stress to counter-balance the fluid pressure. Lab A reported issues in the first two tests: a leakage was reported, not allowing to reach constant pore pressure, and in the first phases the pore fluid pressure was equivalent to the axial stress applied, inducing a movement of the piston. Therefore, the strains reported in the initial phases have been affected and may be not representative of the real material response. A lower pore pressure was used for tests A03 and A04. However, test 4 reported a non-expected pore pressure fluctuation.

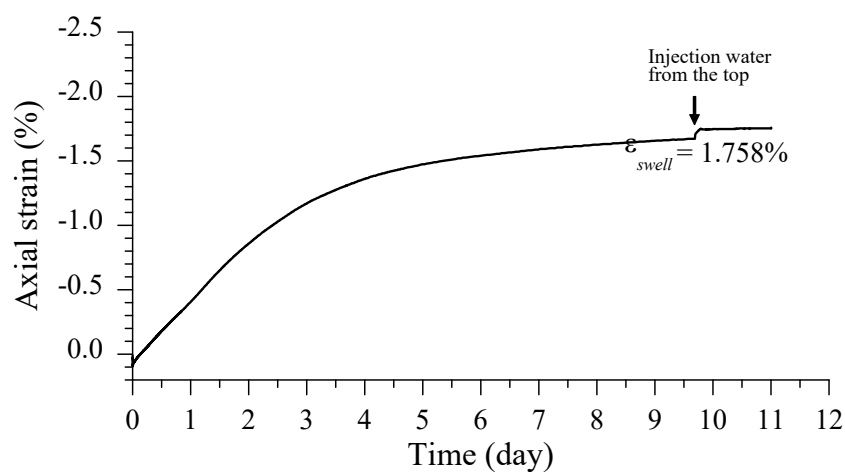


Fig. 6-21: One-dimensional swelling test, evolution of the axial strain in time
Results obtained on specimen D01OS_BUL11.

Tab. 6-5: One-dimensional swelling test results

* Results may be affected by piston's movement during water injection.

Nagra ID	Core depth	Geometry	Formation	ϵ_{swell} (absolute value)	u_w
	[m]			[%]	[kPa]
D01OS_BUL1_1	936.09	S	Opalinus Clay	1.79	50
D02OS_BUL1_1	1'001.29	S	Staffelegg Fm	0.56	50
D03OS_BUL1_1	1'001.29	P	Staffelegg Fm	0.33	50
A01OS_BUL1_1	970.67	S	Opalinus Clay	4.58 *	-
A02OS_BUL1_1	970.68	P	Opalinus Clay	1.40 *	-
A03OS_BUL1_1	970.75	S	Opalinus Clay	0.33	10
A04OS_BUL1_1	970.75	P	Opalinus Clay	2.4	5

7 Representativeness of test results

A critical question revolving around any rock mechanical or geomechanical laboratory programme is how representative the chosen specimens are for a given formation. This depends on several factors, notably material heterogeneity and number of core intervals used to characterise rock and rock mass properties. Tab. 7-1 provides an overview of the average spacing of test intervals, considering the formation thickness and the number of core intervals used for testing, more specifically for strength tests (both TRX and UCS).

Tab. 7-1: Average spacing of compressive strength tests in various formations

Formation	Thickness	Core for TRX/UCS	Spacing
	[m]	[number]	[m]
«Felsenkalke» / «Massenkalk»	150.0	3	50
Schwarzbach Fm.	23.0	2	12
Villigen Fm.	74.5	3	25
Wildeggen Fm.	54.2	1	54
«Herrenwis Unit»	41.9	2	21
Wedelsandstein Fm.	28.4	3	9
Opalinus Clay	104.2	7	15
Staffelegg Fm.	34.6	1	35
Klettgau Fm.	36.8	1	37
Bänkerjoch Fm.	64.6	1	65
Schinznach Fm.	72.7	2	36
Zeglingen Fm.	64.3	1	64
Weitenau Fm.	50.6	2	25

The test interval is relatively small (15 m) in Opalinus Clay and the Wedelsandstein Formation (9 m), but also for the Schwarzbach Formation (12 m), despite these formations exhibiting relatively small material heterogeneity compared to some other formations. The lower spacing in Opalinus Clay is obviously justified by the greater relevance for the exploration programme. In the Malm (e.g. «Felsenkalke» / «Massenkalk» or Wildeggen Formation) and Keuper / Muschelkalk sections (e.g. Bänkerjoch or Zeglingen Formation), the spacing is often > 50 m despite large material heterogeneity. For those intervals, correlation with geophysical logging is considered a key aspect to derive formation specific rock mechanical properties. Such work is documented in a separate report, but Fig. 7-1 provides some insight. It compares continuous velocities derived from the sonic scanner in situ (blue line) and ultrasonic velocities constrained in triaxial tests on core in the laboratory (*cf.* Tabs. 5-4 and 6-2, orange points in Fig. 7-1). Such correlations can be used for both i) checking the plausibility of test results and ii) estimating the variability of material properties in (sub-)formations. Therefore, the tests conducted in the rock mechanical testing programme with formations other than Opalinus Clay (and confining units) are used as guidance to estimate rock mechanical formation properties.

In contrast, the ambition of the geomechanical testing programme (of Opalinus Clay in particular) is to cover the relevant spectrum of material properties by actual laboratory testing. Whether this was achieved is primarily assessed by comparing the basic properties and mineralogy of tested specimens with the larger database (*cf.* Dossier VIII). In this larger database, water content and mineralogy were constrained every 2 to 3 m along the 104 m thick interval of Opalinus Clay, and each measurement reflects a mean value over a 25 cm core interval.

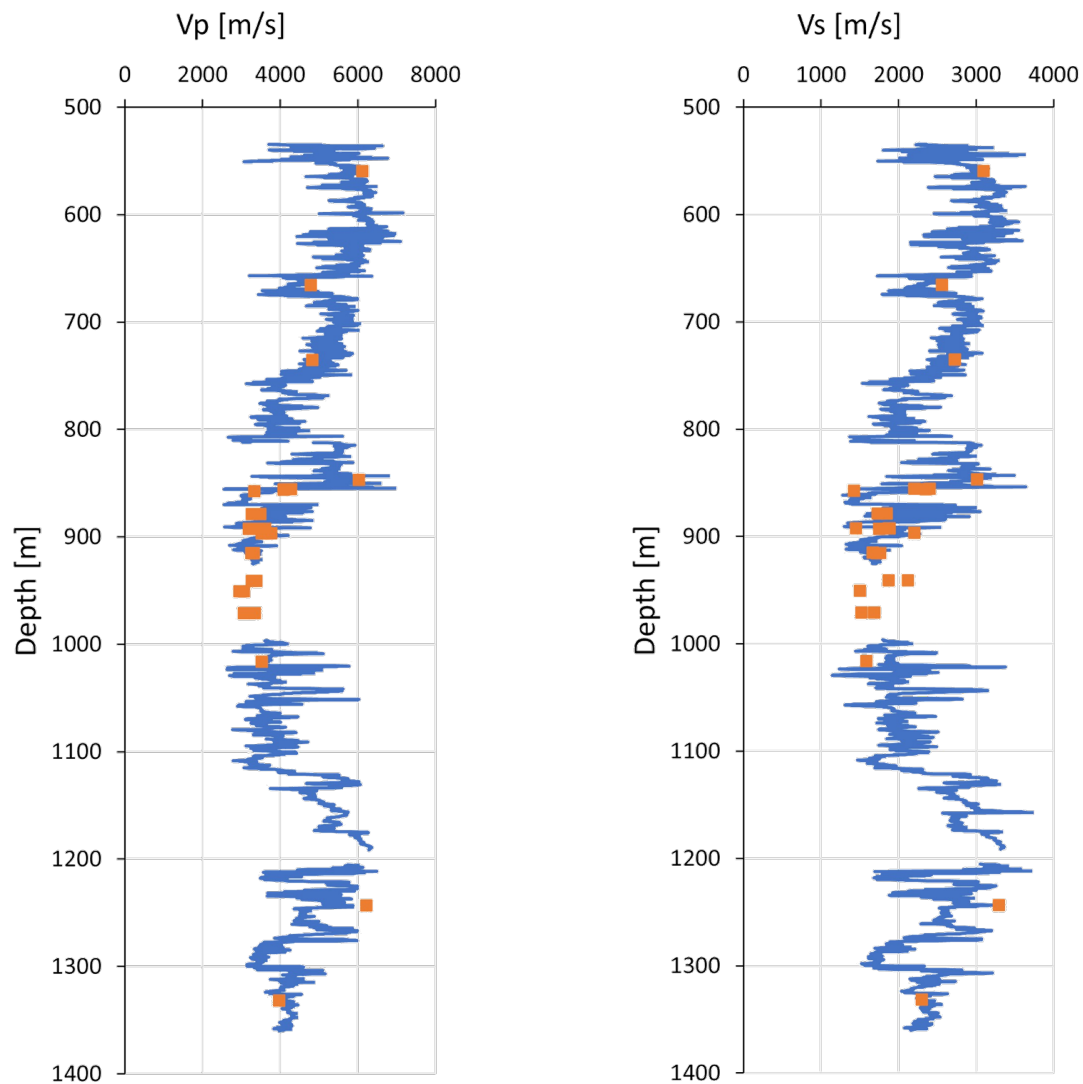


Fig. 7-1: Compressional (a) and shear-wave velocities (b) from borehole and lab testing

Blue line: continuous measurements from borehole sonic scanner (kHz frequency). The gap in the Opalinus Clay between approximately 930 m and 1'000 m was due to bad hole (borehole stability problems), not petrophysical logging. Orange squares: lab-measurements (MHz frequency).

The water content of geomechanically tested samples was measured on trimming from the sample preparation and on the samples themselves (before and after testing). For a same sample, these values were averaged, and the error bar constructed to include the variability in the measurements. Those results are reported versus the samples sourcing depth in Fig. 7-2. Based on water content, the comparison with the larger database reveals that the spectrum of water content is covered by the geomechanical tests.

The bulk mineralogy was also assessed for the tested specimens. However, it is noted that for a given depth interval multiple «twin specimens» were sourced to constrain stress-dependent properties of the same material. As the material variability in the horizontal direction in the Opalinus Clay is considered very small compared to the variability in the vertical direction, only one specimen was generally used for a given depth interval as a reference mineralogy for other specimens. This assumption verified with repeated analyses of two plugs from an identical depth interval. The clay mineral content in the two analyses varied by only 2 wt.-%, consistent with a value of 3 wt.-% in a previous comparison (Minardi et al. 2019). Therefore, the bulk mineralogy obtained on one specimen was attributed to the other samples from the same vertical interval of the core. The clay-mineral content (%) is reported in Fig. 7-3 against the depth, together with the large database obtained from University of Bern (*cf.* Dossier VIII).

It is acknowledged that Sub-Unit 4 of Opalinus Clay (*cf.* Section 5.4.1 and Fig. 6-6) was not tested. But the comparisons of Figs. 7-2 and 7-3 provide evidence that the material heterogeneity is covered in the tests.

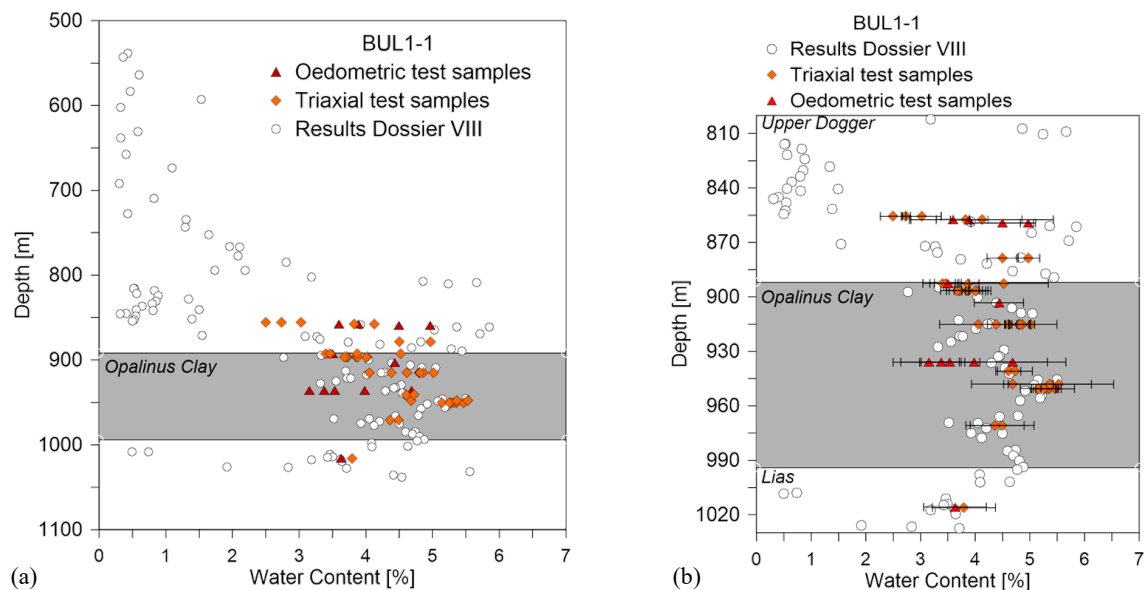


Fig. 7-2: Comparison of the water content vs. depth from the large data base and from those tested in this report

Results include those from Dossier VIII for the BUL1-1 borehole (a) and zoom into the Opalinus Clay and confining units' section (b).

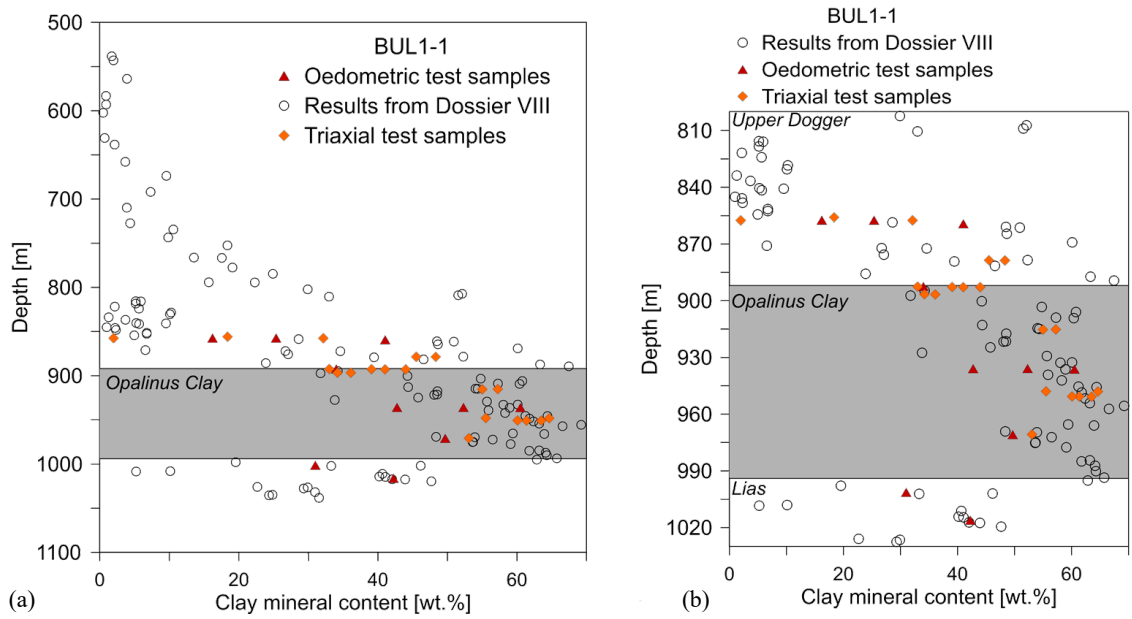


Fig. 7-3: Clay-mineral content vs. depth from the large data base and from those tested in this report

Results include those from Dossier VIII for the BUL1-1 borehole (a) and zoom into the Opalinus Clay and confining units' section (b).

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Appendix A: Photo documentation of the rock mechanical testing programme

TS (according to well/depth)

BUL1-1

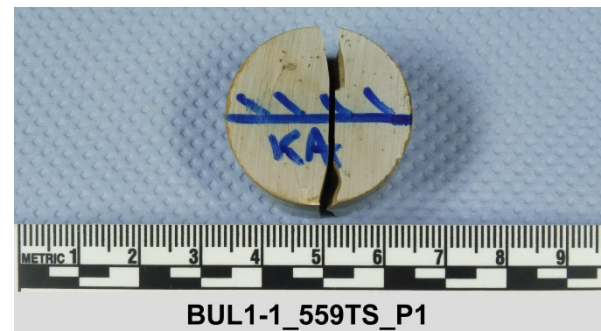
before testing

after testing

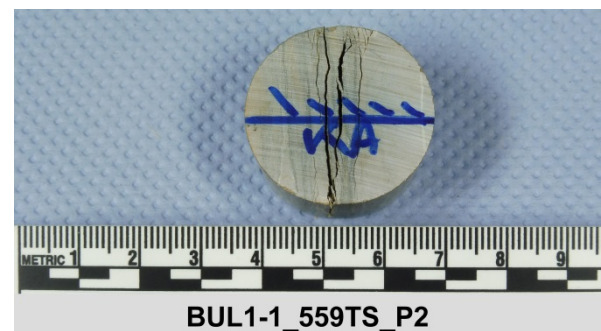
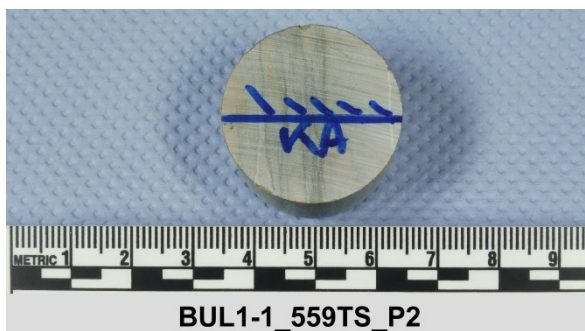
559.22 m



559.27 m



559.32 m

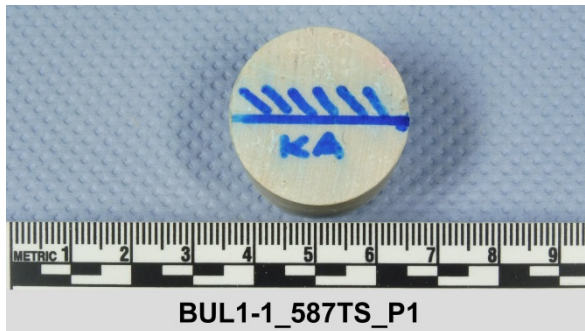


587.62 m



before testing**after testing**

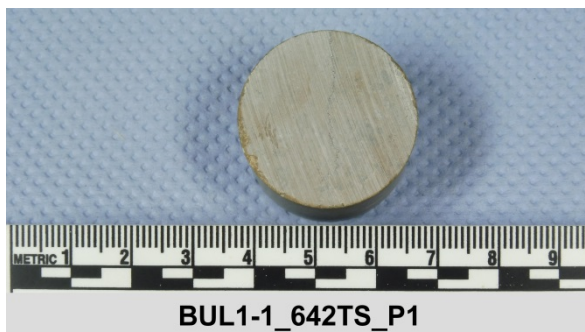
587.67 m



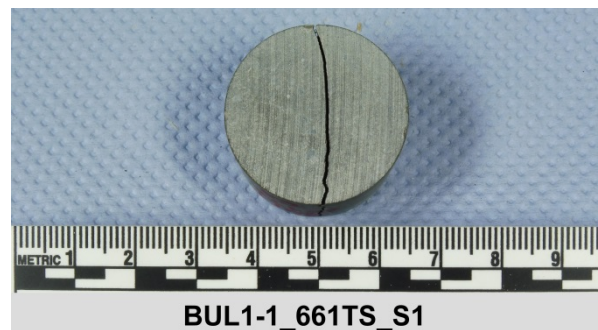
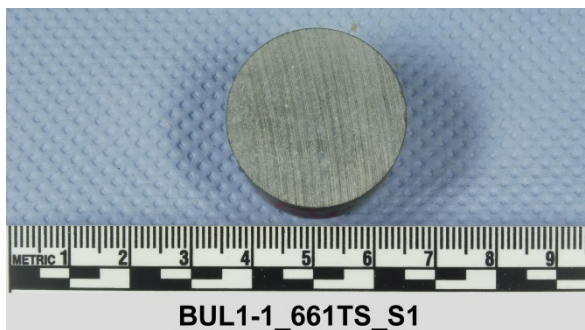
642.53 m



642.55 m



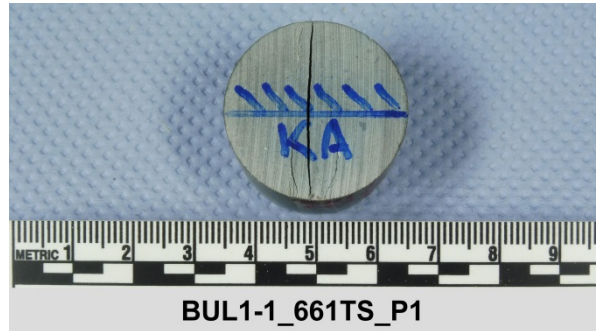
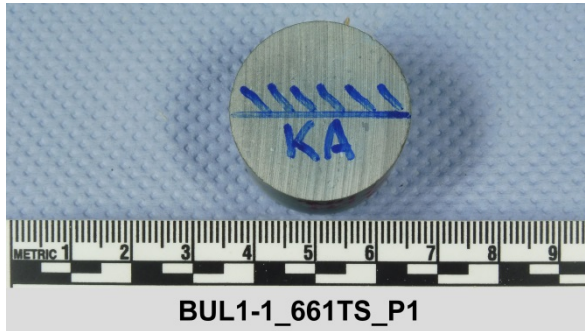
661.22 m



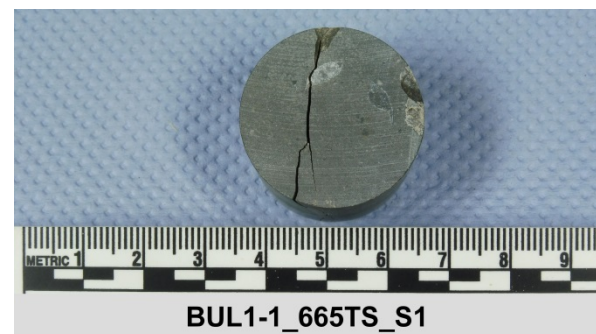
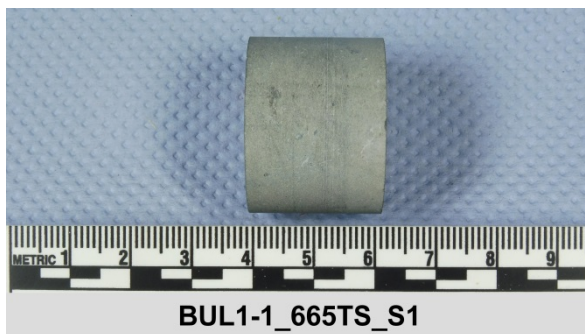
before testing

after testing

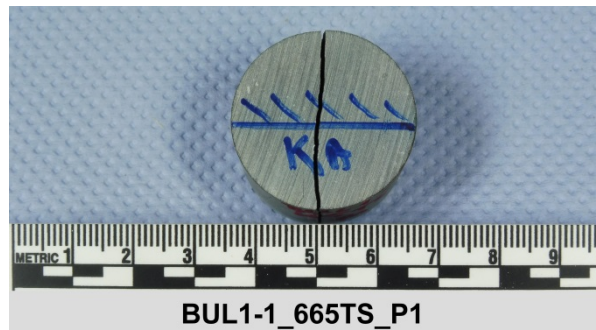
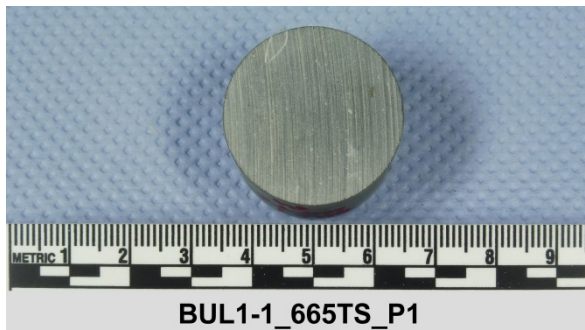
661.29 m



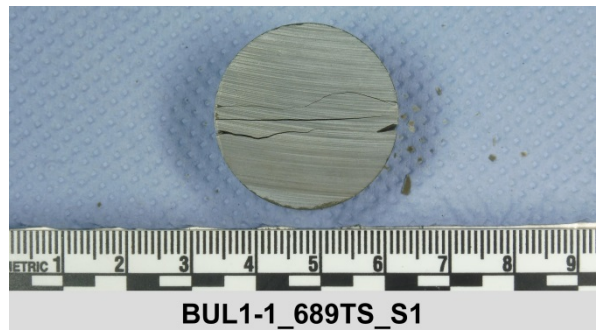
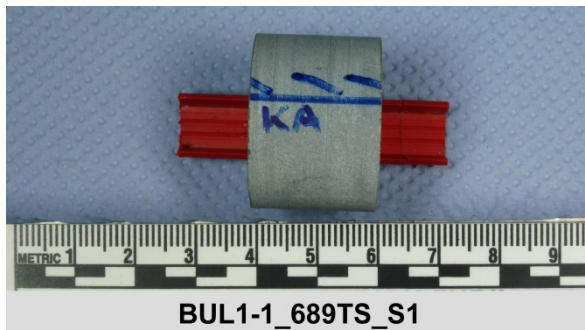
665.24 m



665.30 m



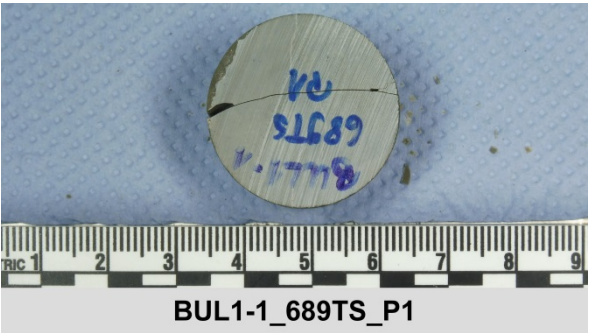
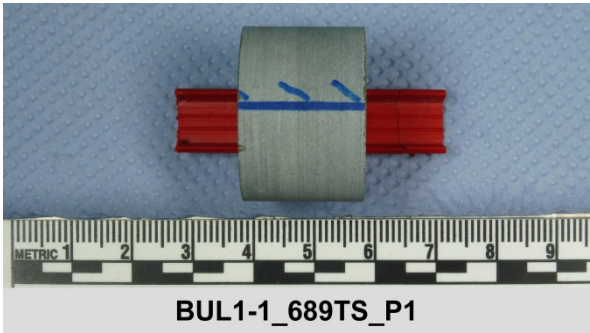
689.67 m



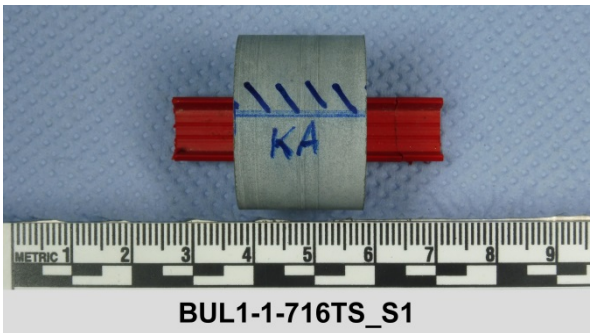
before testing

after testing

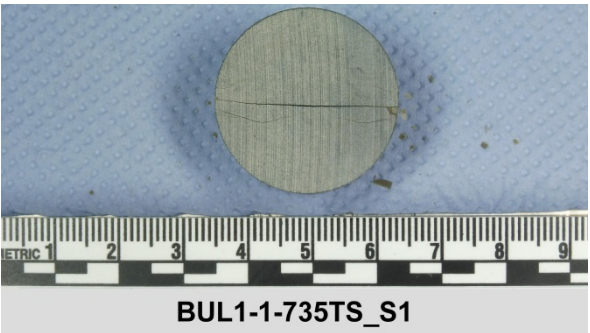
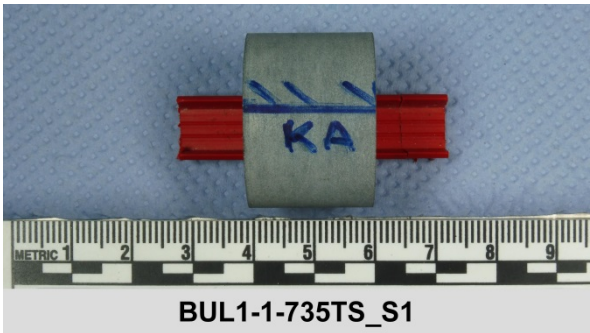
689.74 m



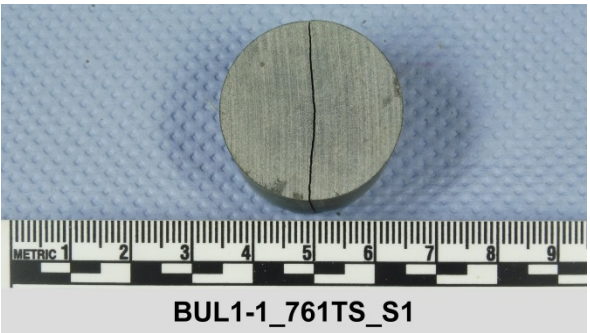
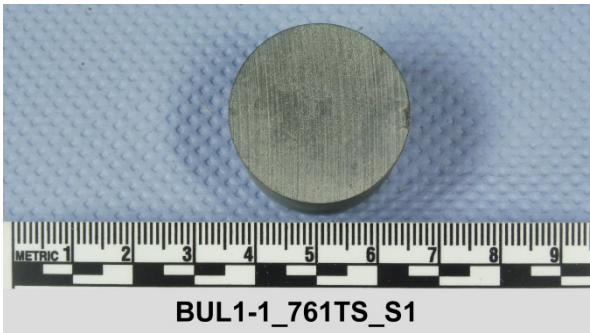
716.32 m



735.21 m



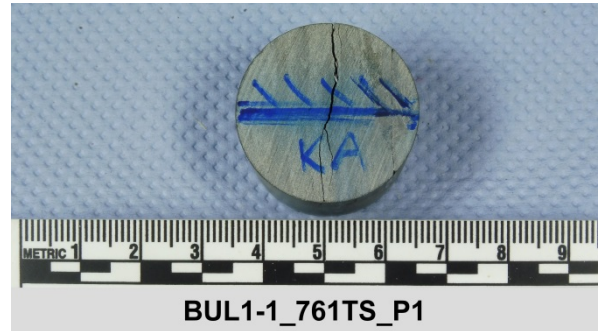
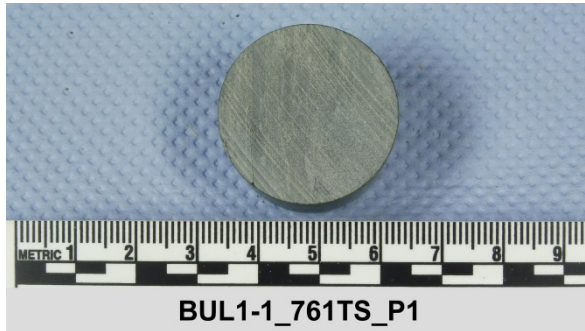
761.67 m



before testing

after testing

761.72 m



816.73 m



816.90 m

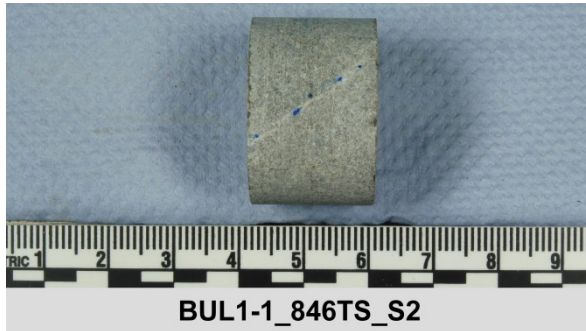


846.59 m

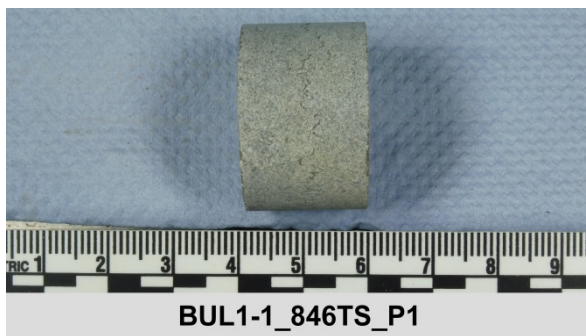


before testing**after testing**

846.64 m



846.75 m



TS (according to well/depth)

before testing

1'055.00 m



BUL1-1B

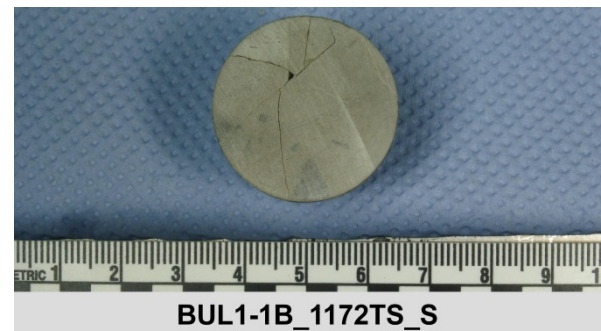
after testing



1'099.99 m



1'172.13 m



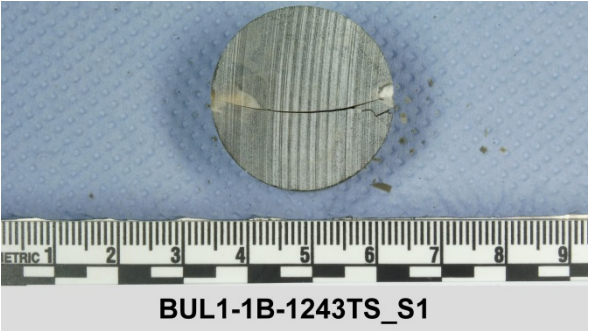
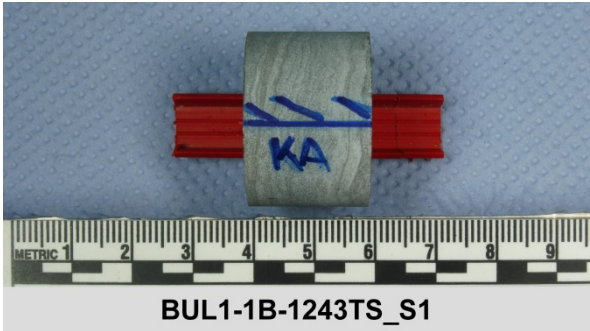
1'194.84 m



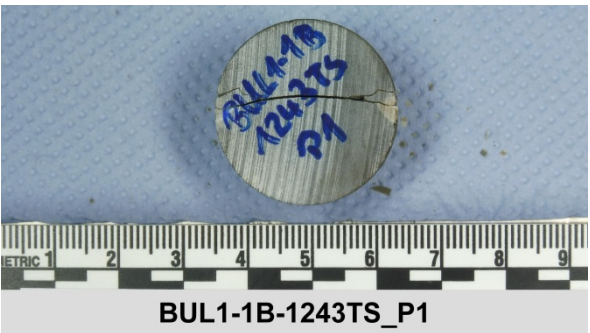
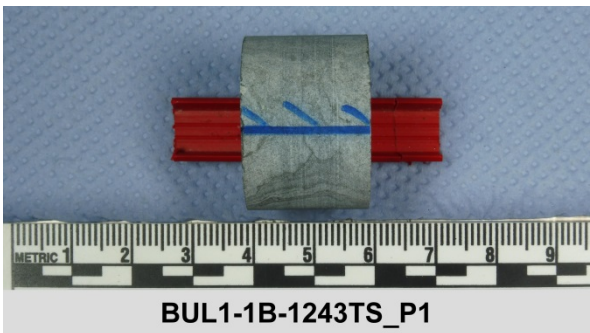
before testing

after testing

1'243.29 m



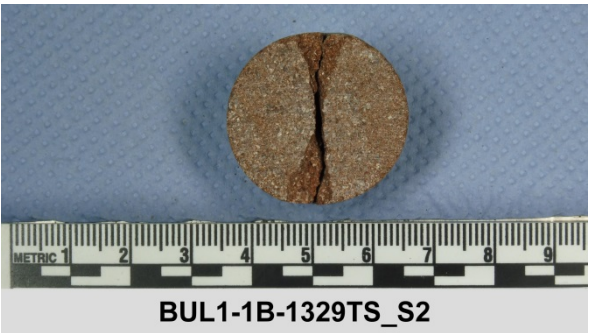
1'243.36 m



1'329.78 m



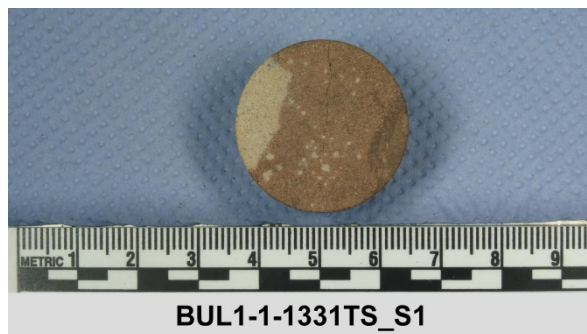
1'330.08 m



before testing

after testing

1'331.58 m



1'331.65 m



UCS (according to well/depth)

BUL1-1

before testing

after testing

559.27 m



559.27 m



587.70 m



587.78 m



before testing

after testing

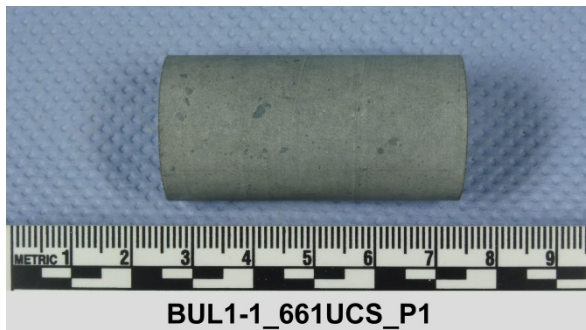
642.60 m



642.61 m



661.34 m



661.34 m



before testing

after testing

665.28 m



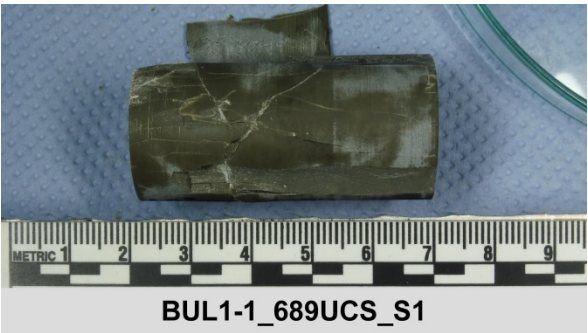
665.30 m



689.51 m

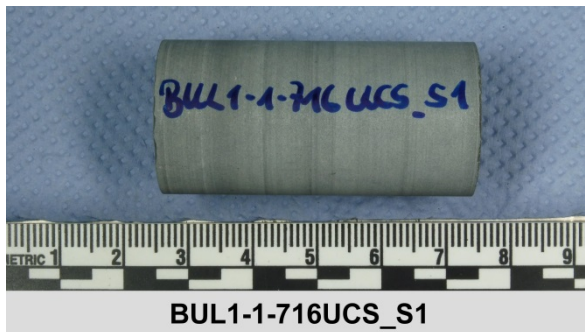


689.55 m



before testing**after testing**

716.39 m



716.48 m



716.64 m



735.12 m



before testing**after testing**

735.39 m



761.67 m



761.84 m



816.64 m



before testing

after testing

816.85 m



846.79 m



846.85 m



UCS (according to well/depth)

BUL1-1B

before testing

after testing

1'054.91 m



1'055.00 m



1'099.79 m



1'099.85 m

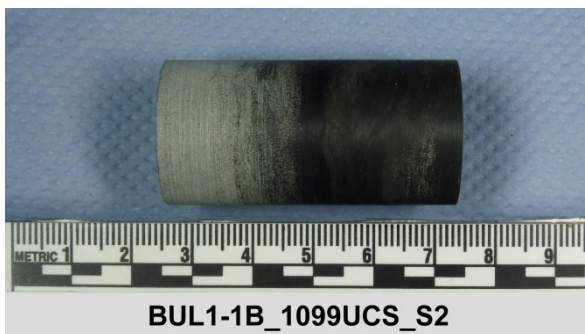


before testing**after testing**

1'099.86 m (failed during mounting)



1'100.04 m



1'172.10 m



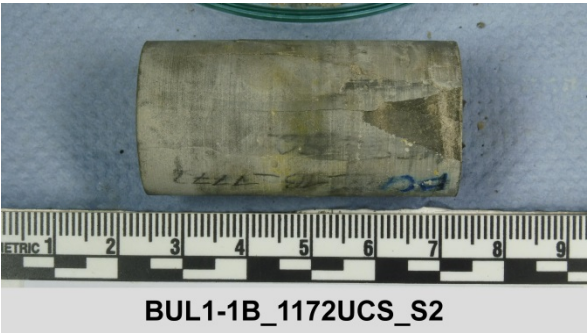
1'172.22 m



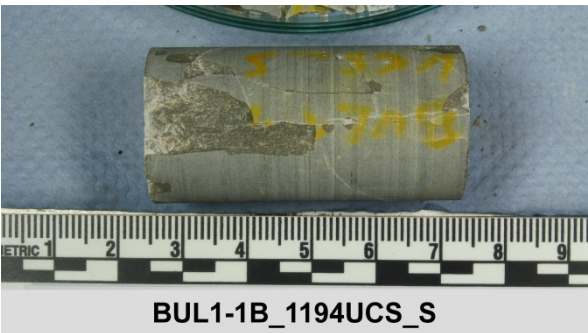
before testing

after testing

l'172.27 m



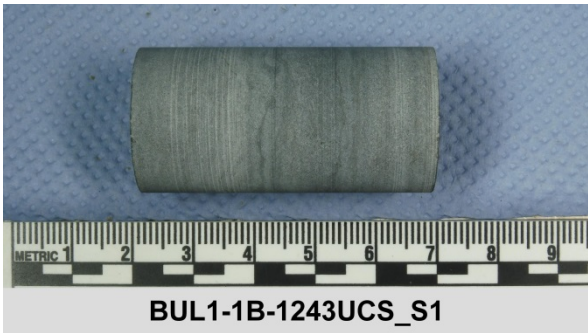
l'194.54 m



l'194.56 m



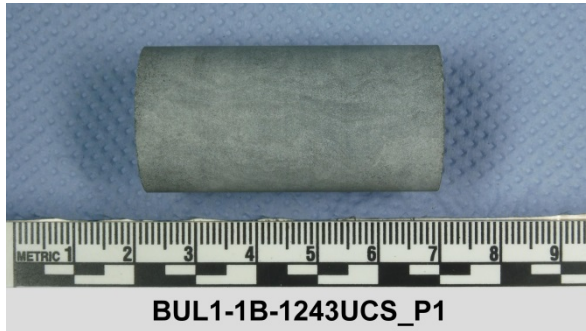
l'243.23 m



before testing

after testing

l'243.41 m



l'329.83 m



l'329.84 m



l'329.99 m



before testing

1'331.51 m



after testing



TRX (according to well/depth)**before testing**

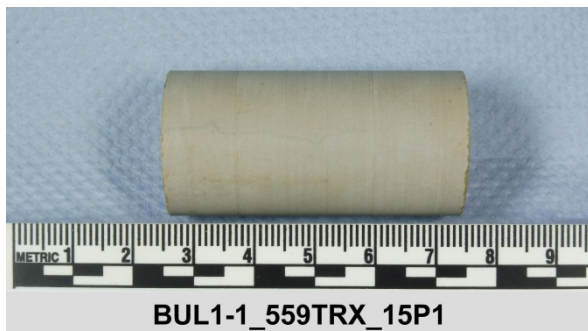
559.41 m

**BUL1-1****after testing**

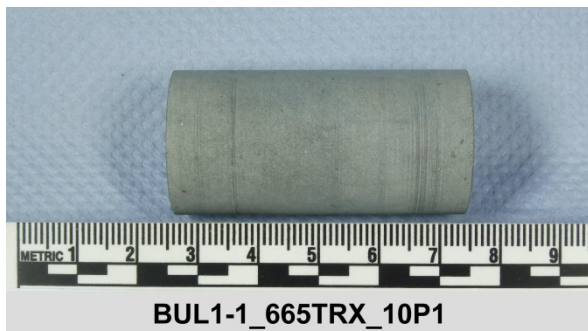
559.46 m



559.46 m

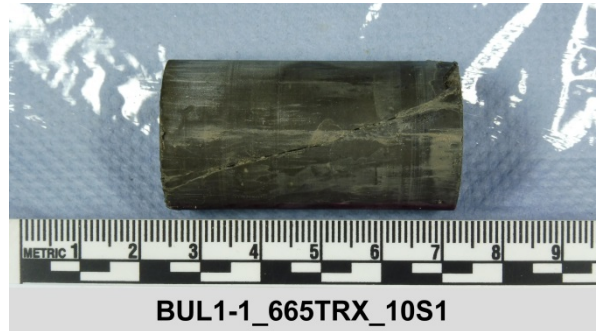


665.38 m



before testing**after testing**

665.40 m



665.42 m



735.15 m (failed)



761.82 m (failed due to a software error)



before testing

after testing

761.84 m



761.86 m



816.69 m



816.69 m



before testing**after testing**

816.82 m



846.85 m



846.92 m



846.92 m



TRX (according to well/depth)

before testing

1'243.23 m



BUL1-1B

after testing



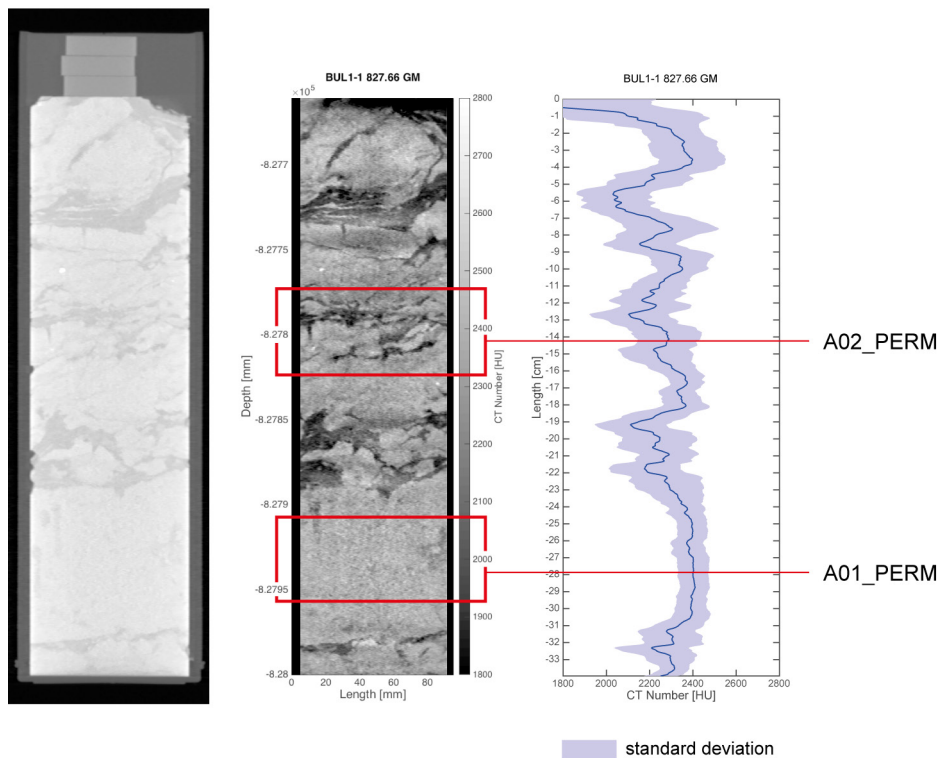
1'331.51 m



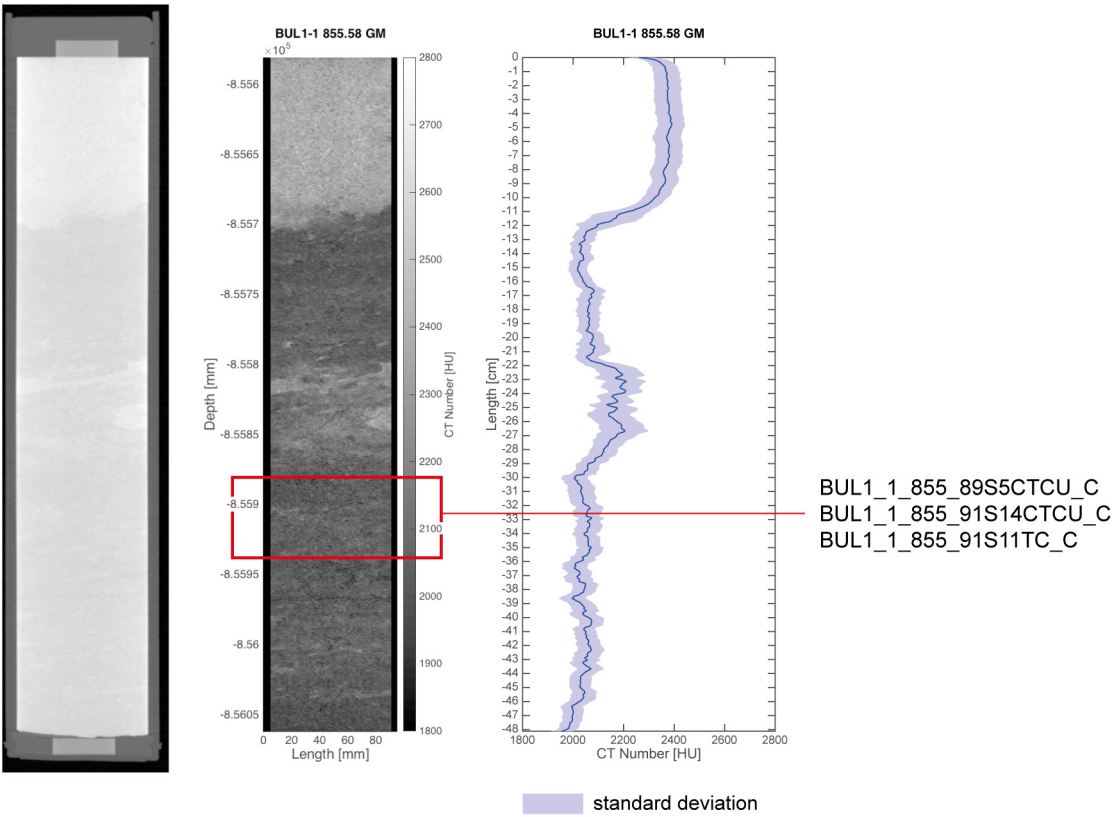
Appendix B: XCT cross-sections with selection of test specimens

The computed tomography scans of the tested cores are here reported, and the sections used in the testing programme are indicated.

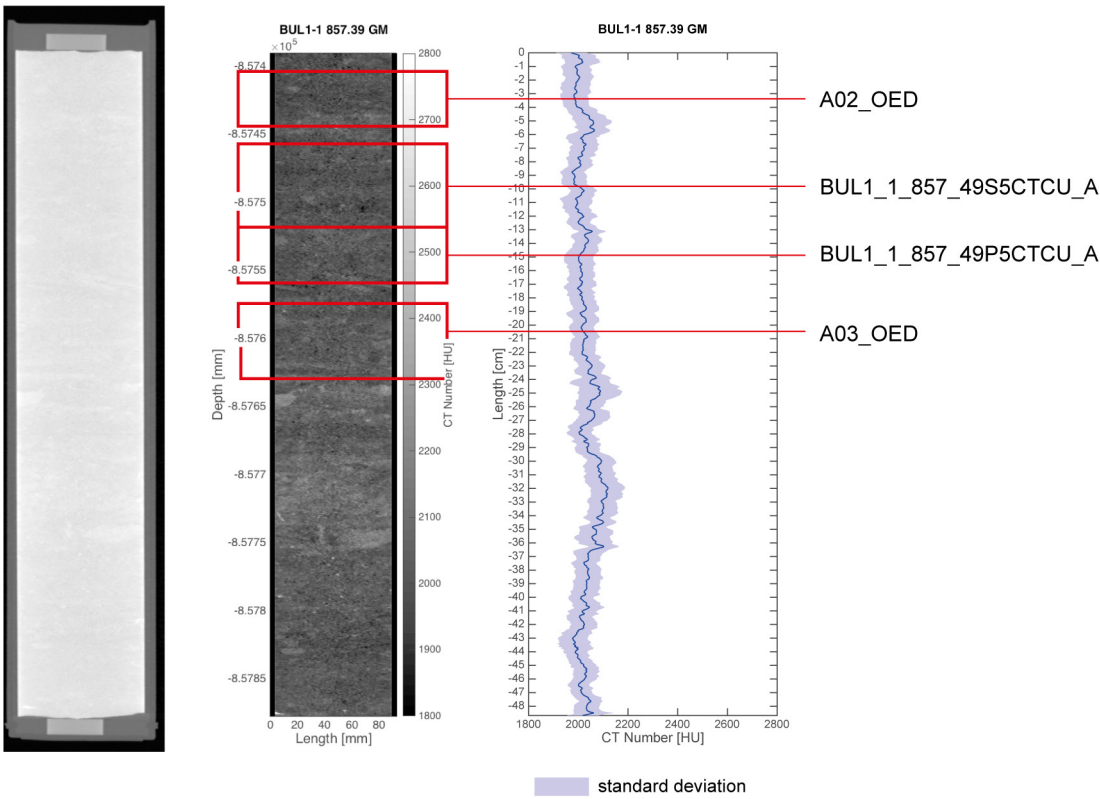
BUL1-1 827.66/35 GM



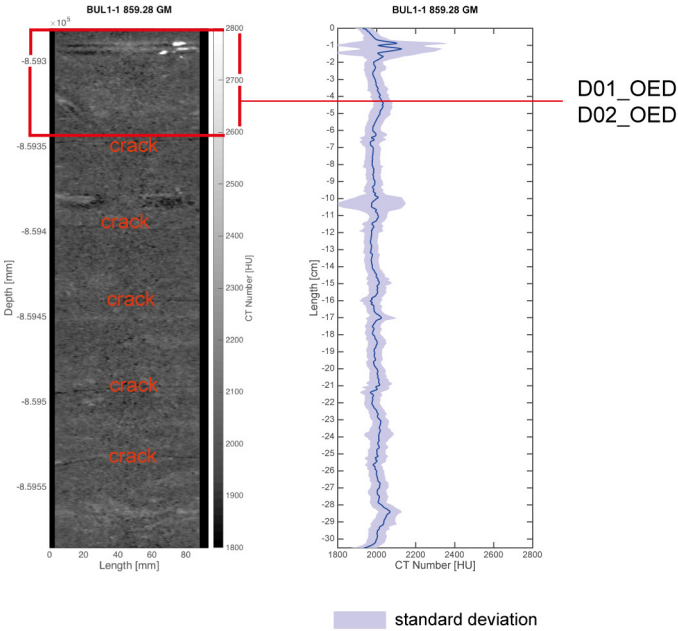
BUL1-1 855.58/49 GM



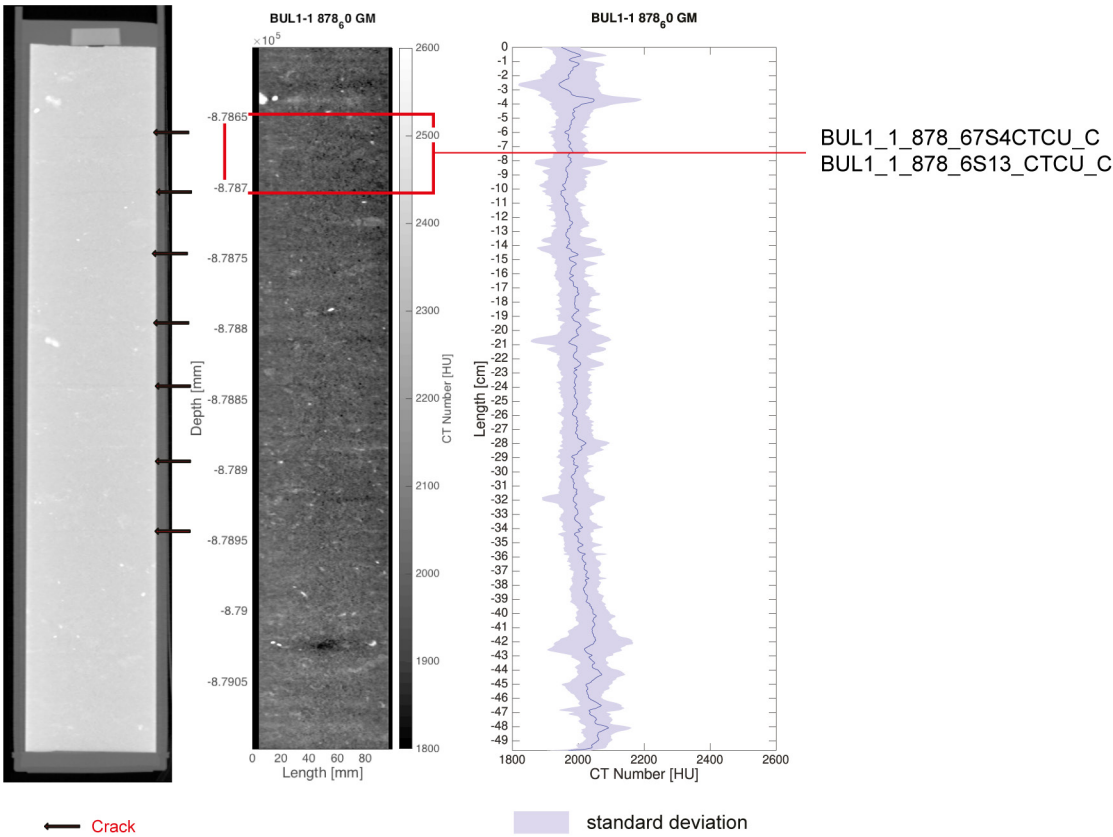
BUL1-1 857.39/40 GM



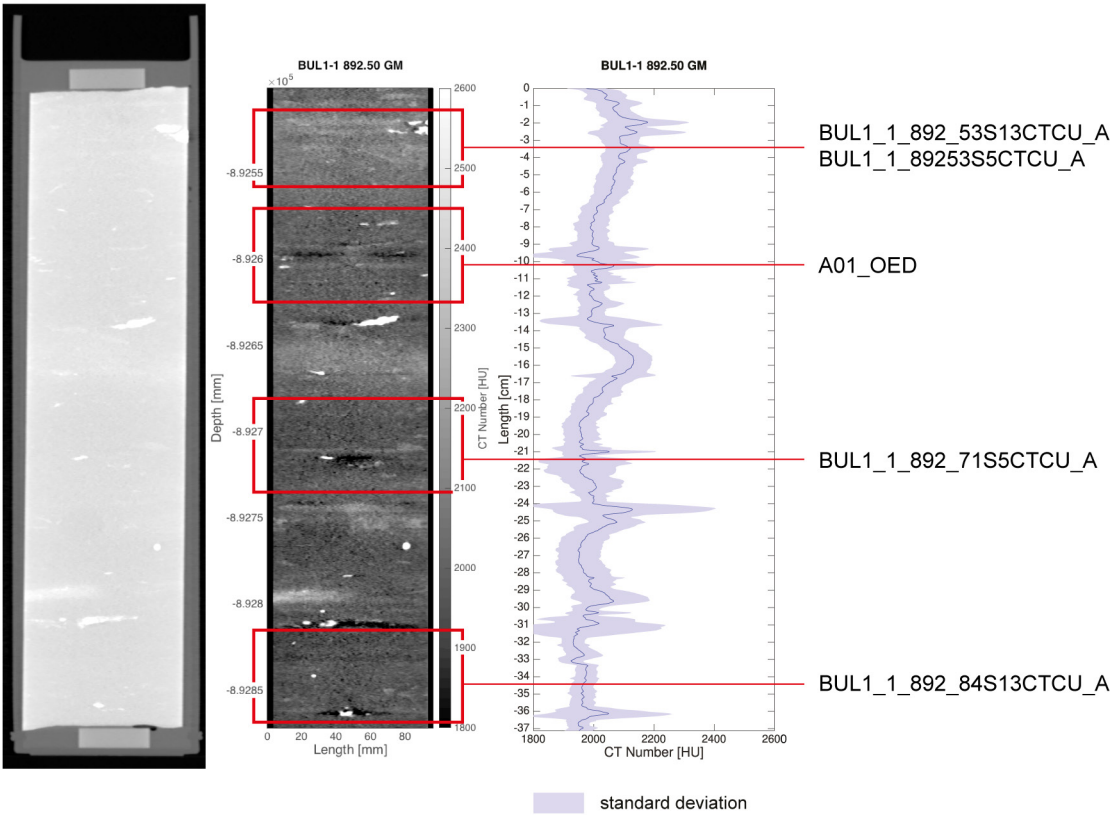
BUL1-1 859.28/31 GM



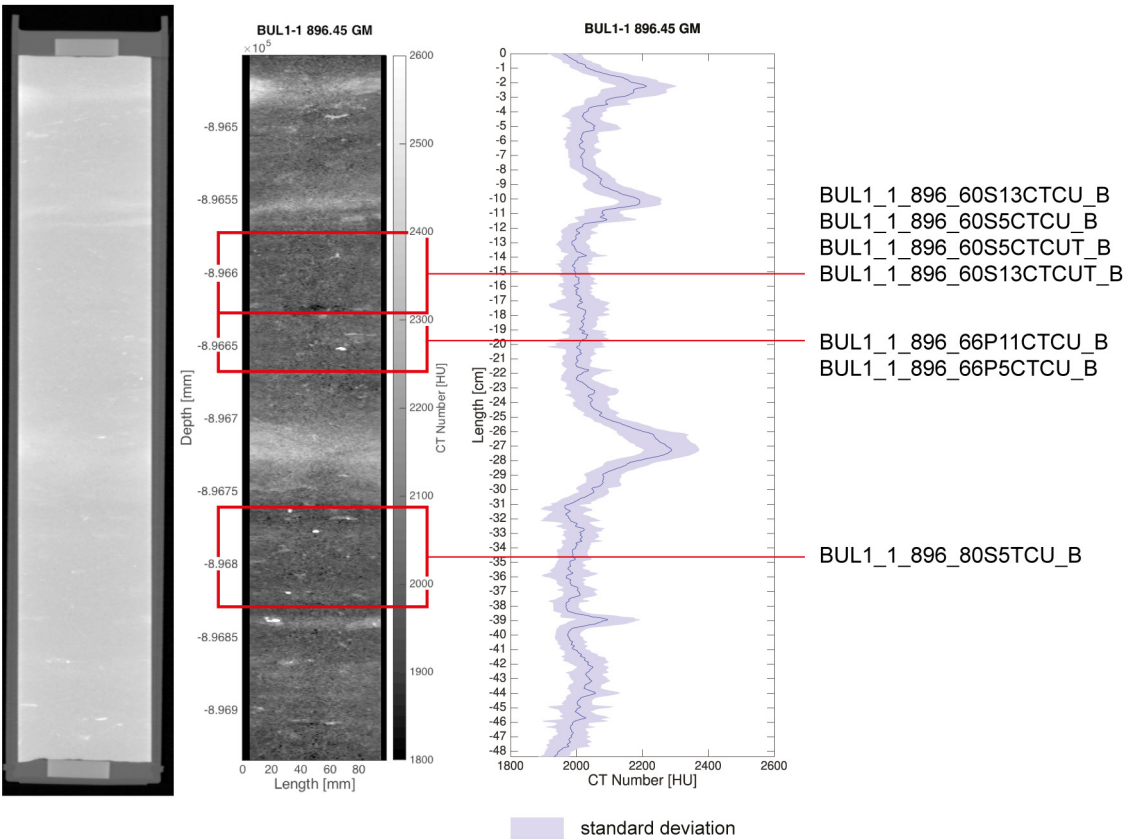
BUL1-1 878.60/50 GM



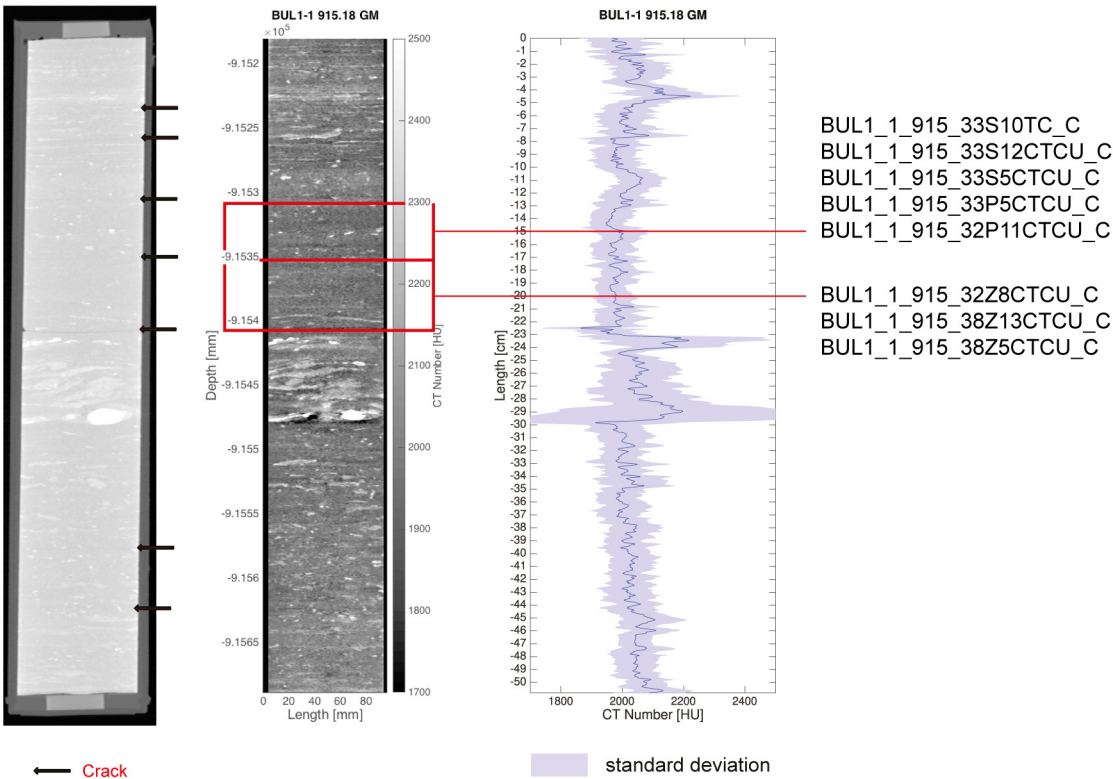
BUL1-1 892.50/38 GM



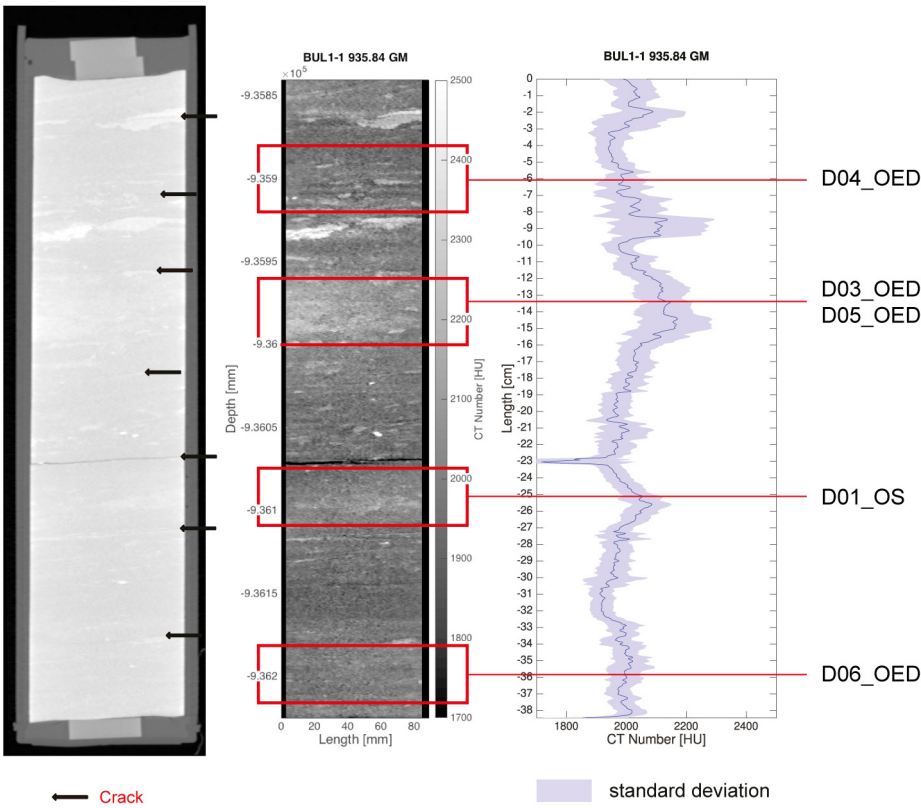
BUL1-1 896.45/49 GM



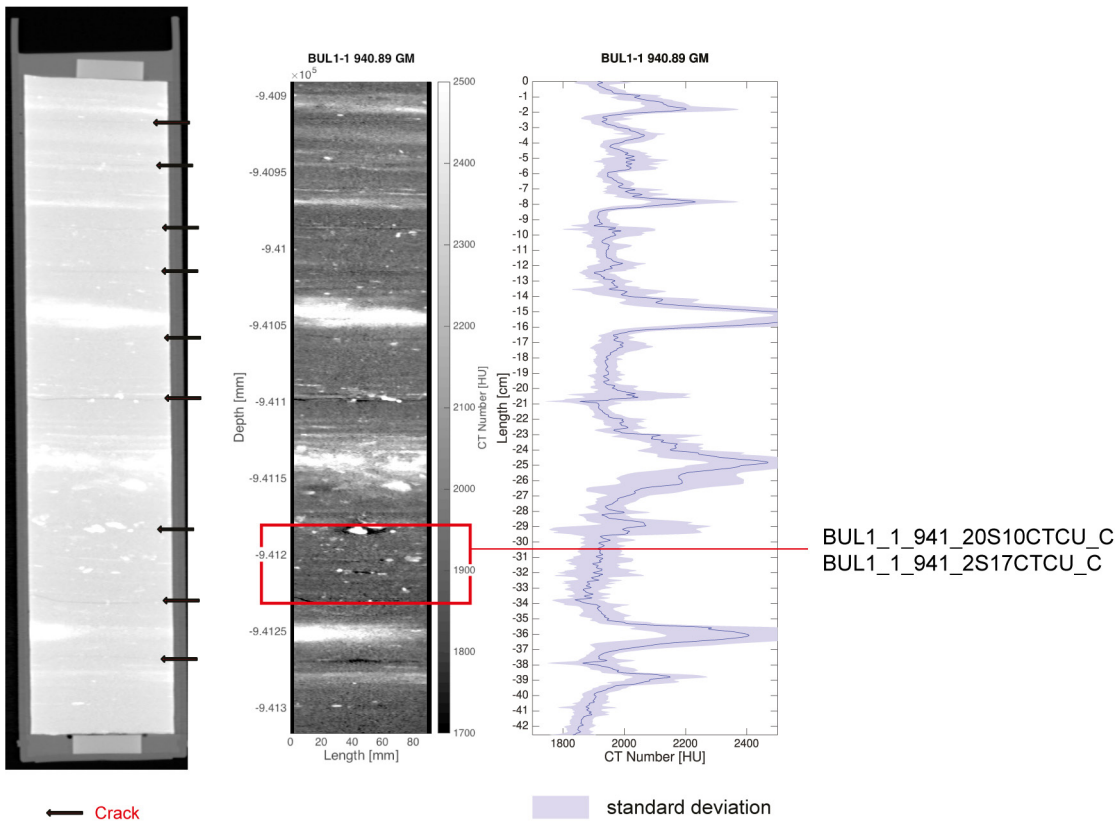
BUL1-1 915.18/51 GM



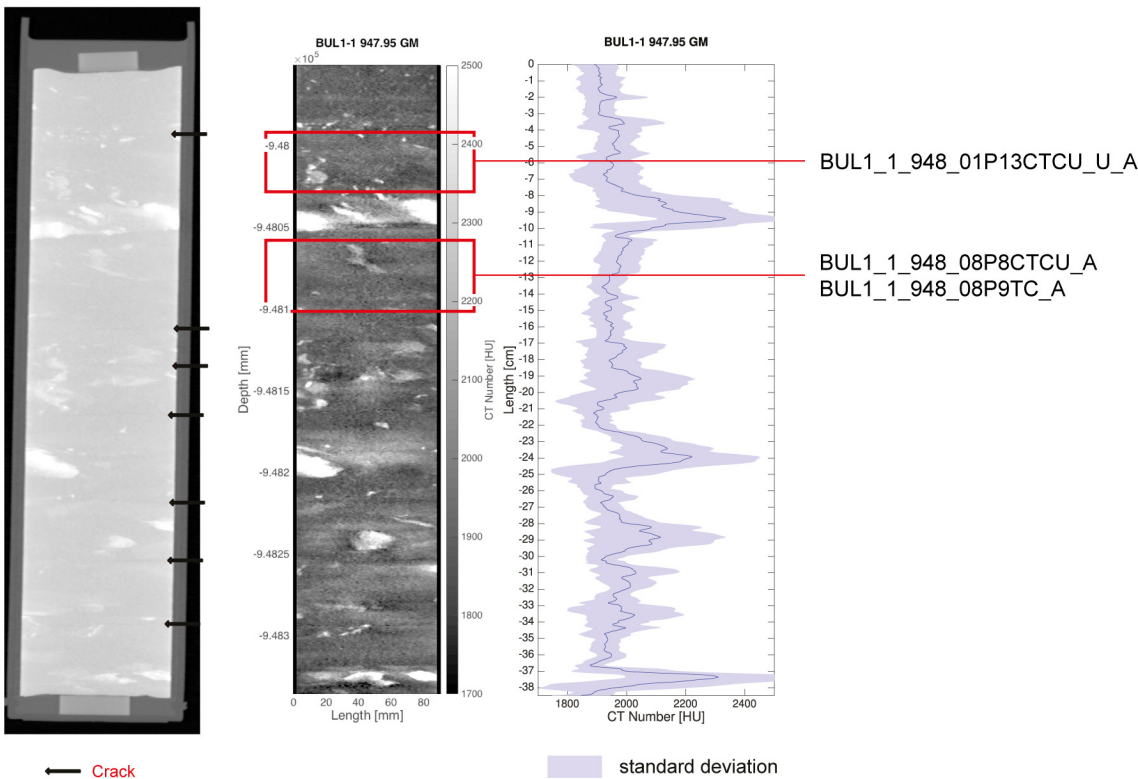
BUL1-1 935.84/39 GM



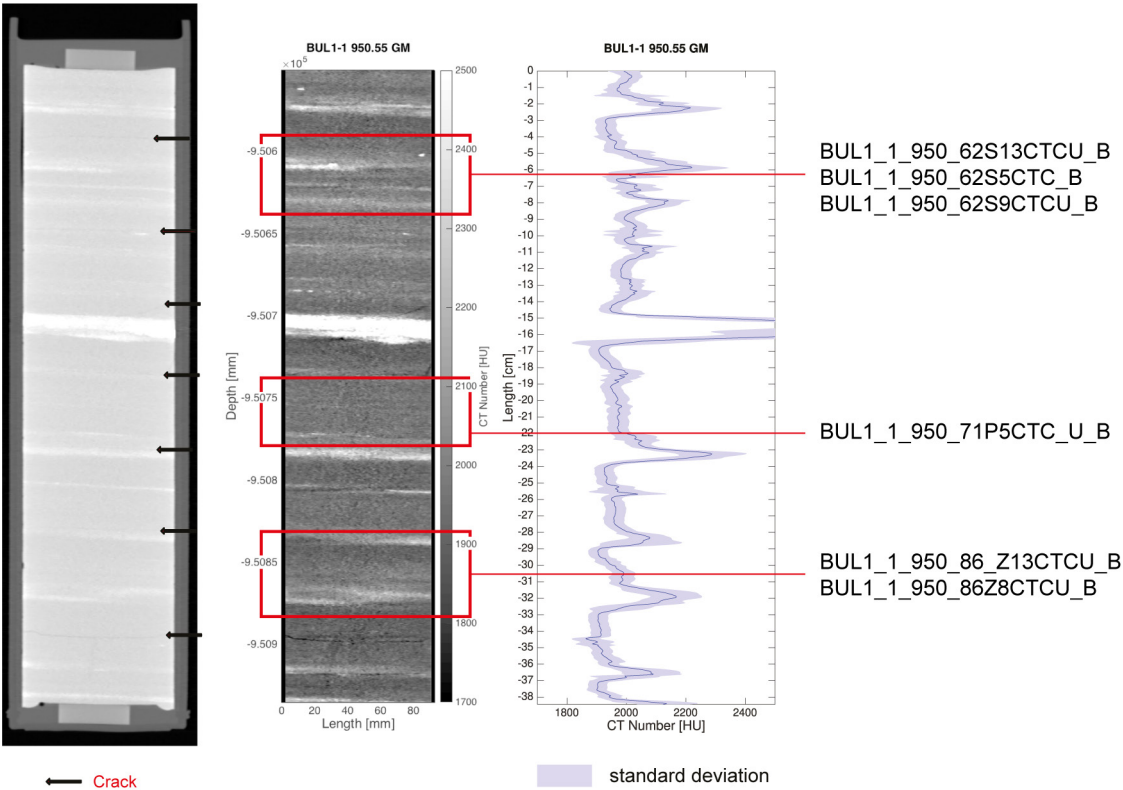
BUL1-1 940.89/43 GM



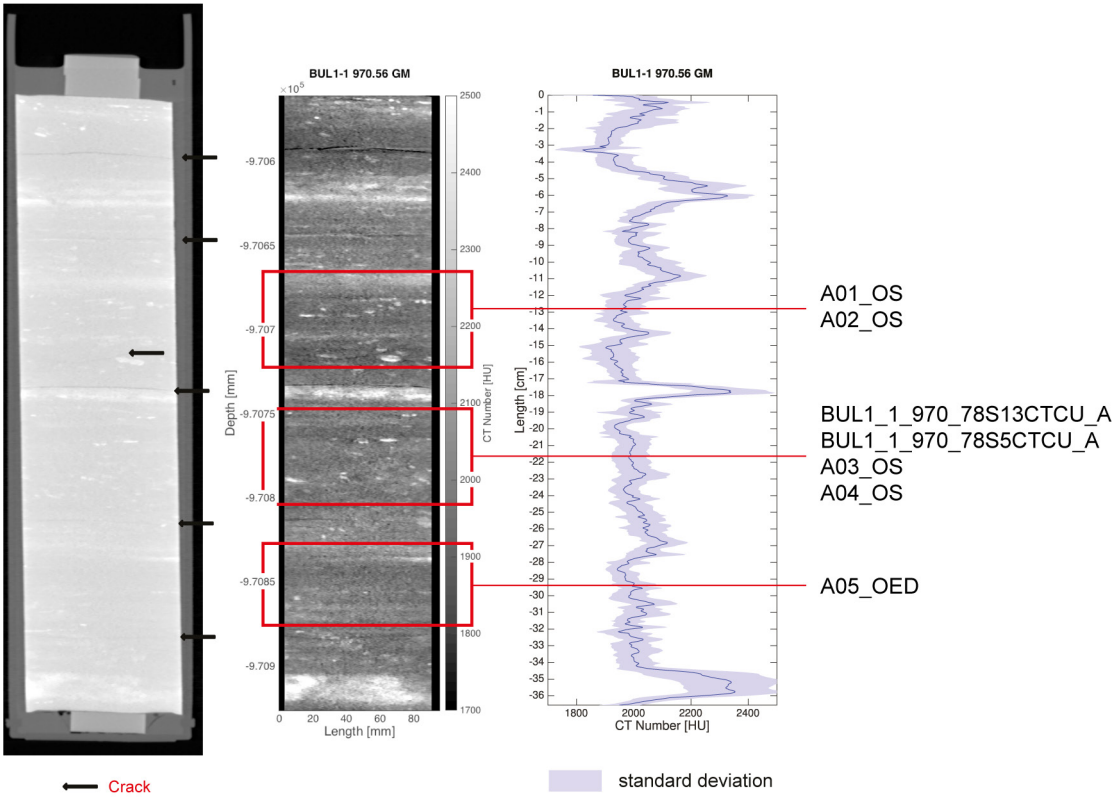
BUL1-1 947.95/39 GM



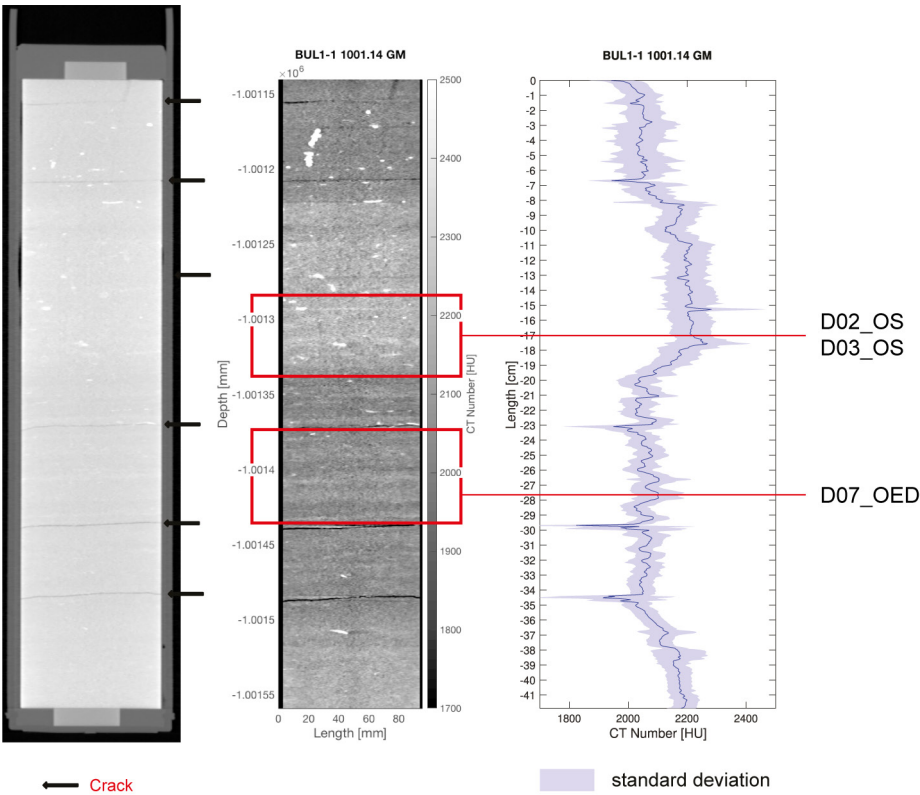
BUL1-1 950.55/40 GM



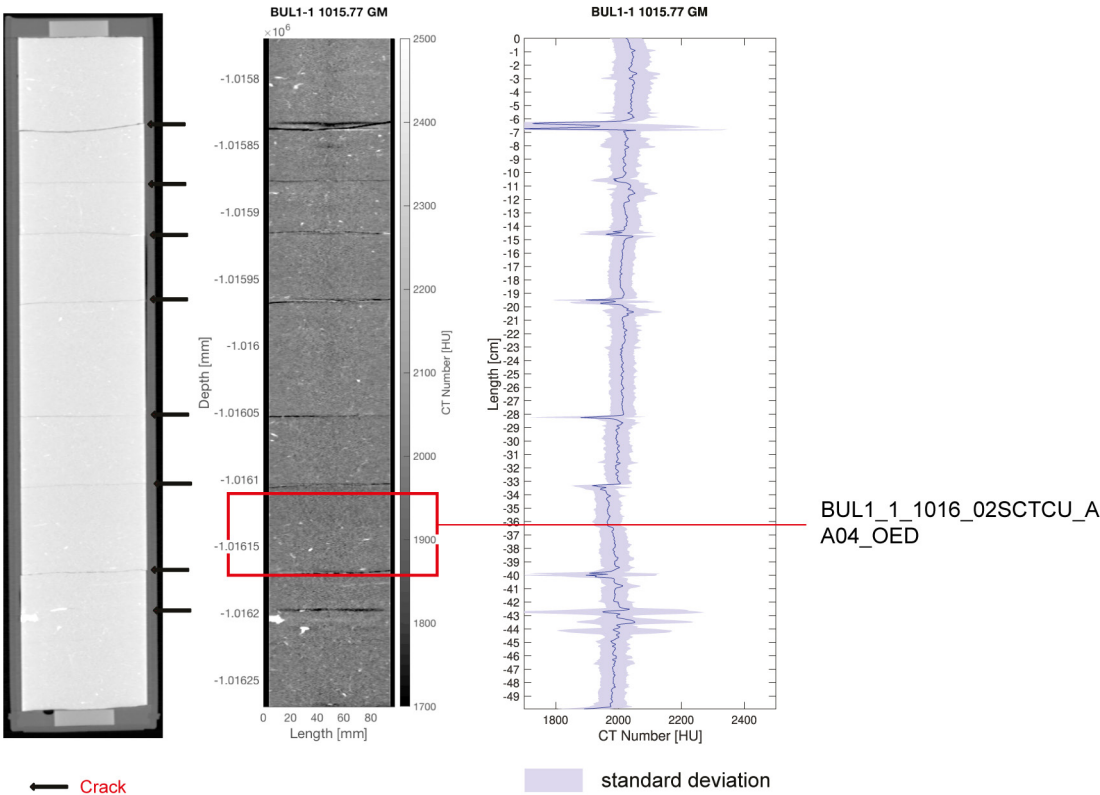
BUL1-1 970.56/37 GM



BUL1-1 1001.14/43 GM



BUL1-1 1015.77/50 GM

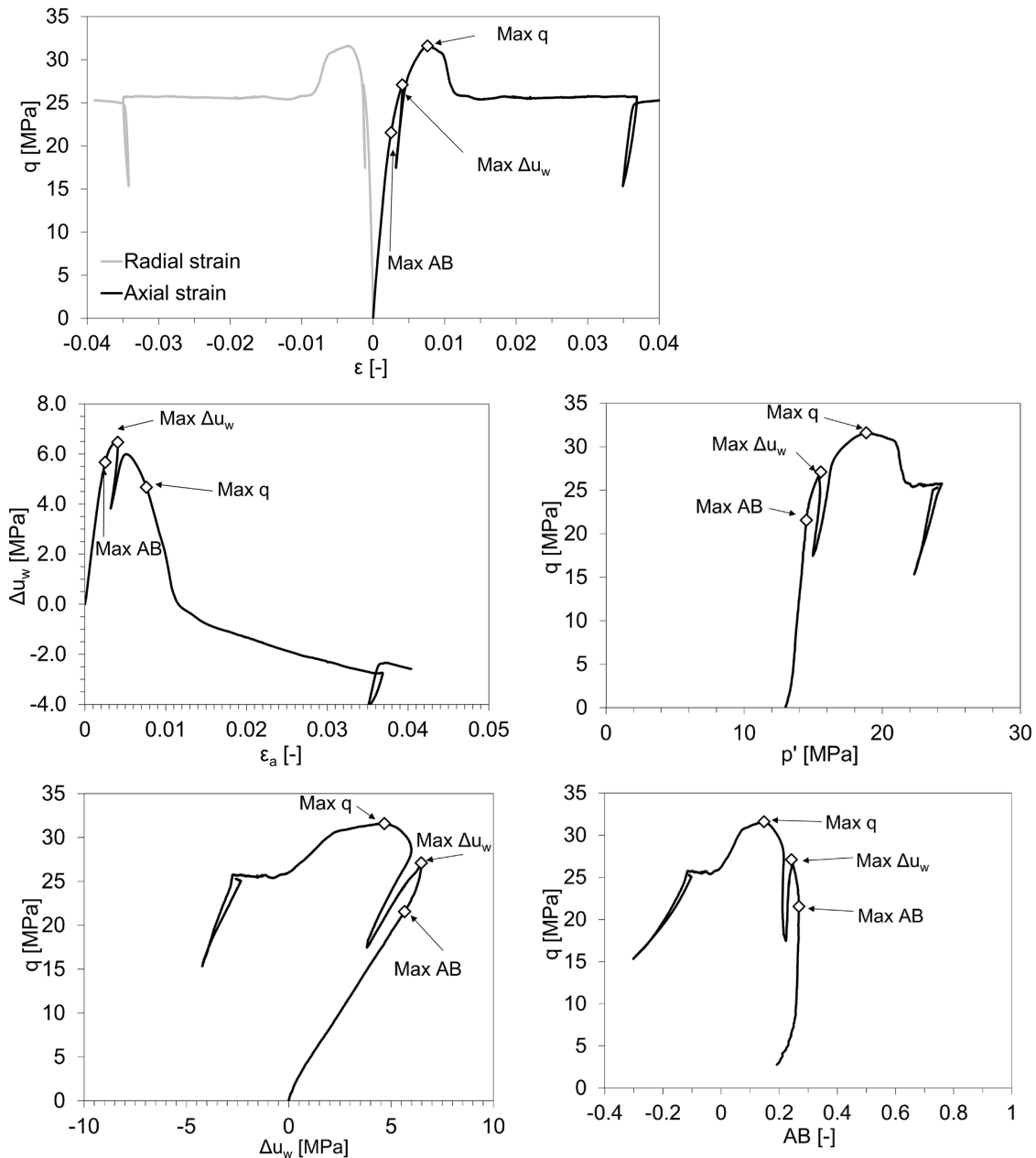


BUL1_1_1016_02SCTCU_A
A04_OED

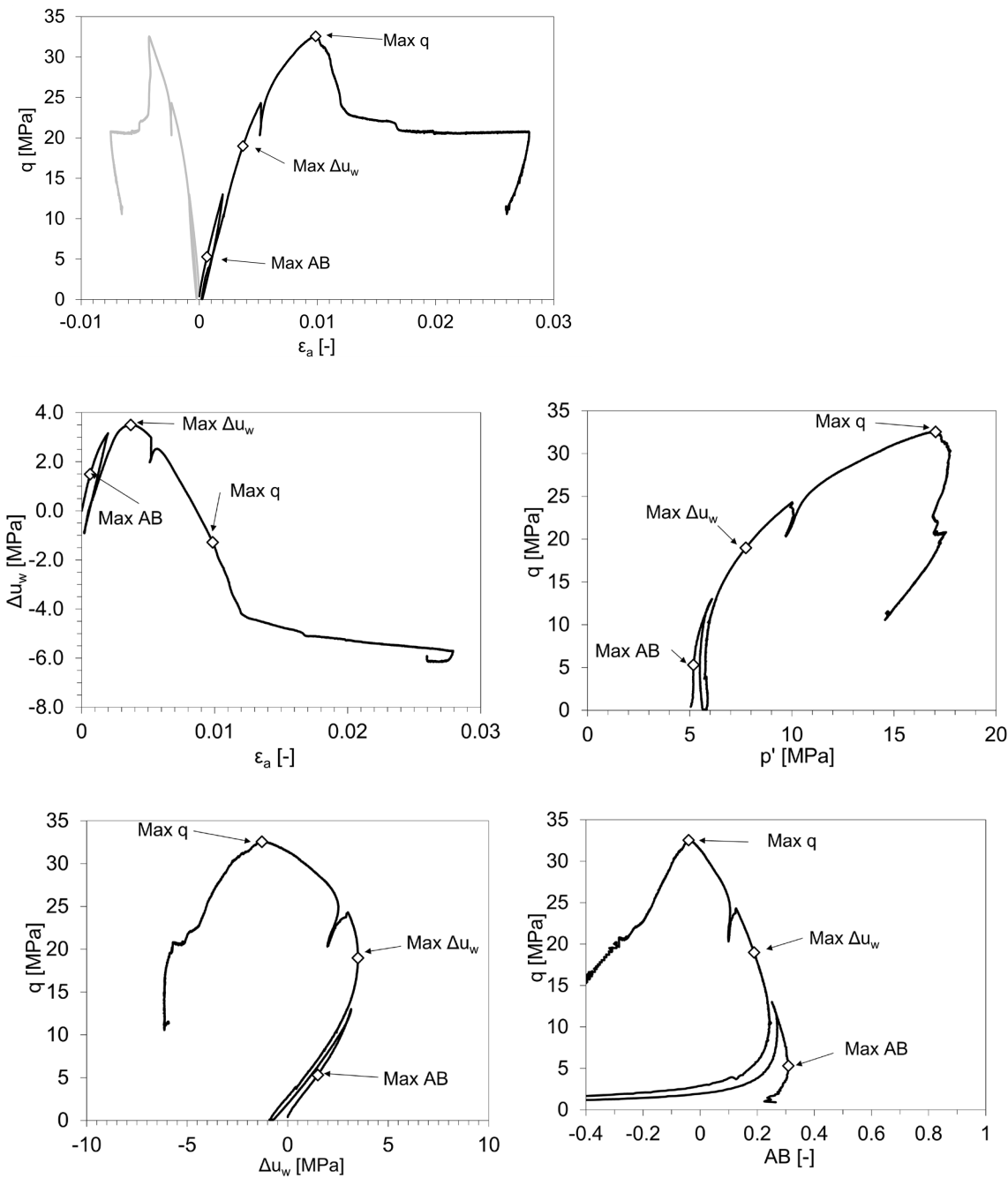
Appendix C: Diagnostic plots of triaxial tests (geomechanical testing programme)

For each triaxial tests, the diagnostic plots are here reported. The plots include: the evolution of axial and radial strains with deviatoric stress q , the pore pressure change Δu_w during shearing versus the axial strain ε_a , the stress path in the p' - q plane, the pore pressure variation Δu_w and the AB parameter evolution with q . In all the plots, the conditions at which the maximum deviatoric stress (Max q), the maximum pore pressure change (Max Δu_w) and the maximum AB (Max AB) (excluding evolution during unload/reload loops) are marked, as mentioned in Section 6.2.3.4 Testing conditions are reported in Table 6-2.

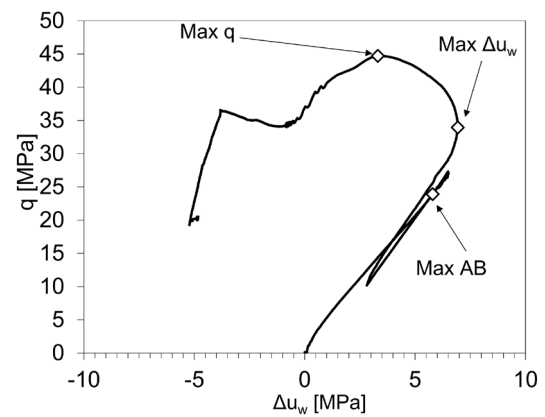
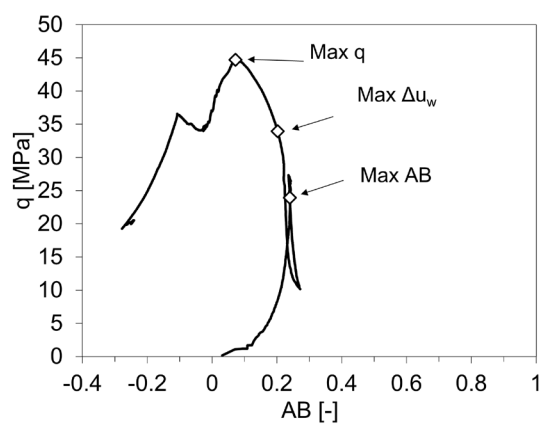
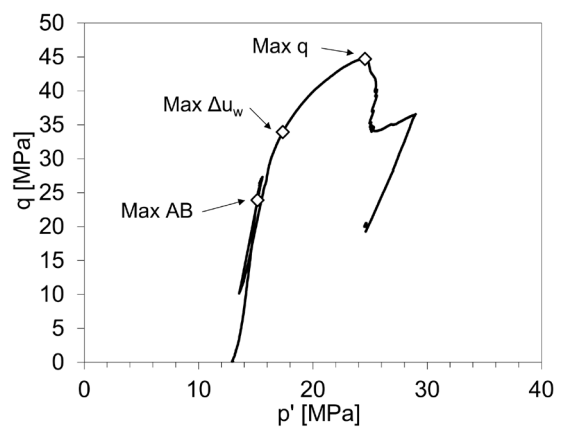
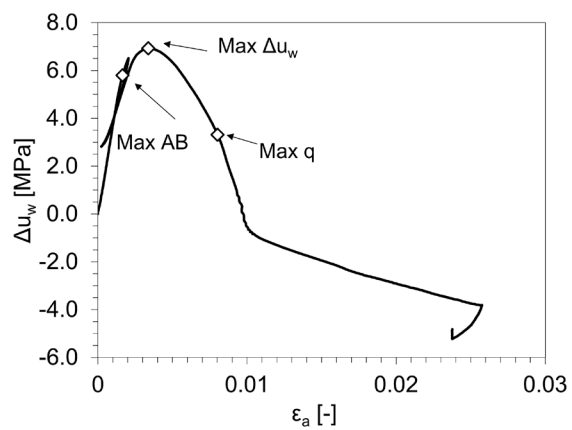
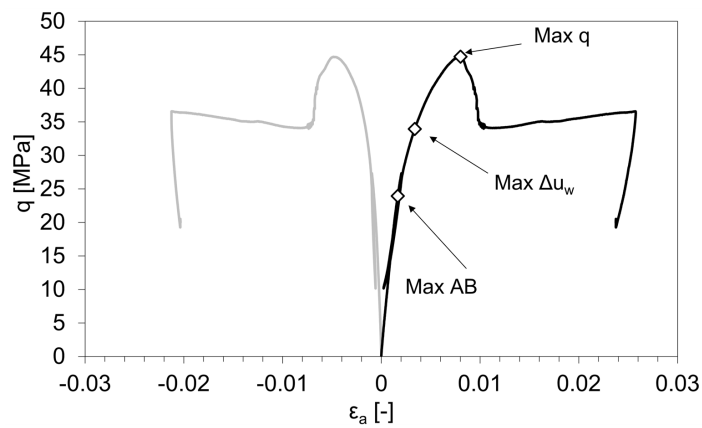
BUL1-1_892_84S13CTCU_A



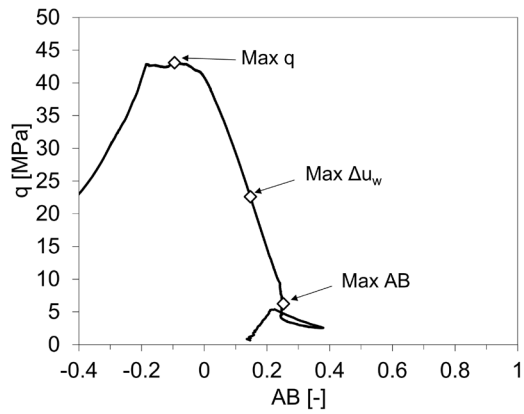
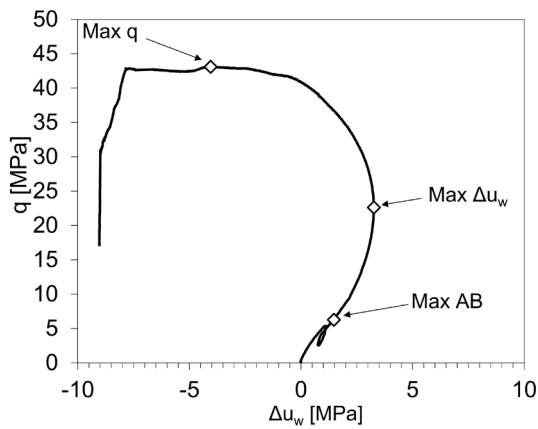
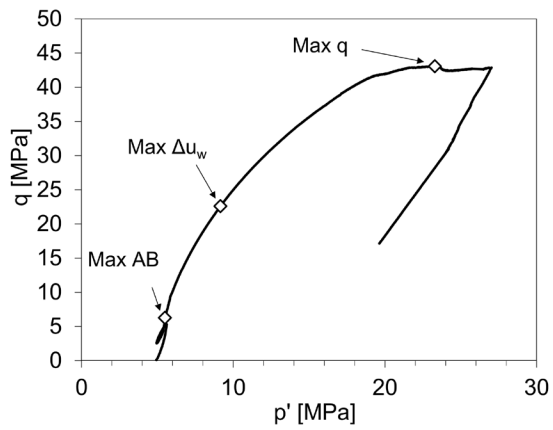
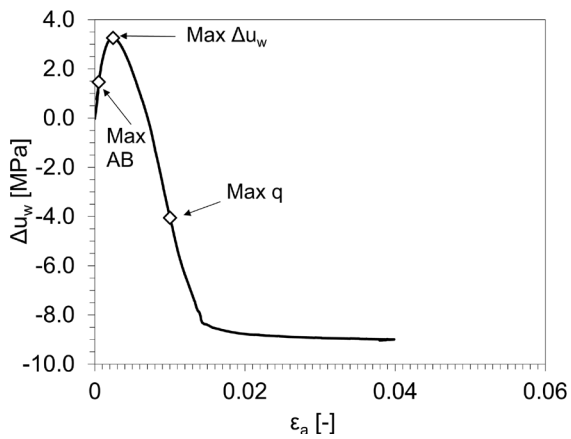
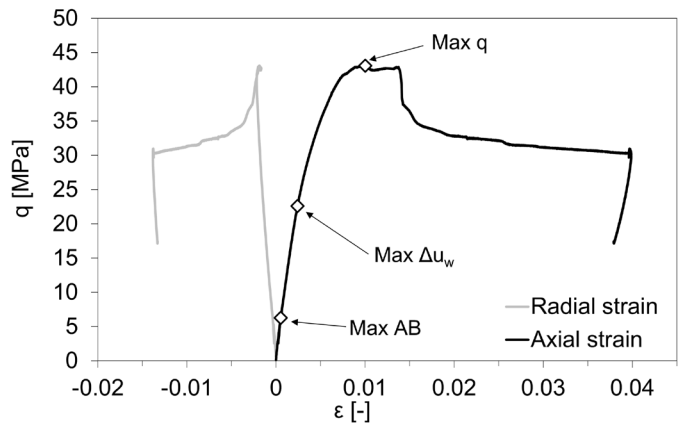
BUL1-1_892_71S5CTCU_A



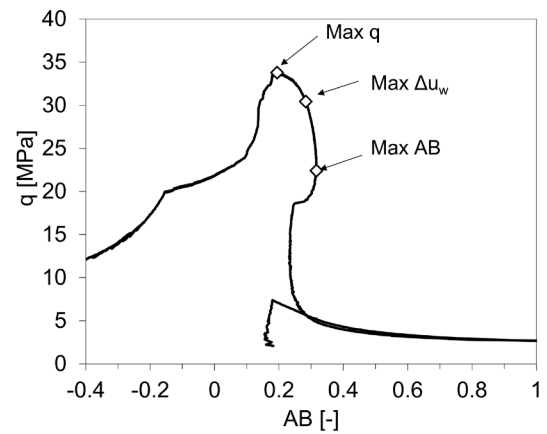
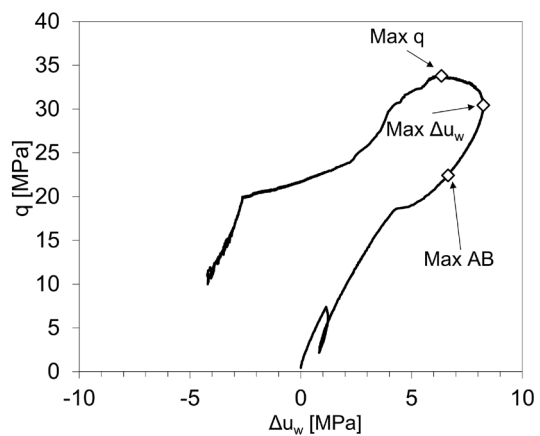
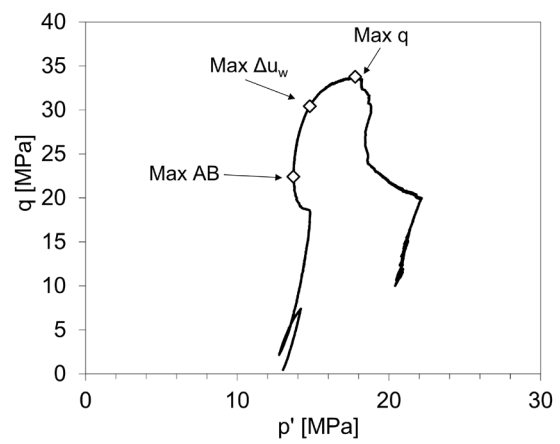
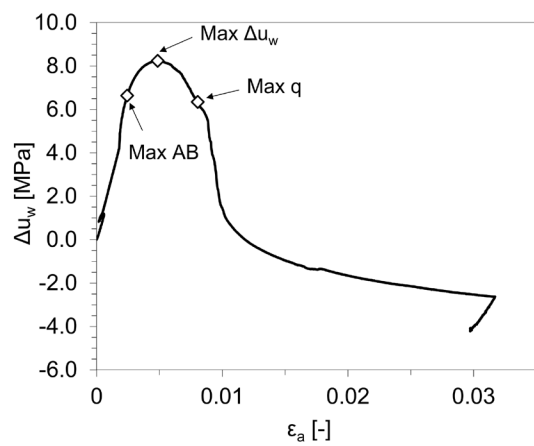
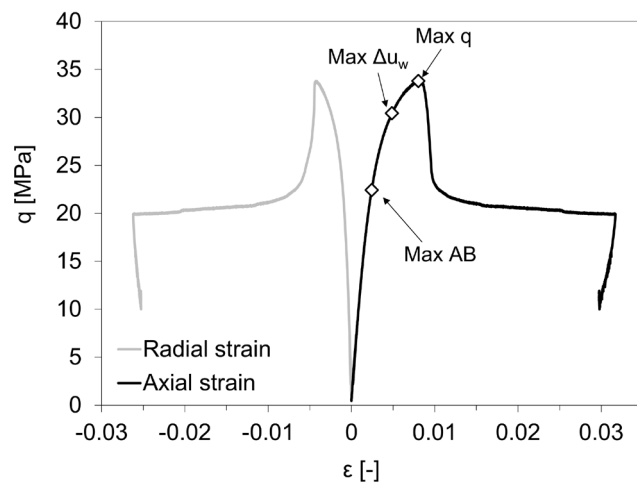
BUL1-1_892_53S13CTCU_A



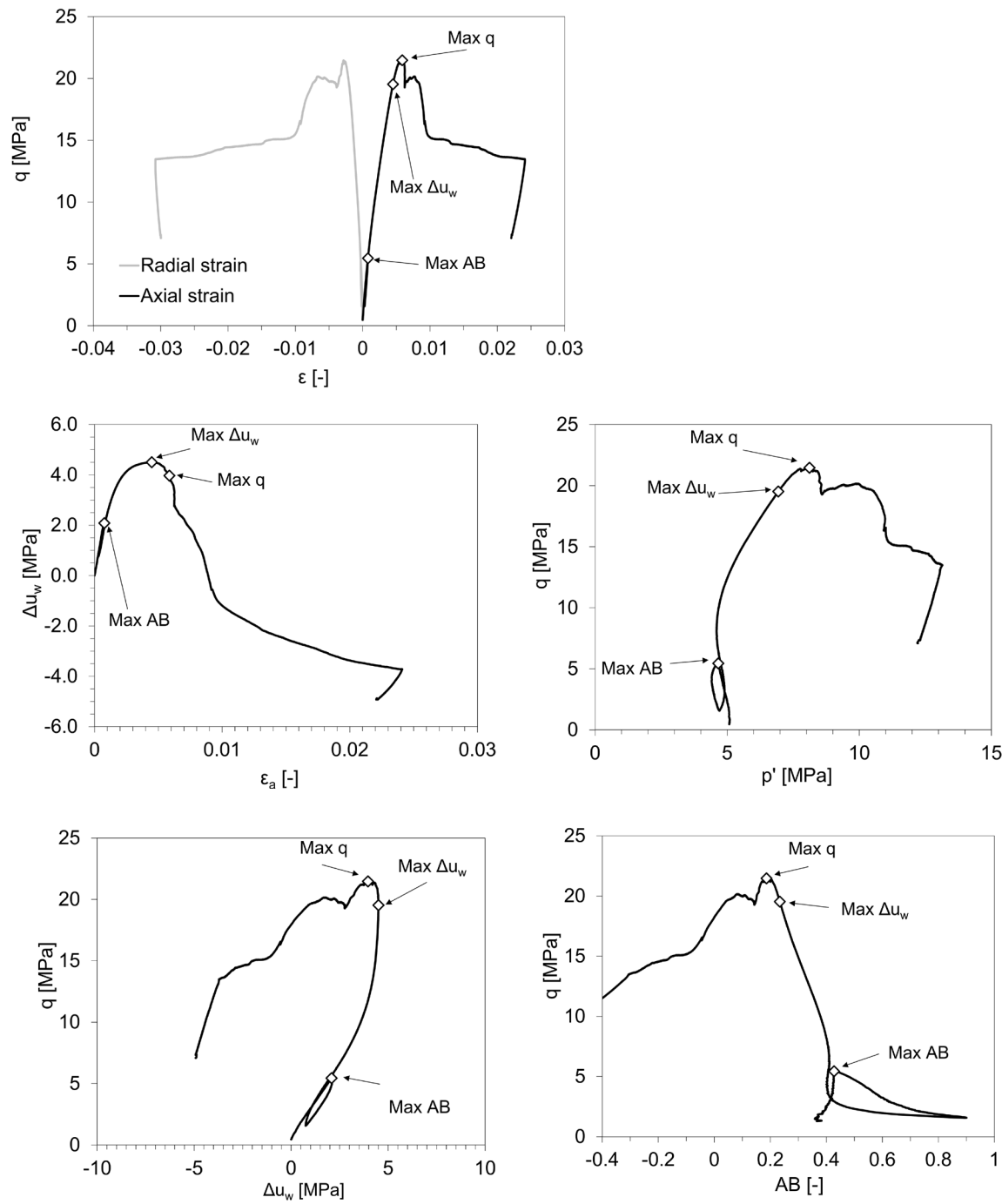
BUL1-1_89253S5CTCU_A



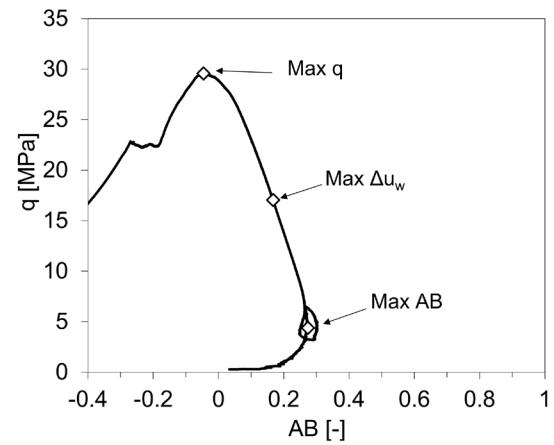
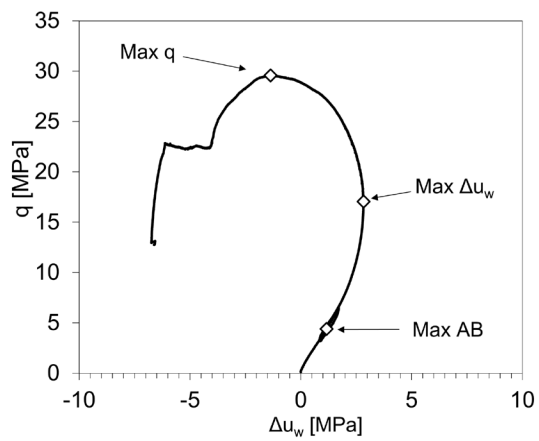
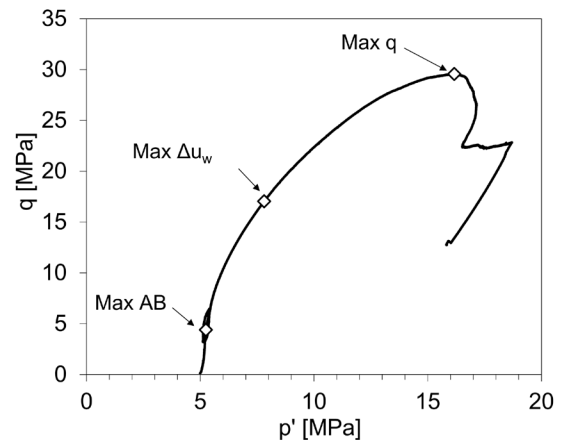
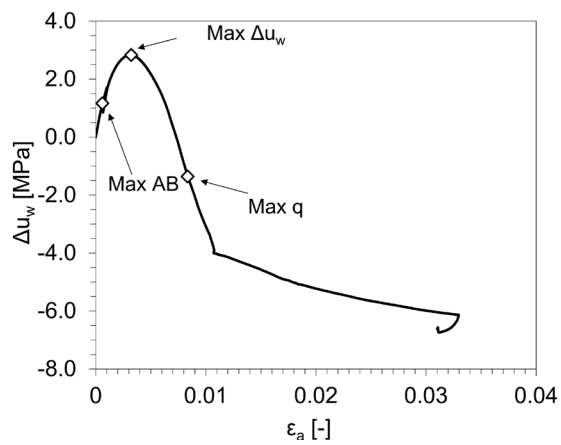
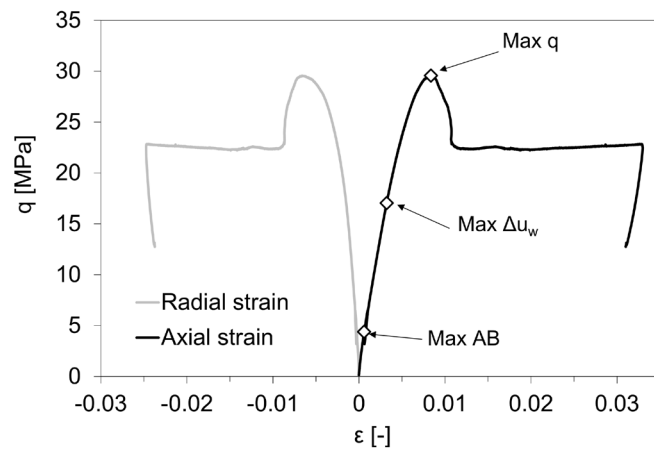
BUL1-1_970_78S13CTCU_A



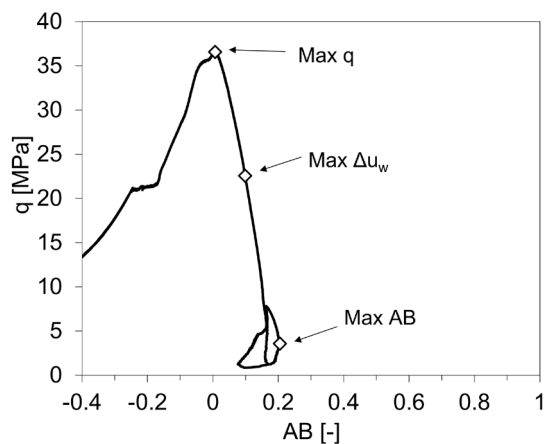
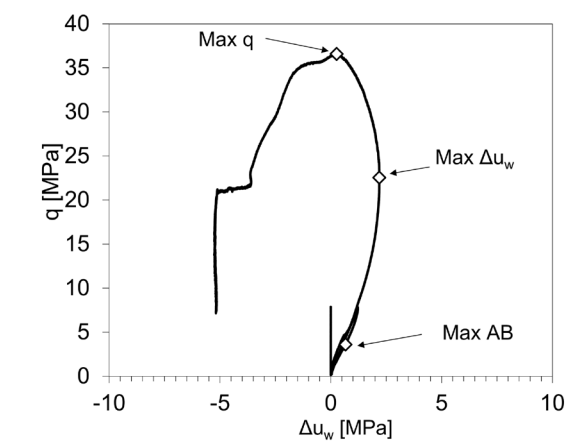
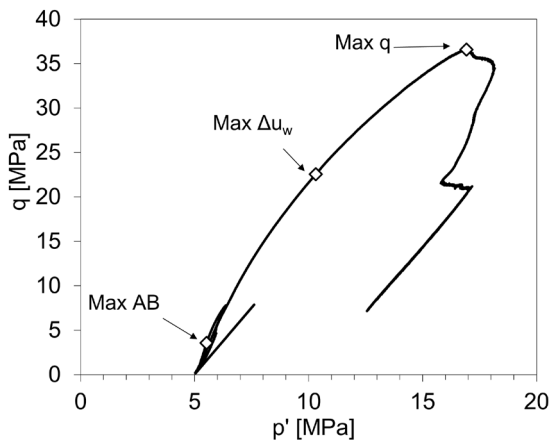
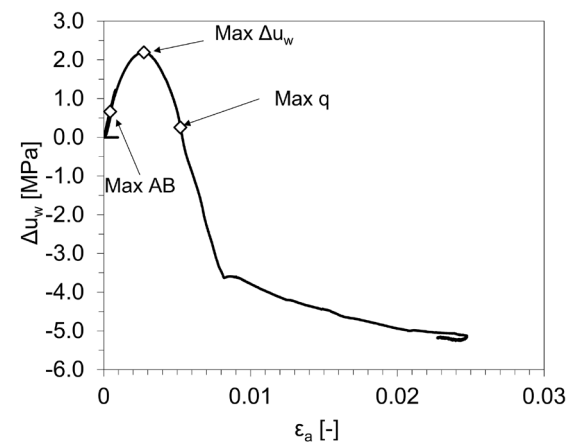
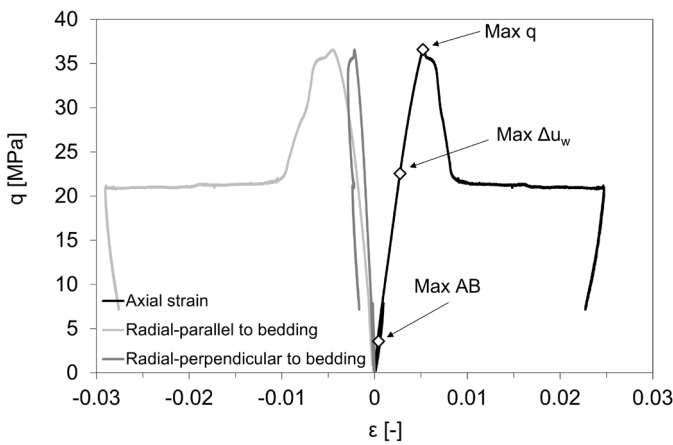
BUL1-1_970_78S5CTCU_A



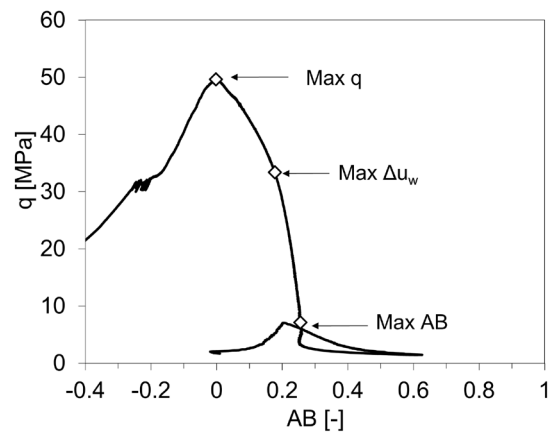
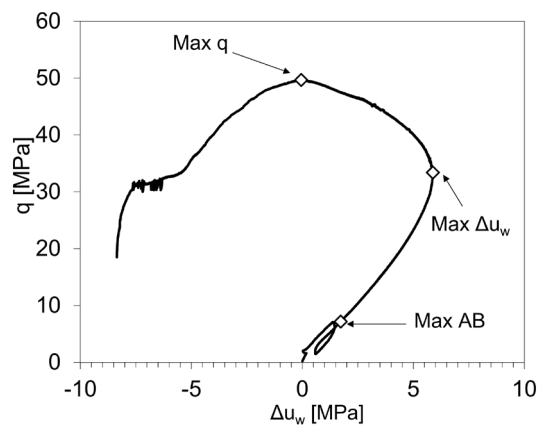
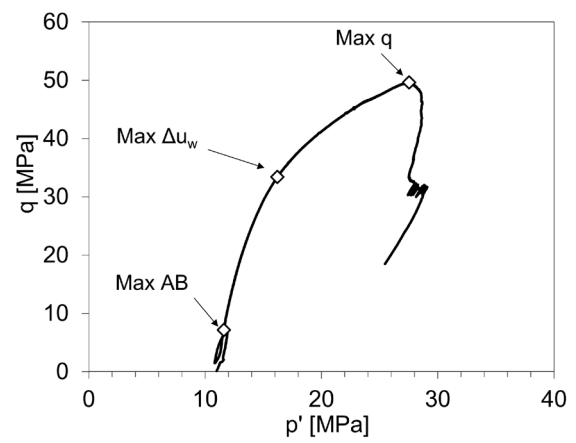
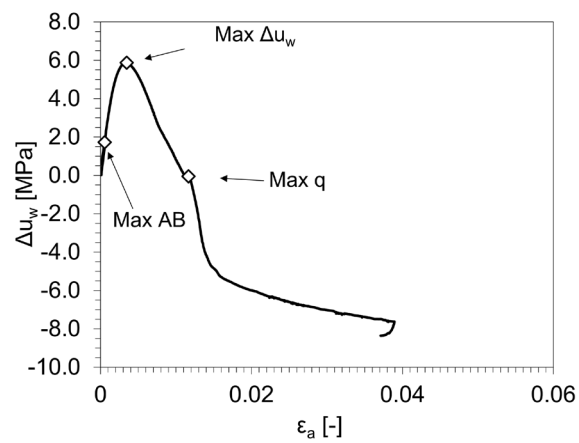
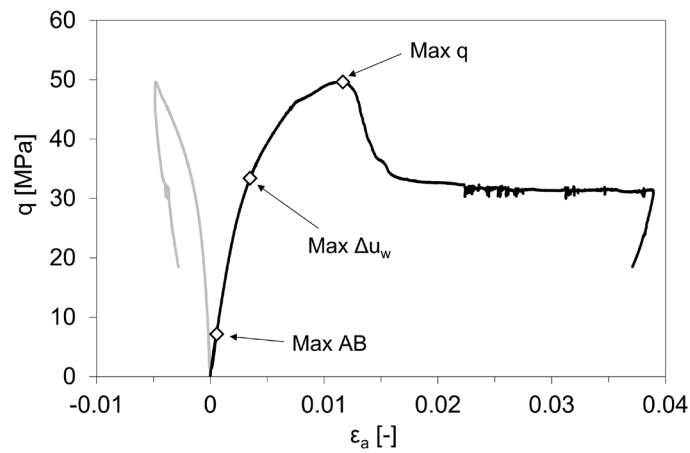
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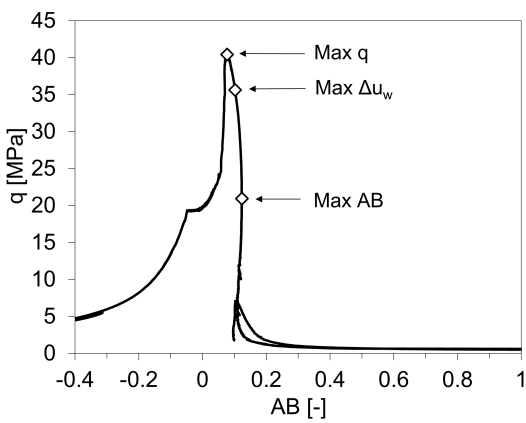
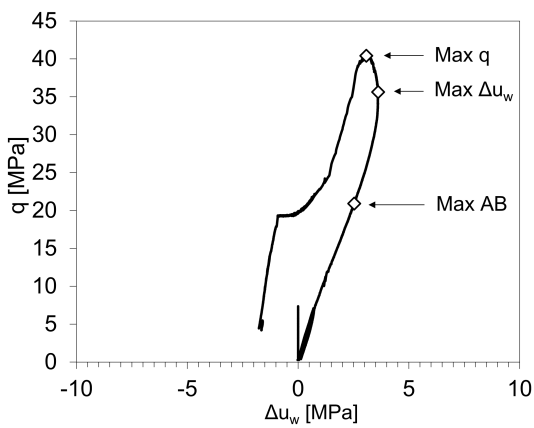
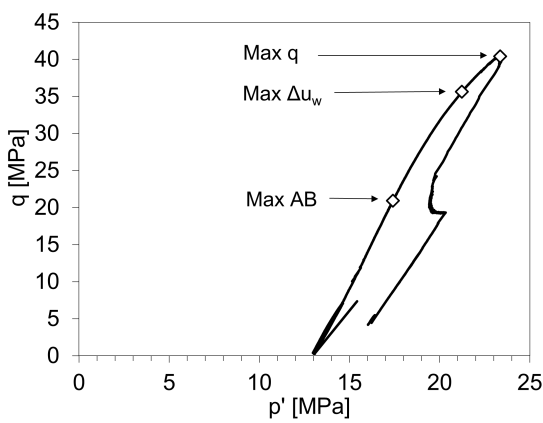
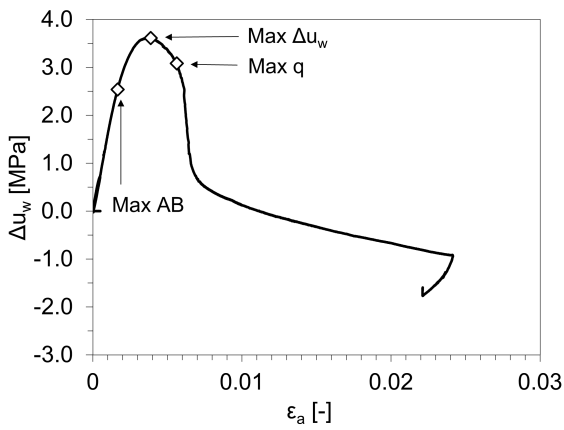
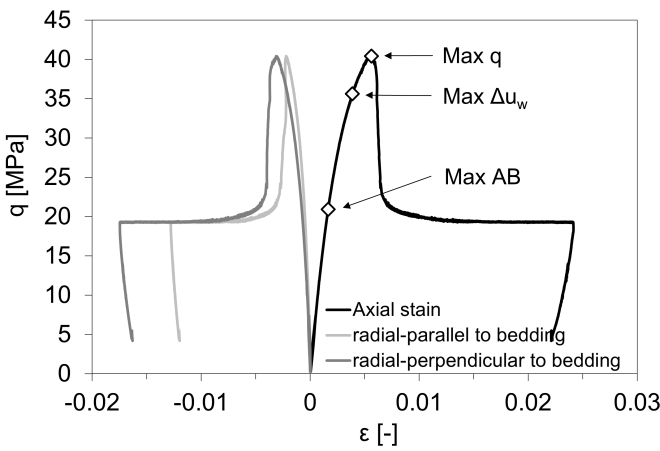
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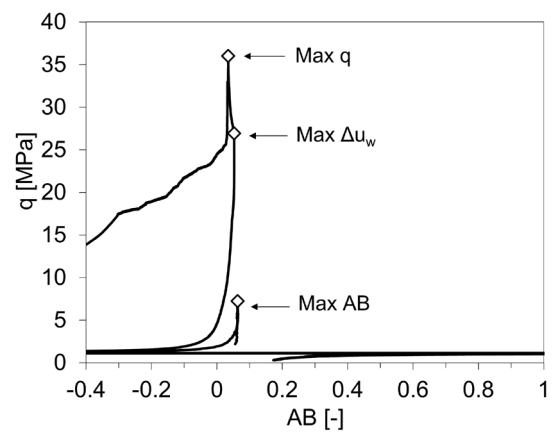
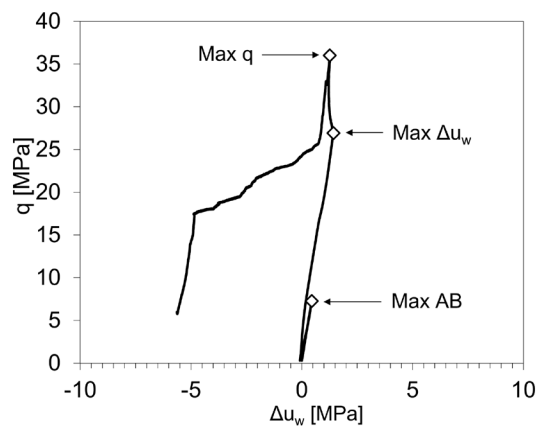
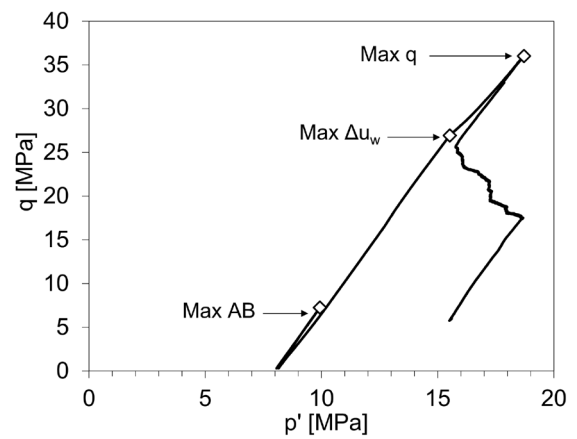
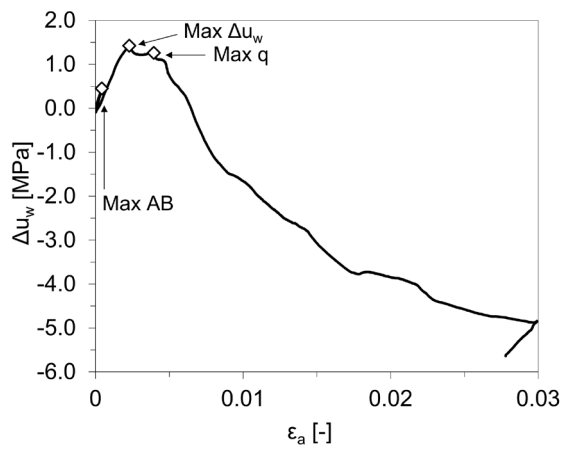
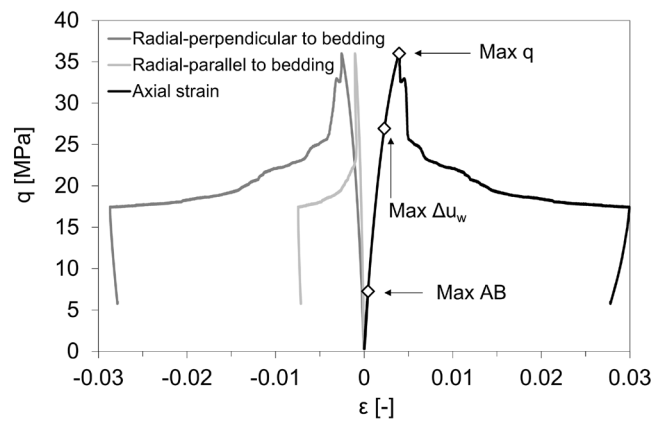
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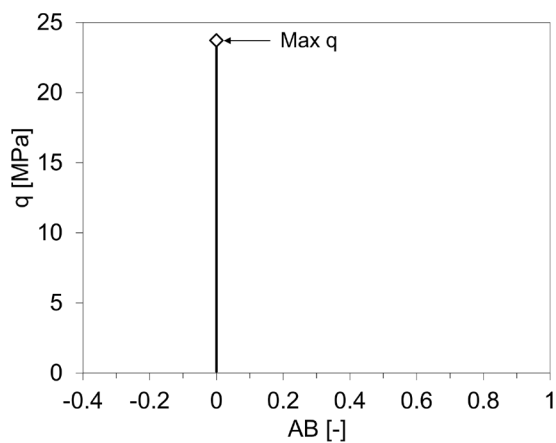
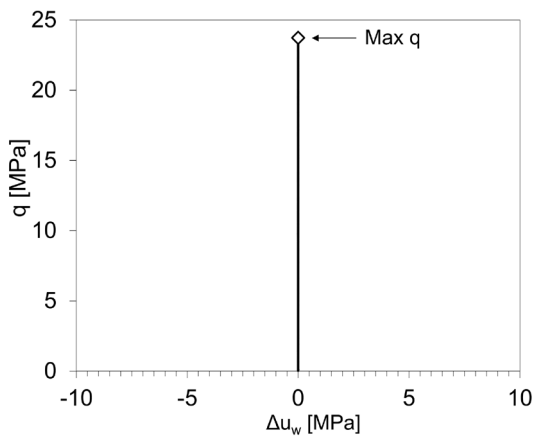
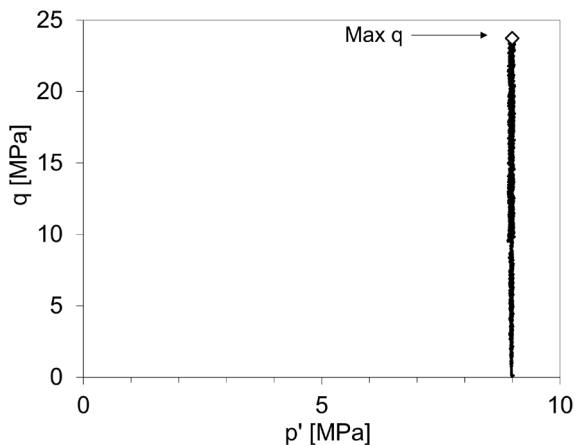
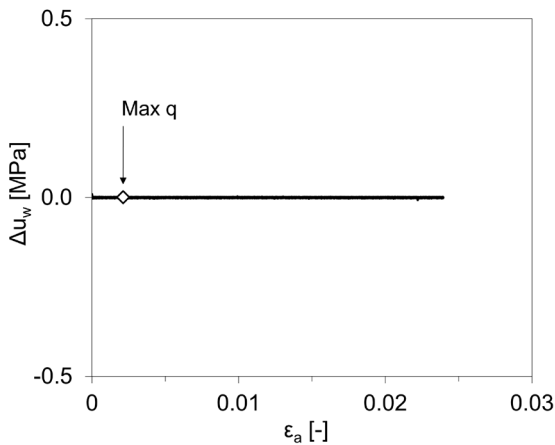
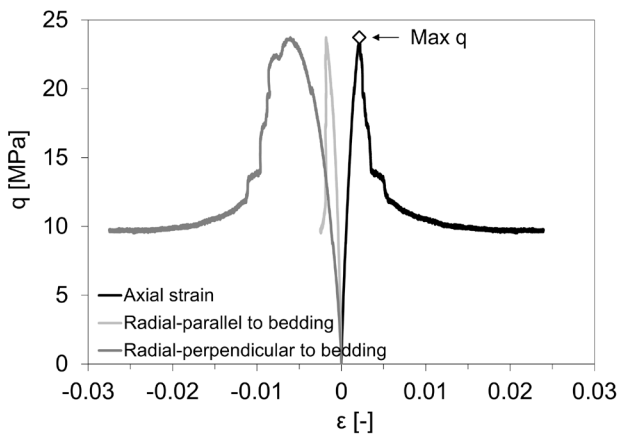
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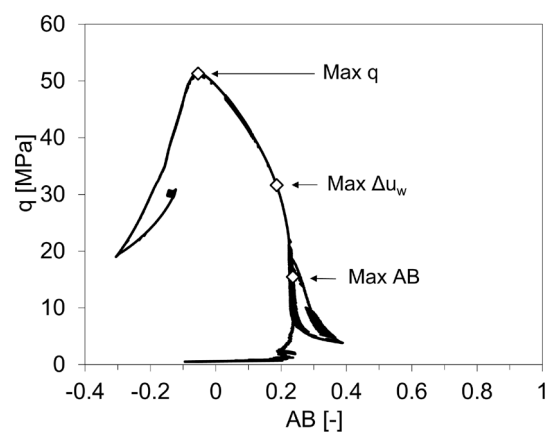
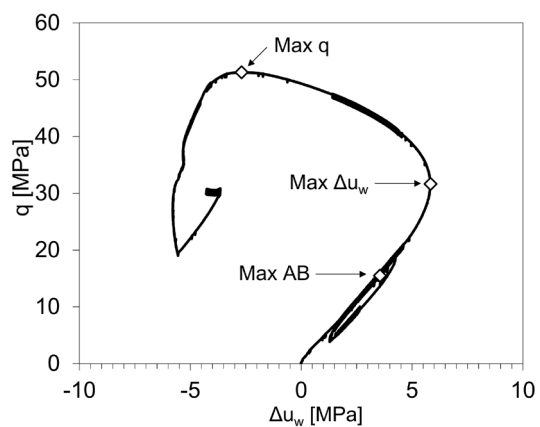
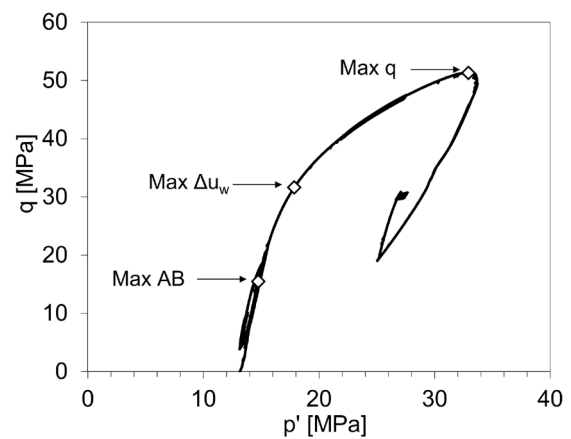
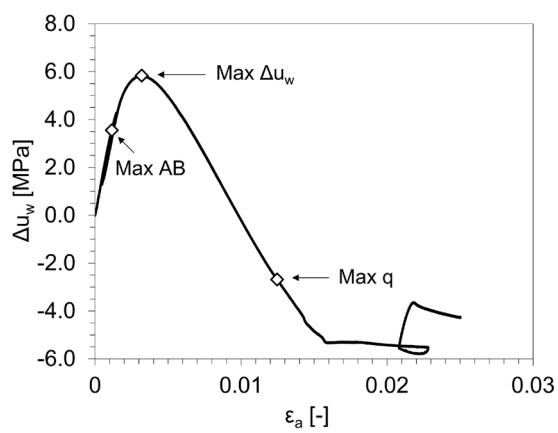
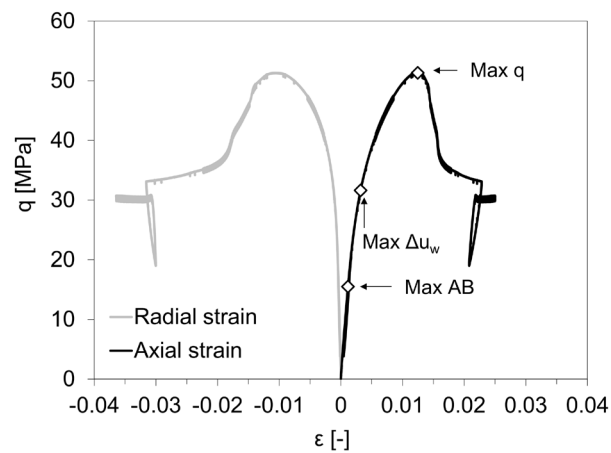
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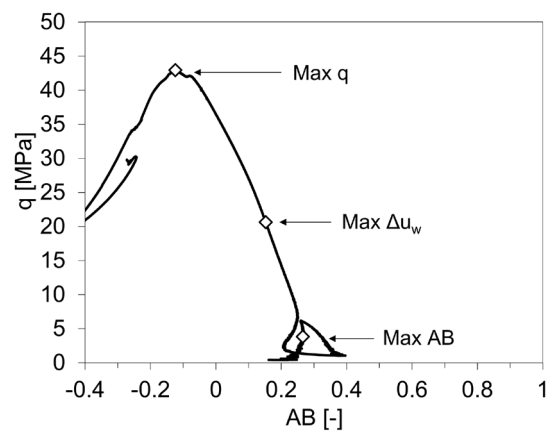
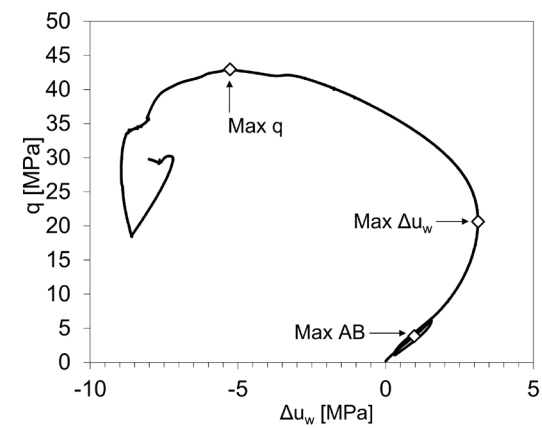
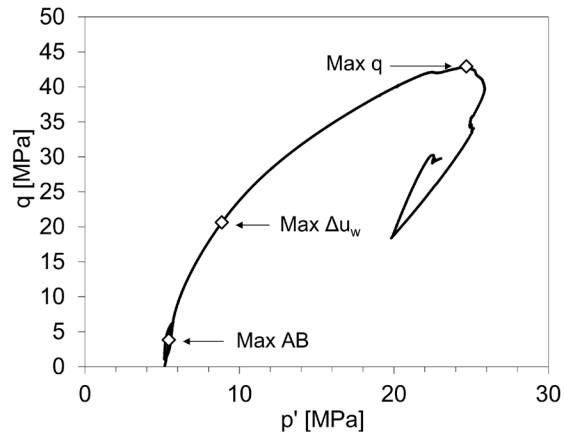
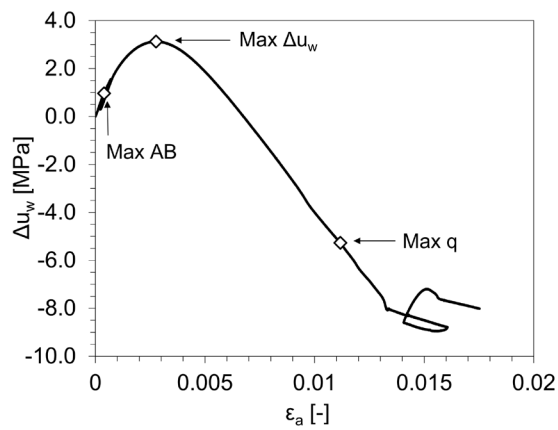
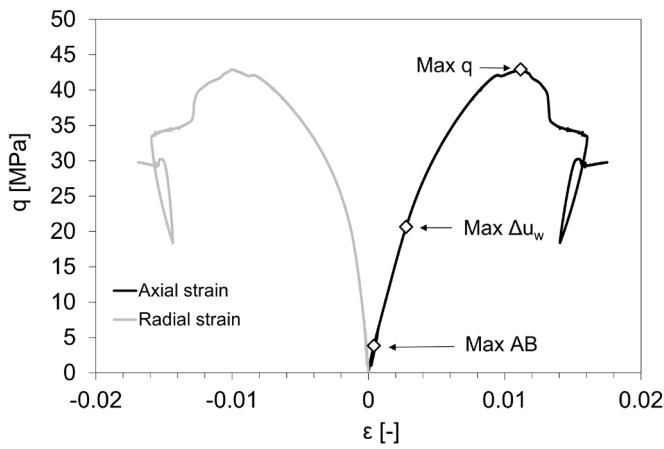
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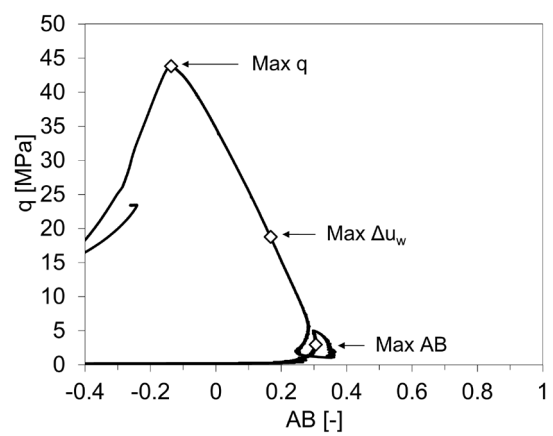
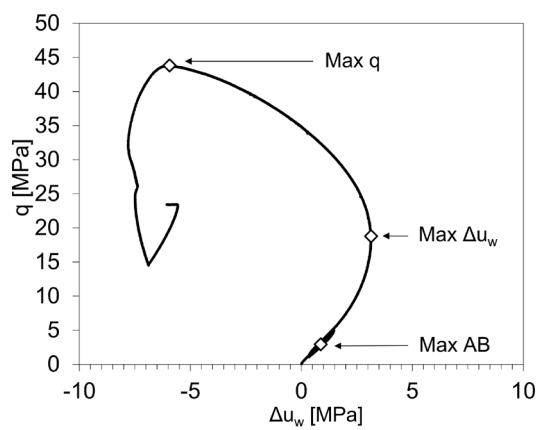
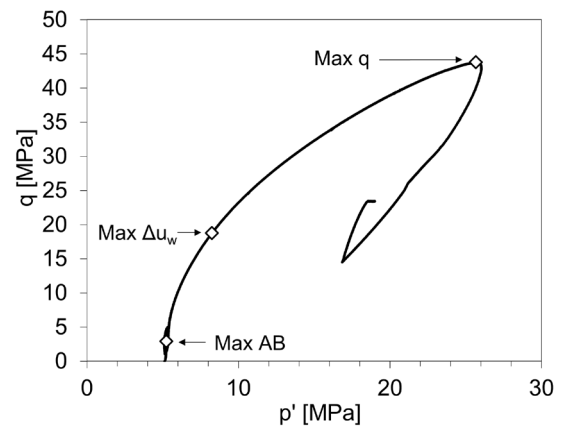
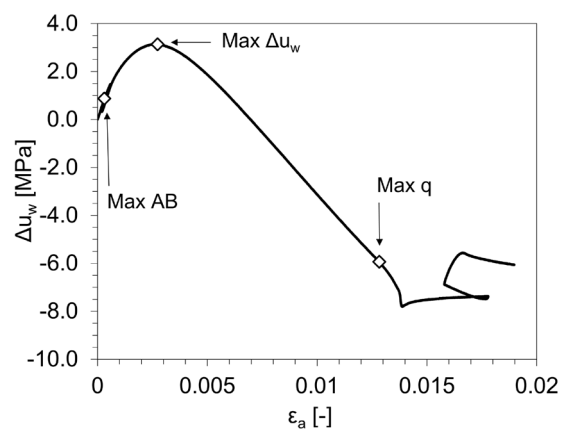
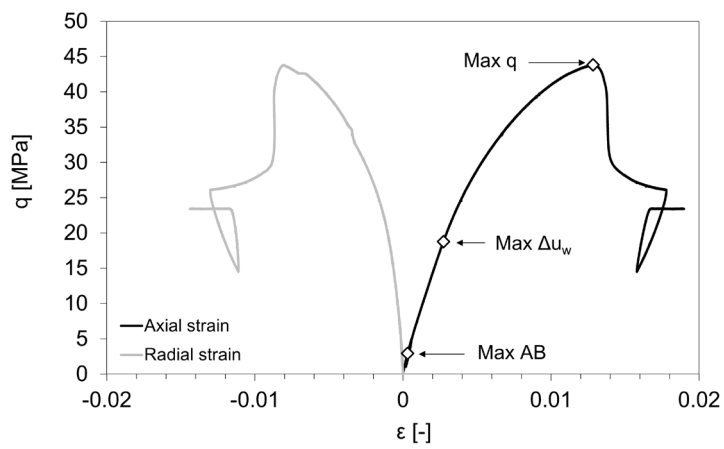


BUL1_1_896_60S13CTCU

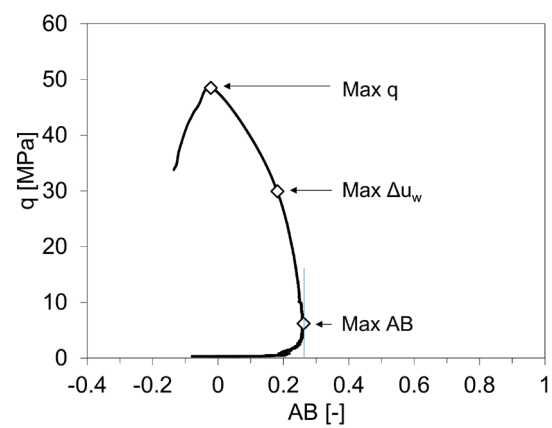
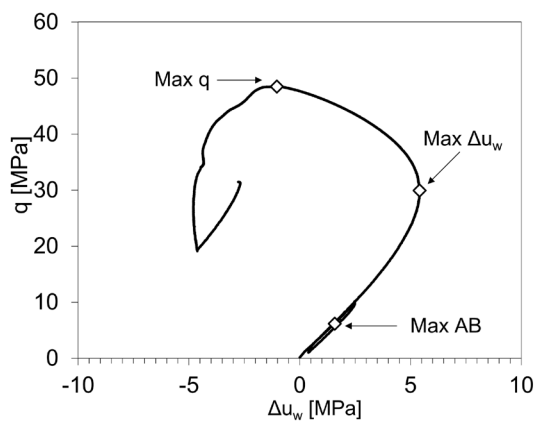
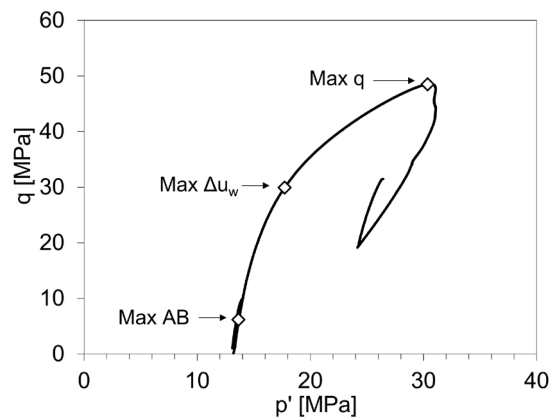
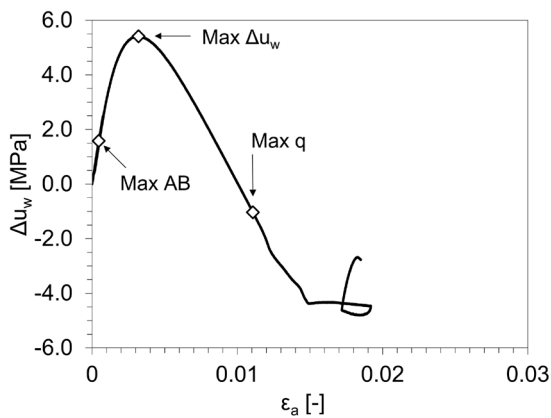
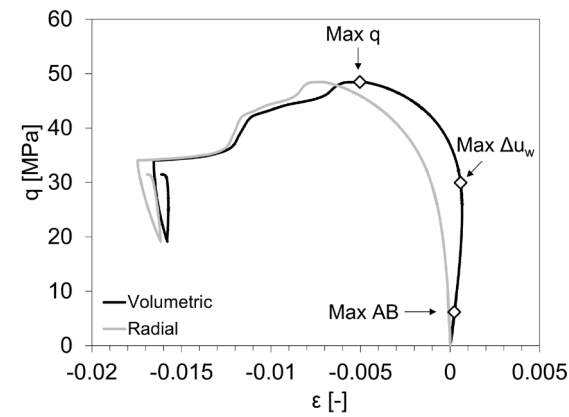


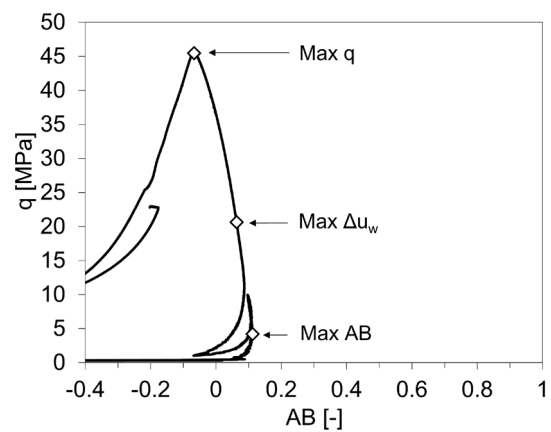
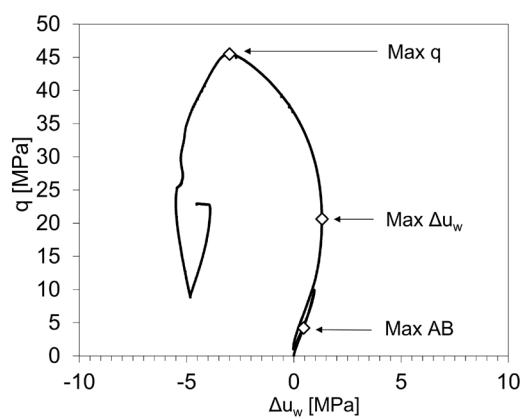
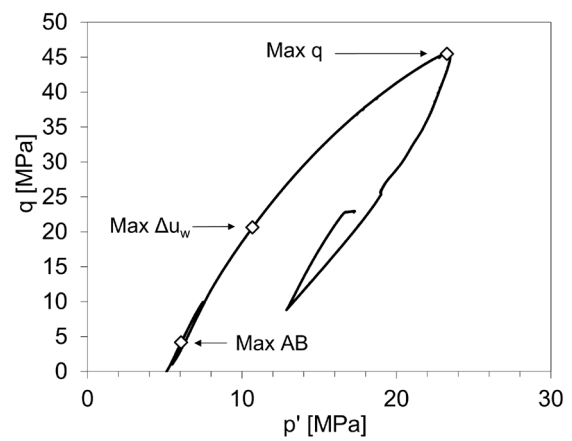
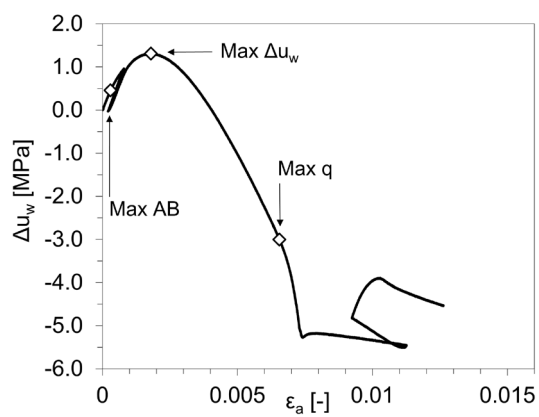
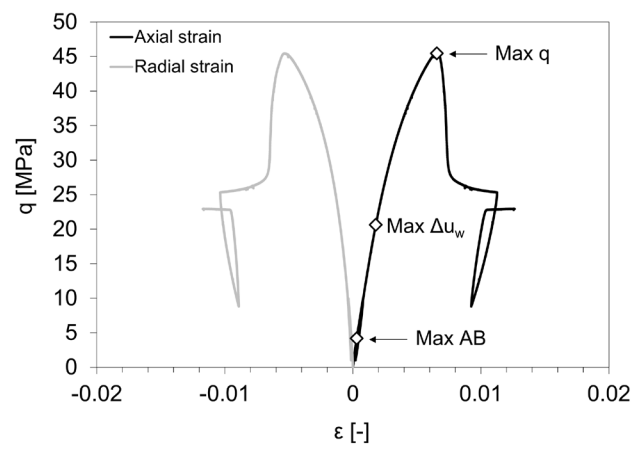
BUL1_1_896_60S5CTCU_B



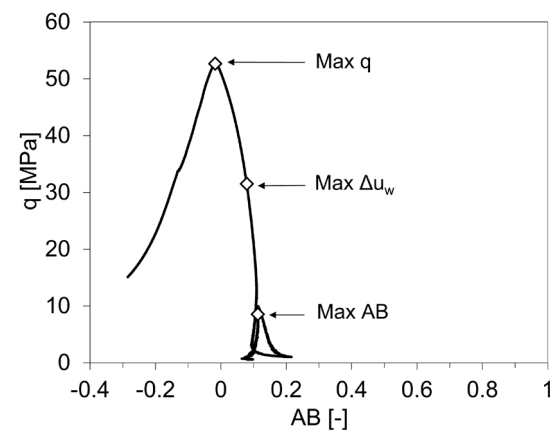
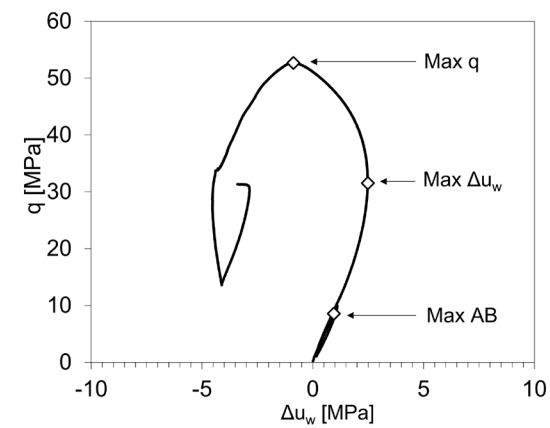
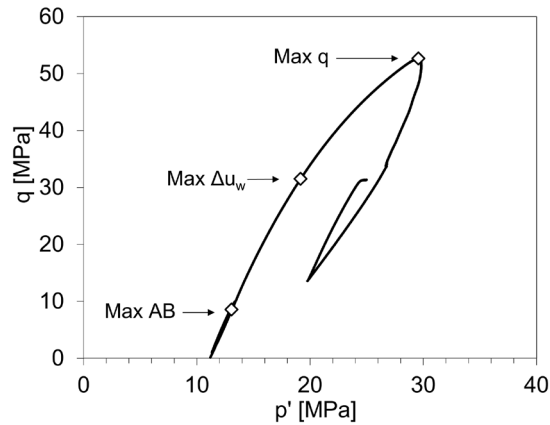
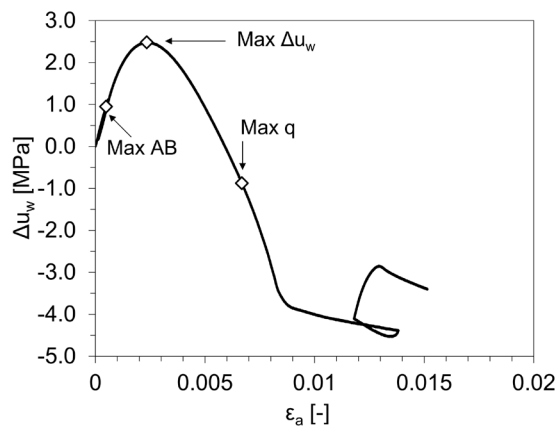
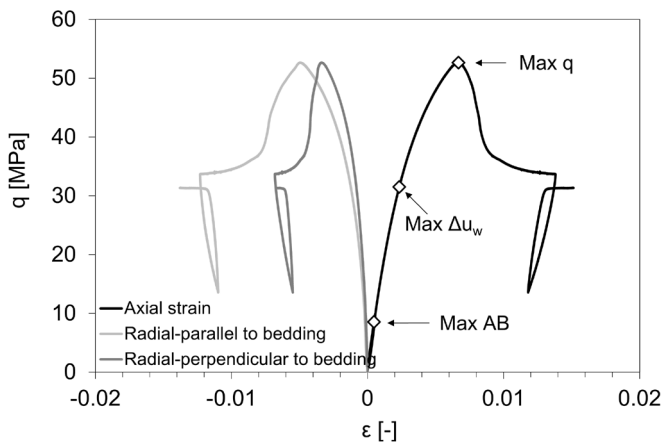
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BUL1_1_896_60S13CTCUT_B

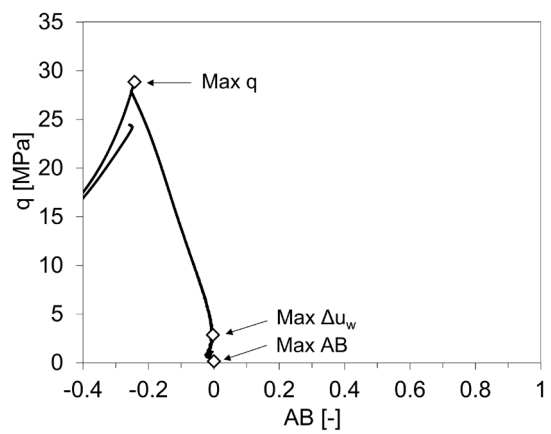
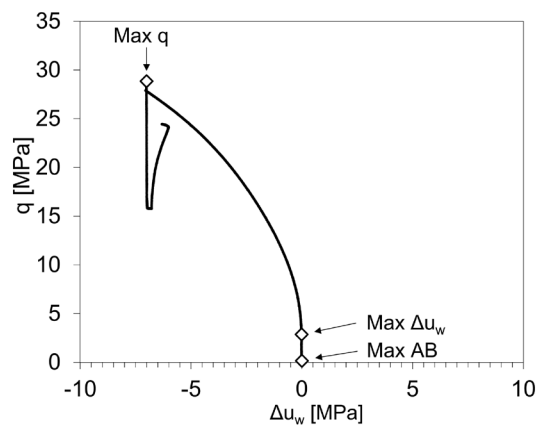
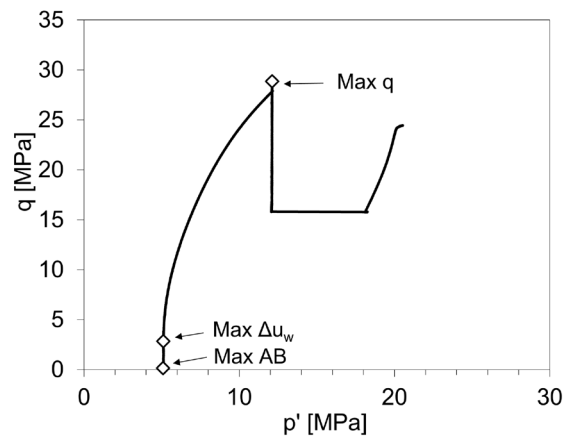
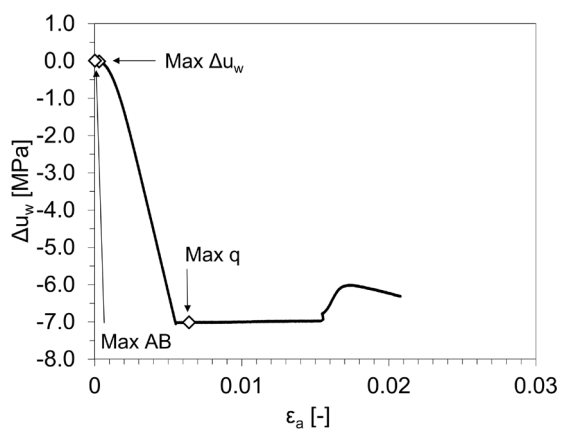
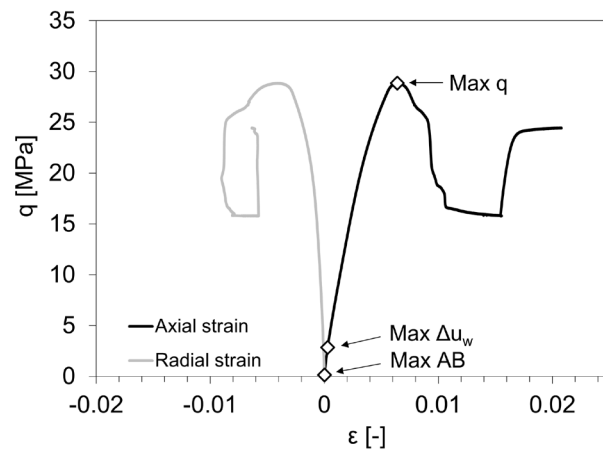


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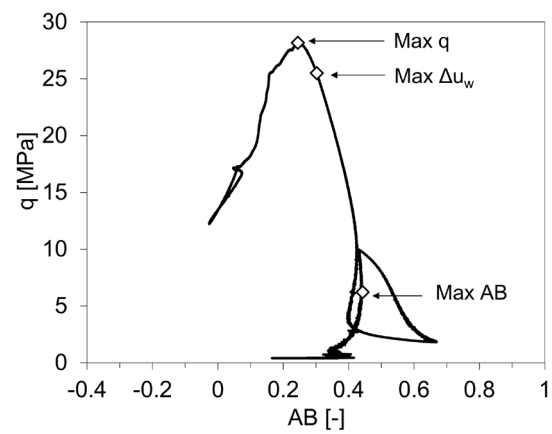
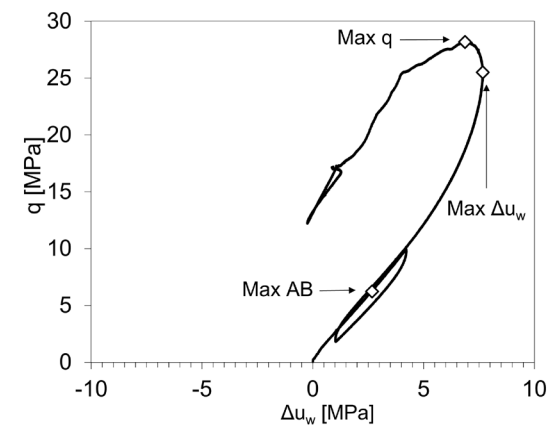
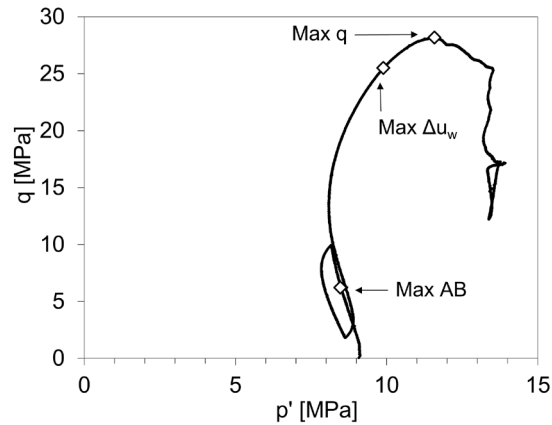
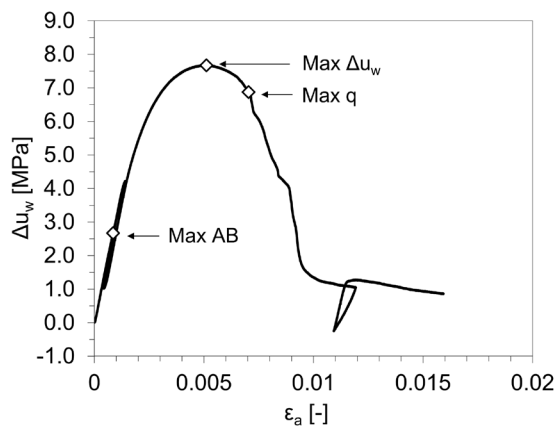
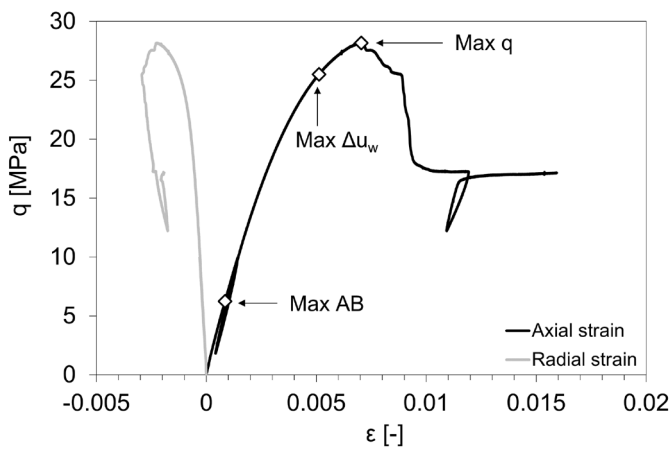
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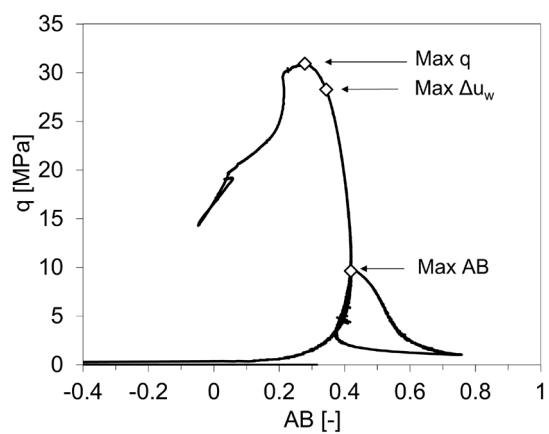
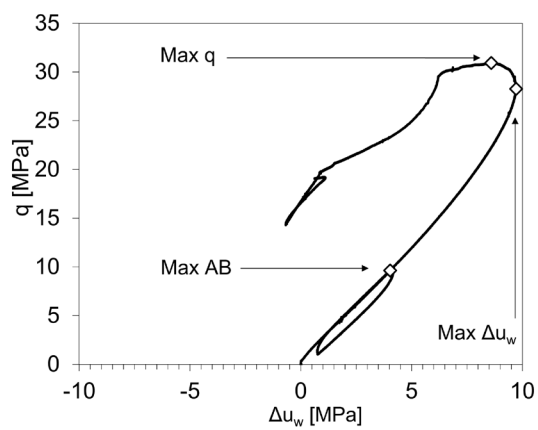
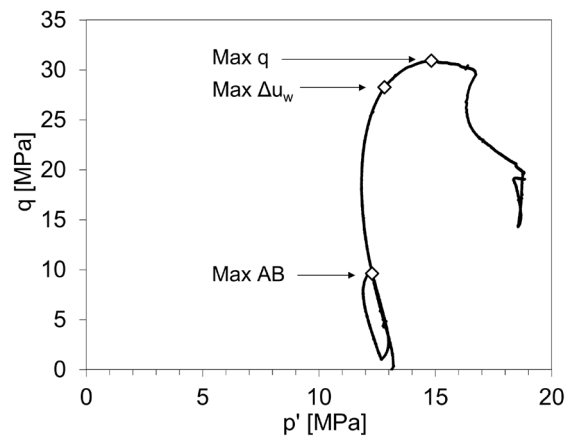
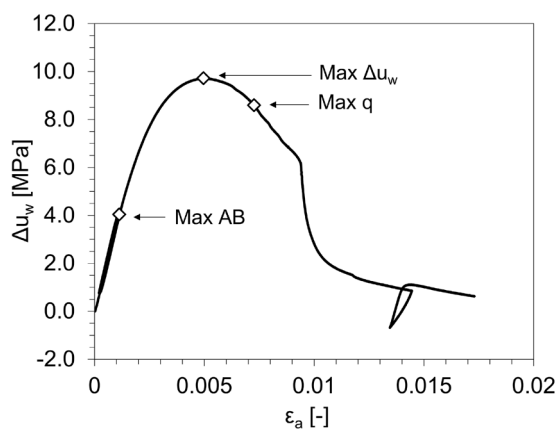
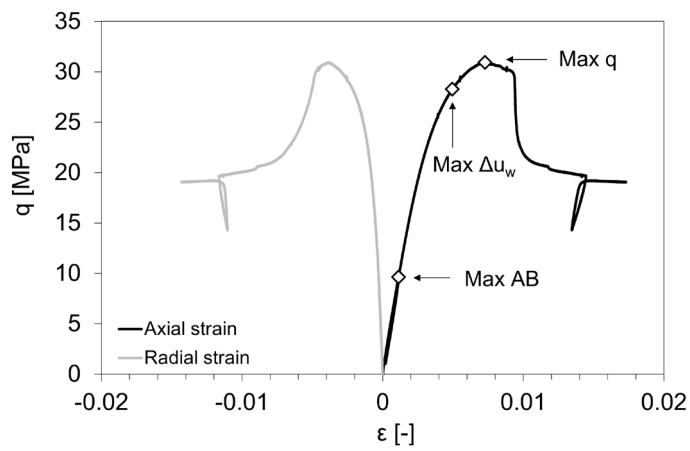


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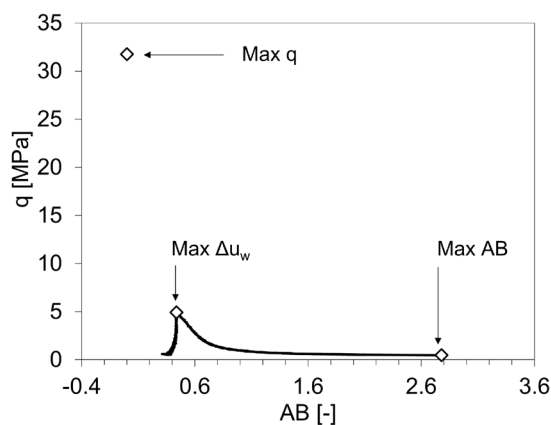
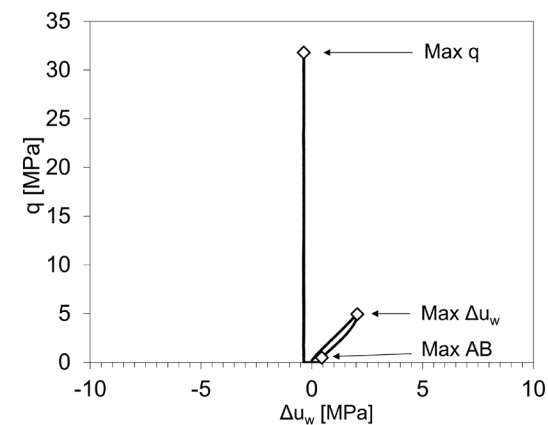
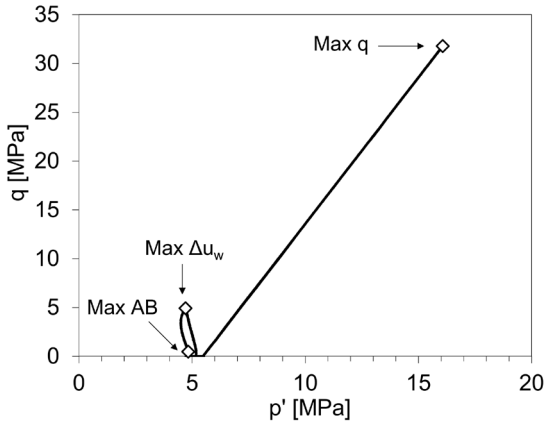
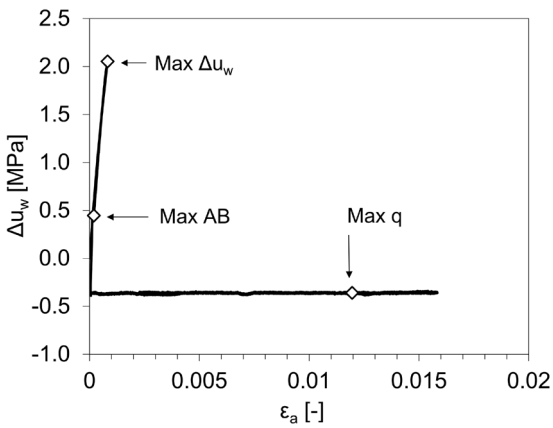
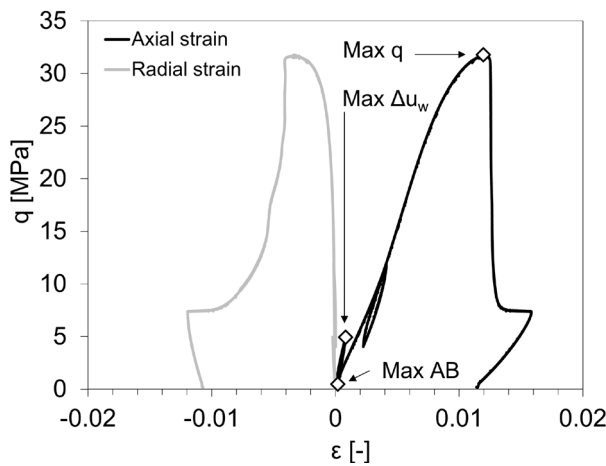


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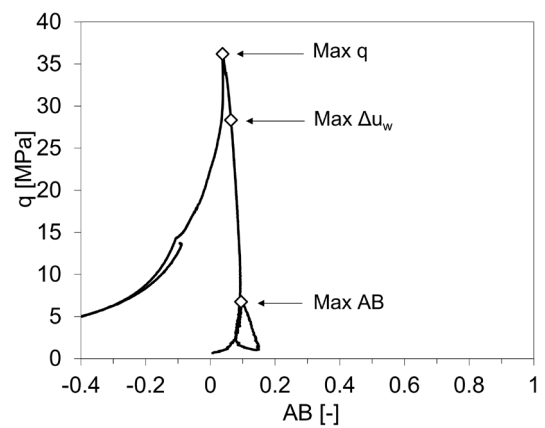
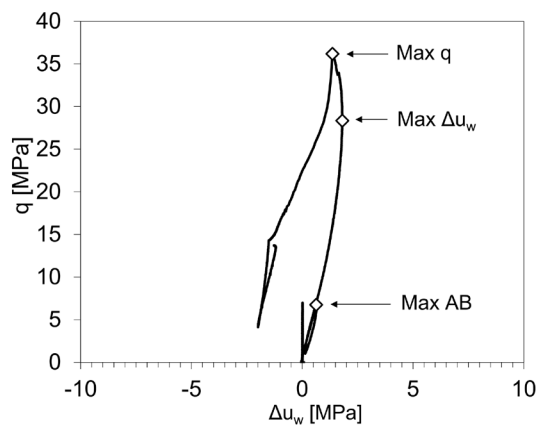
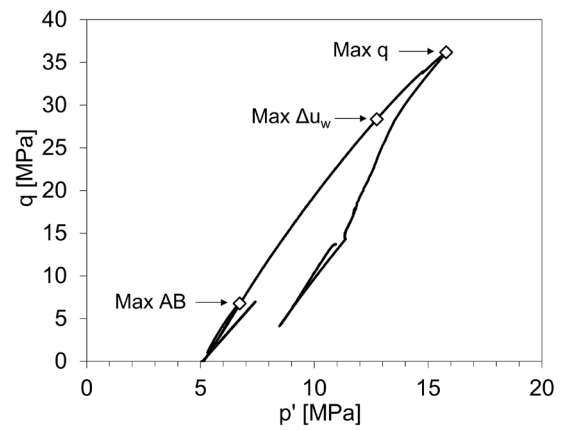
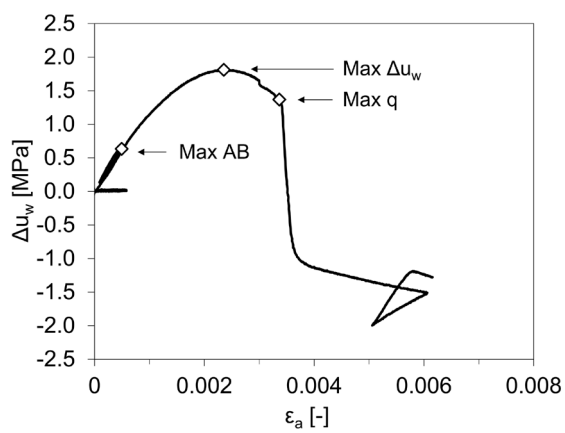
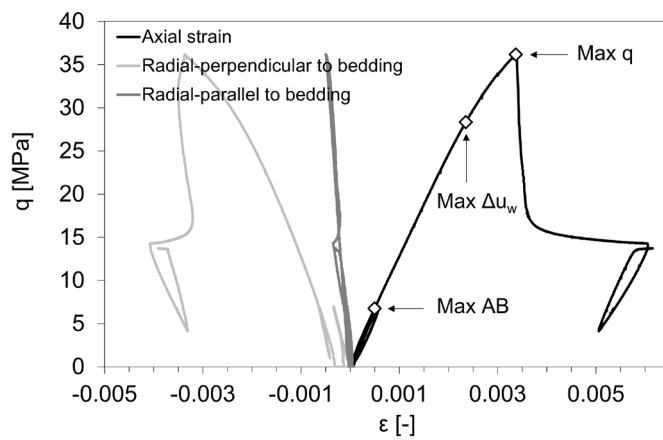


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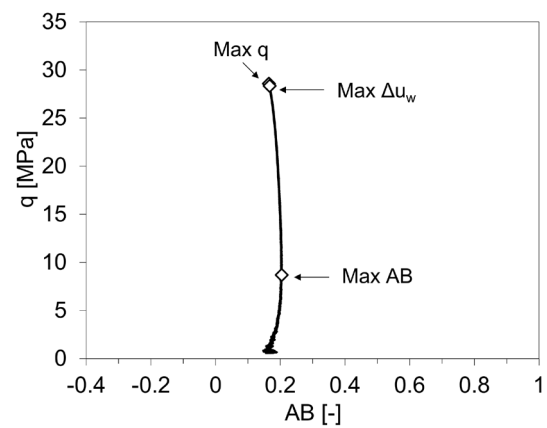
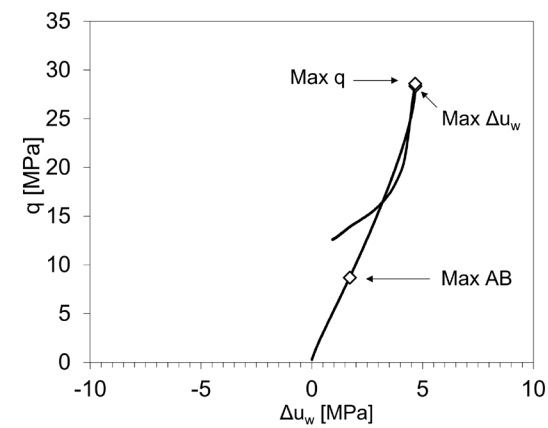
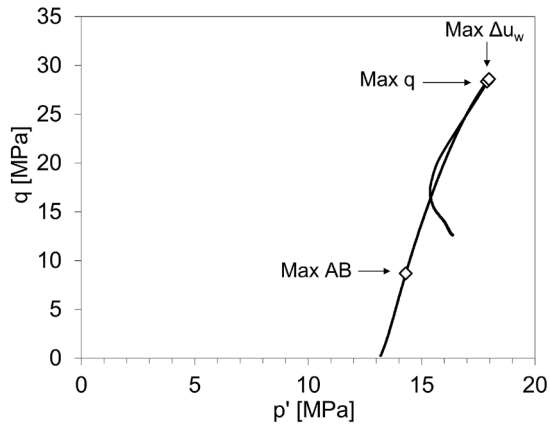
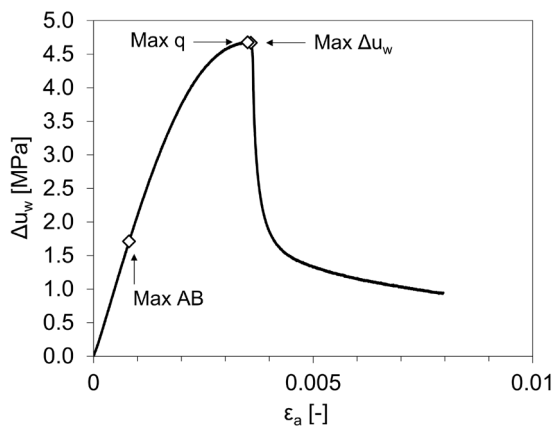
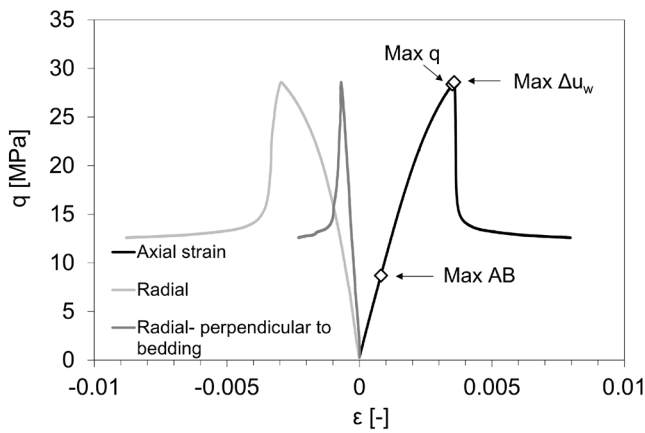
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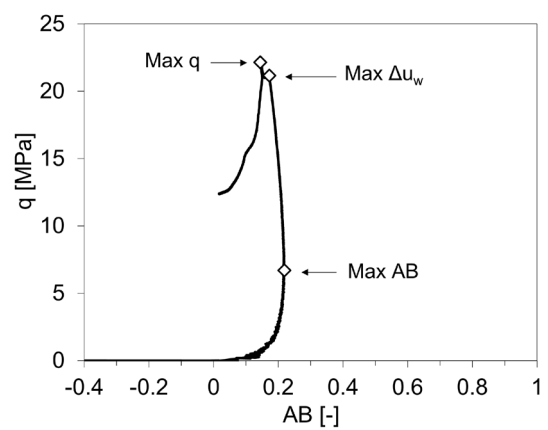
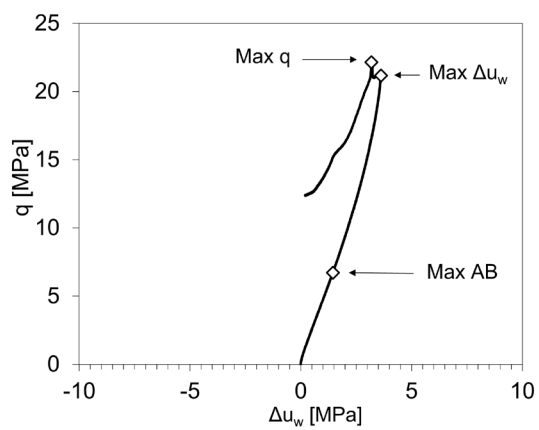
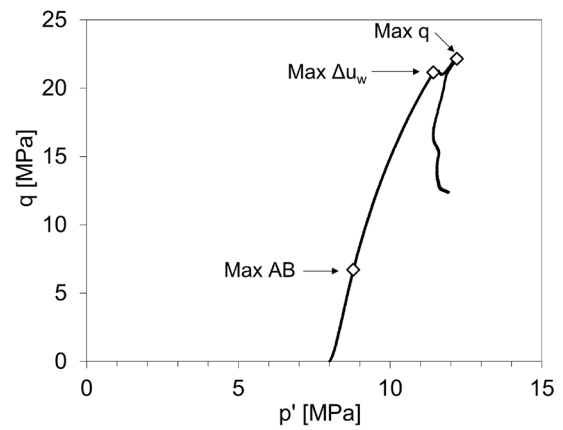
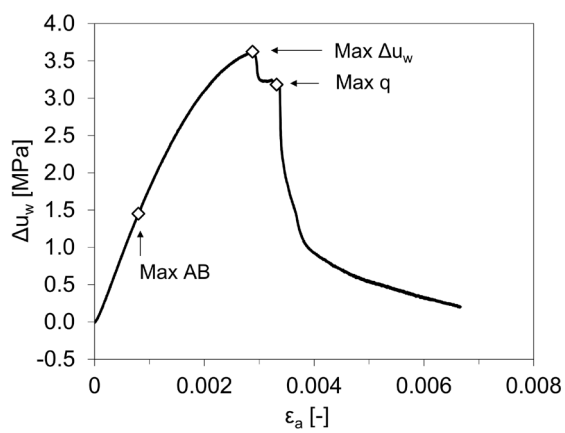
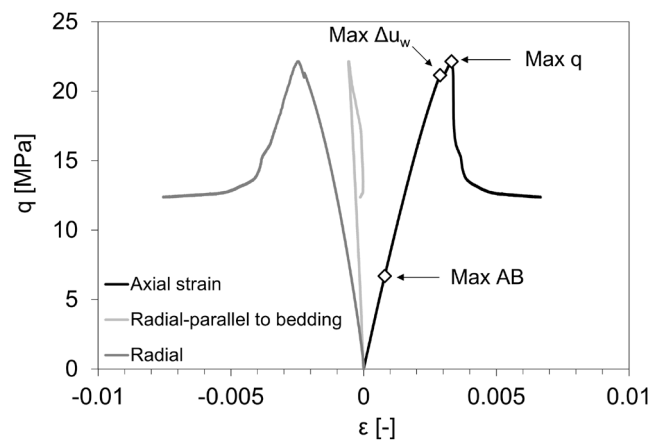


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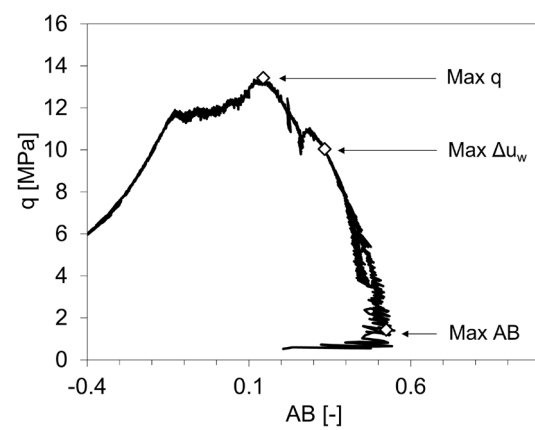
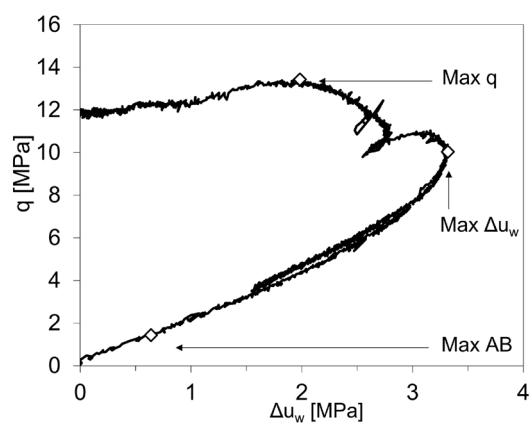
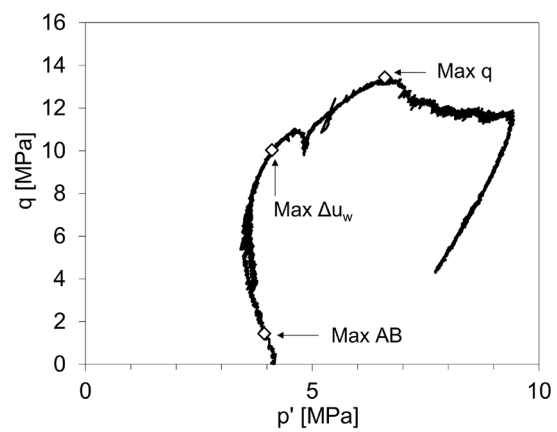
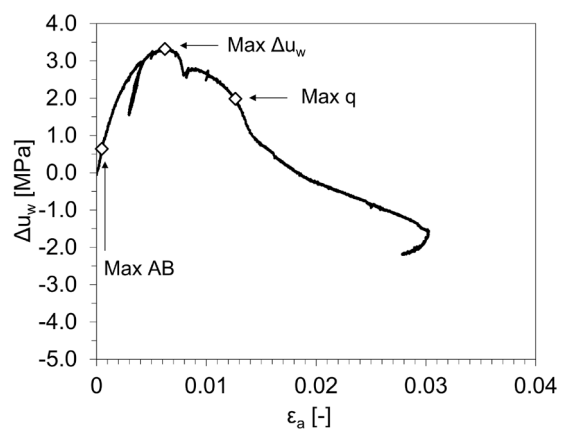
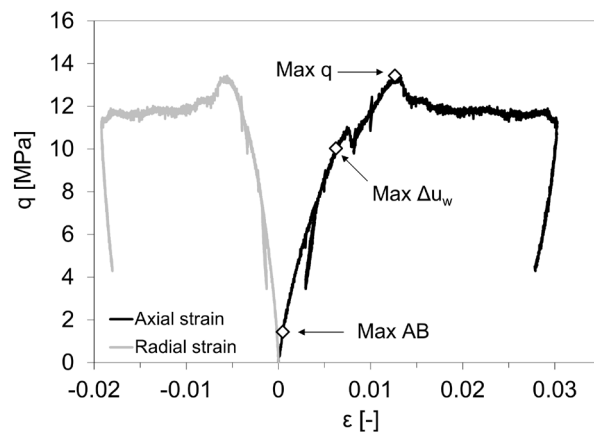


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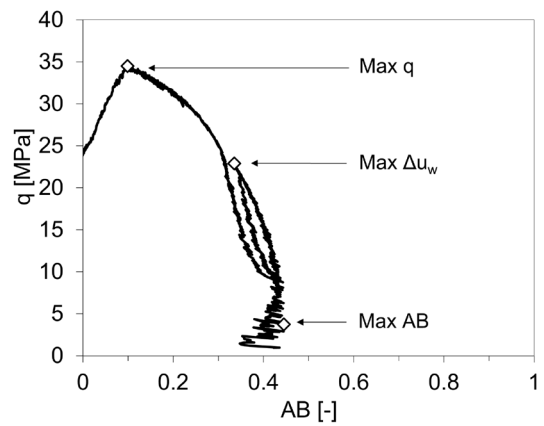
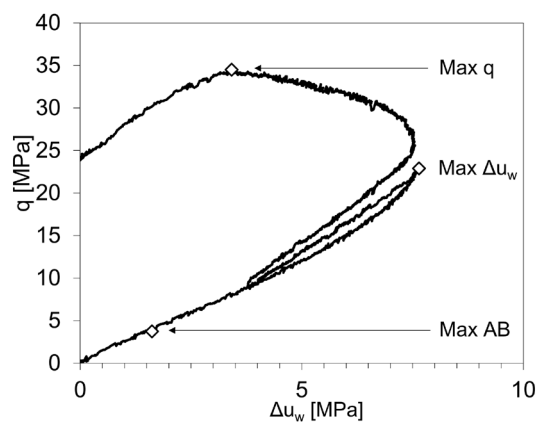
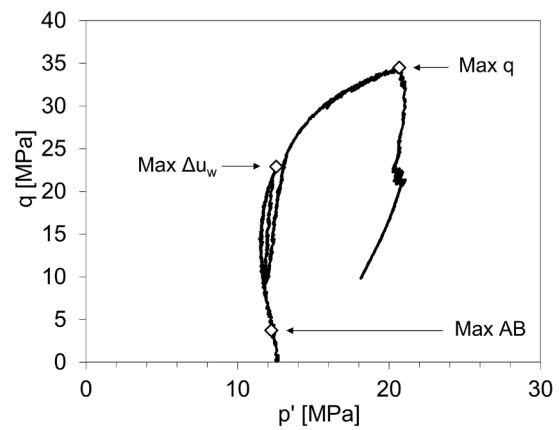
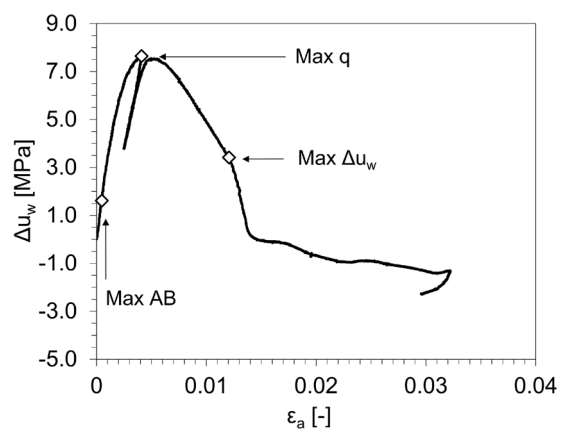
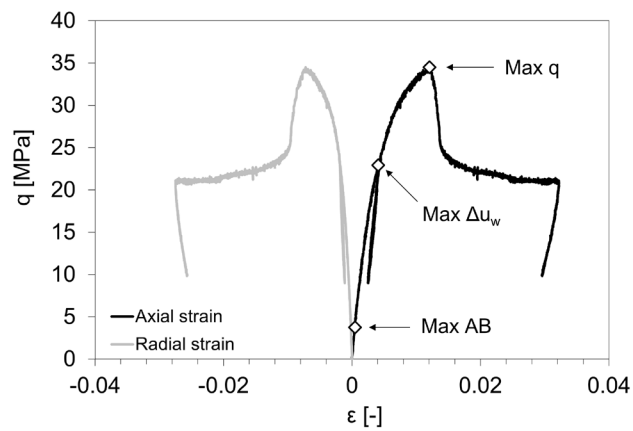


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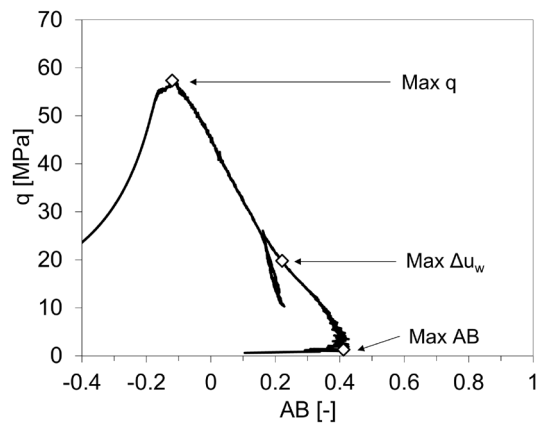
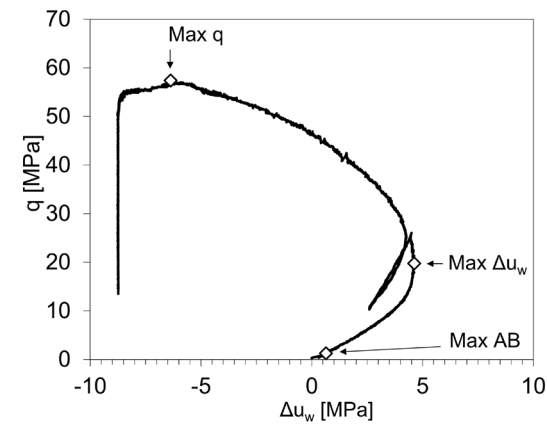
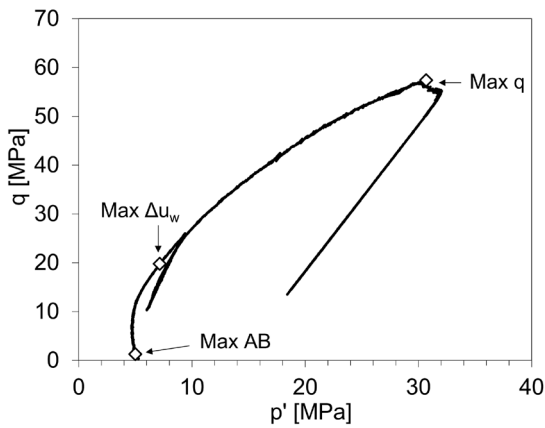
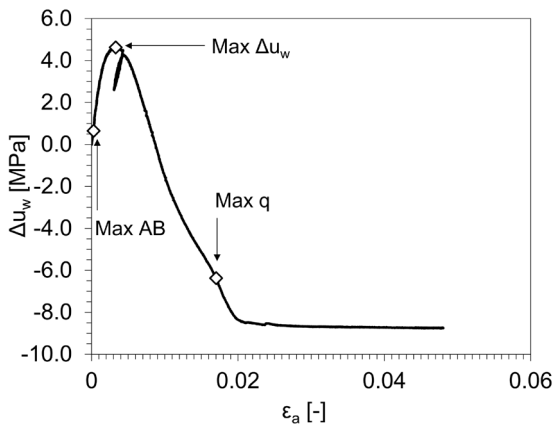
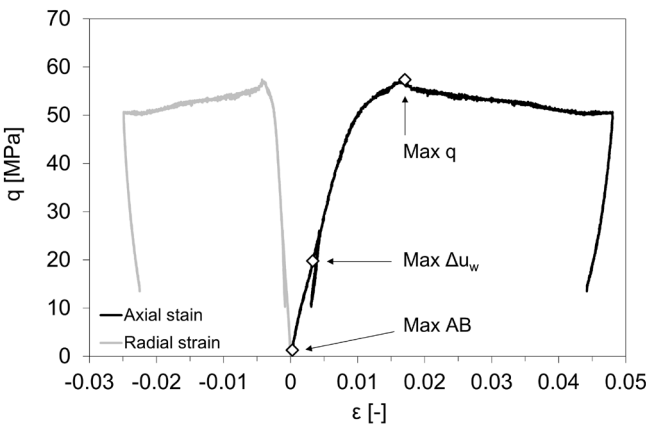
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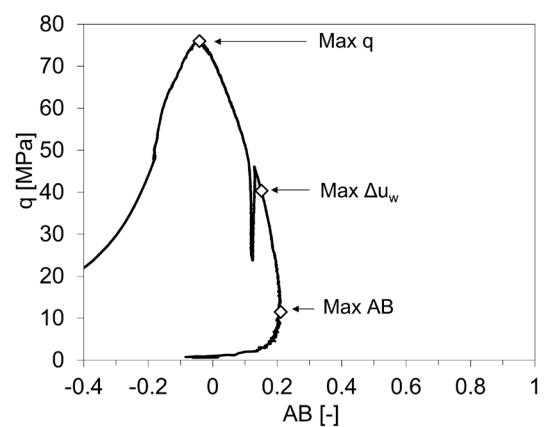
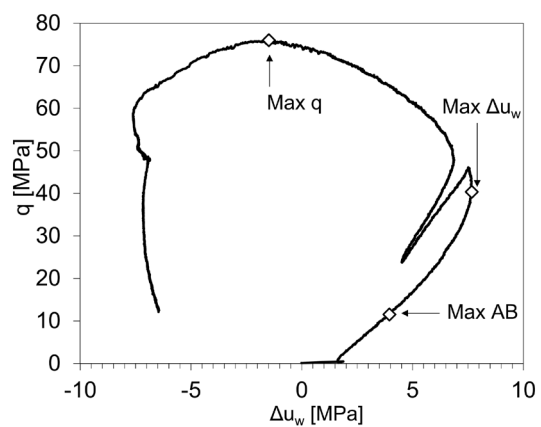
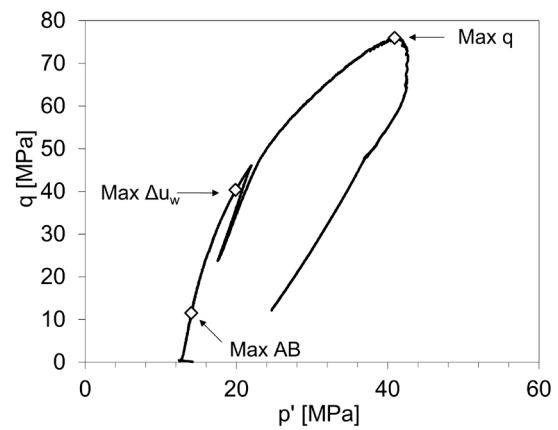
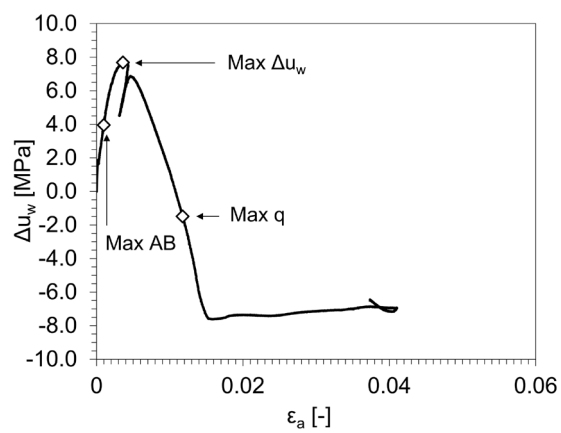
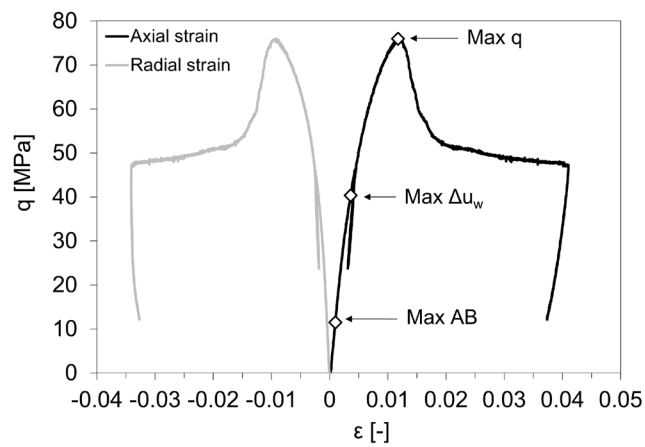
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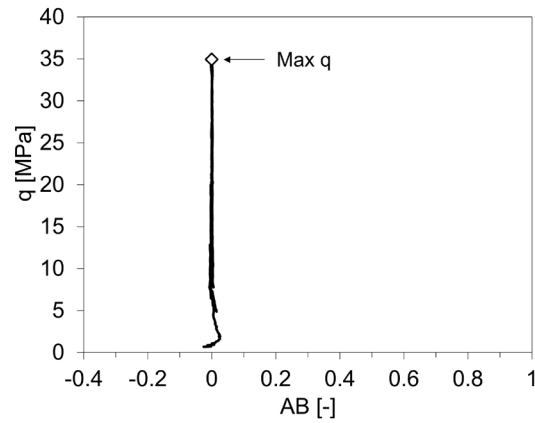
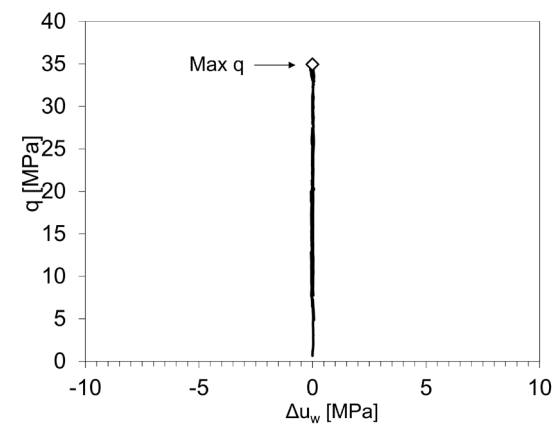
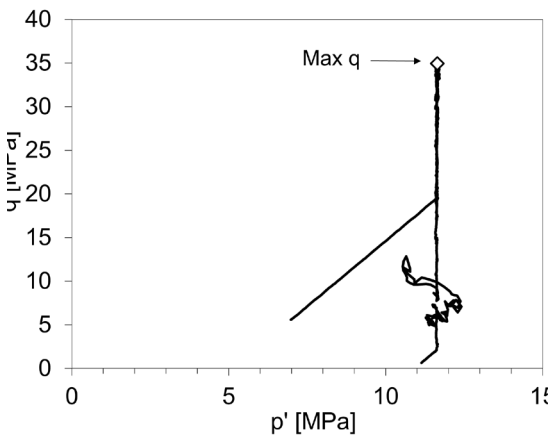
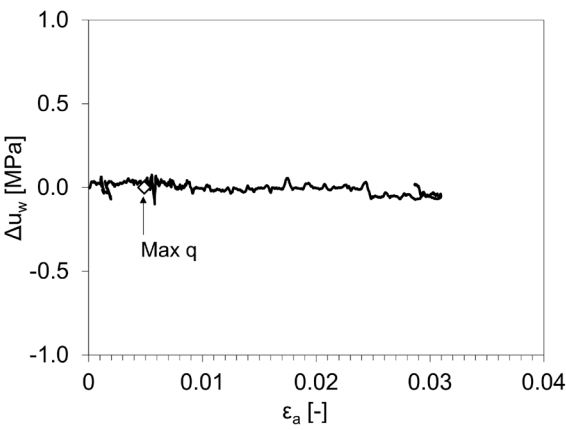
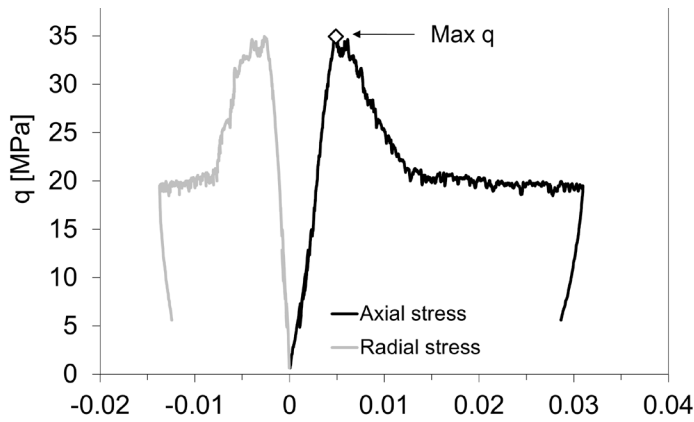
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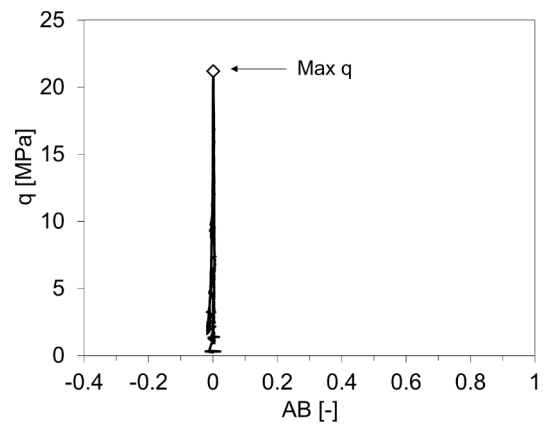
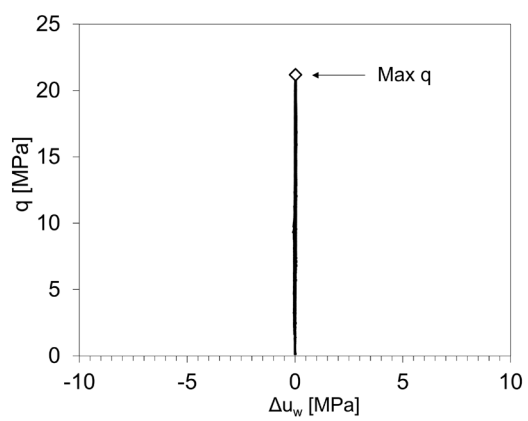
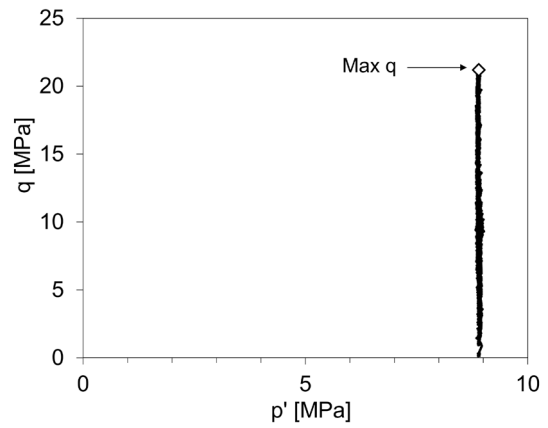
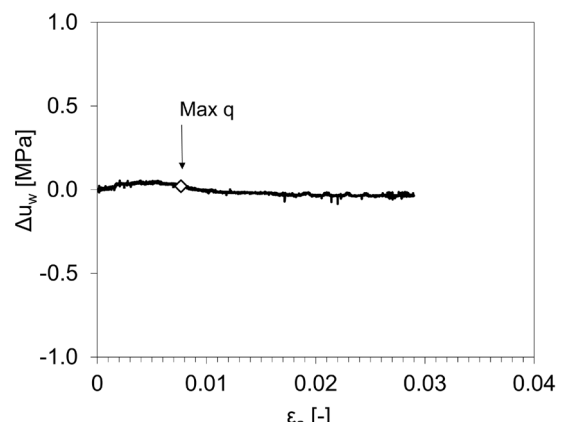
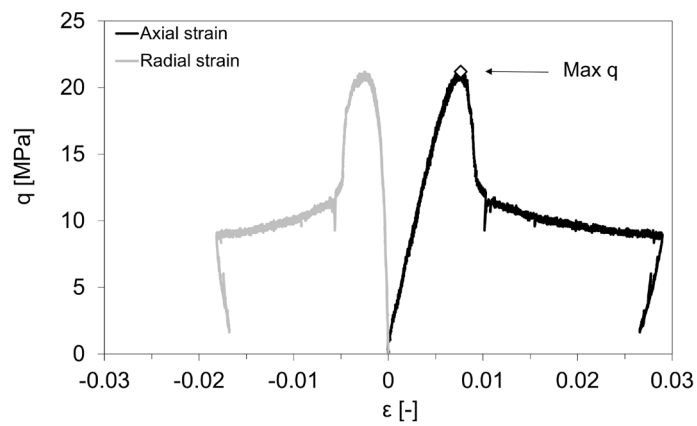
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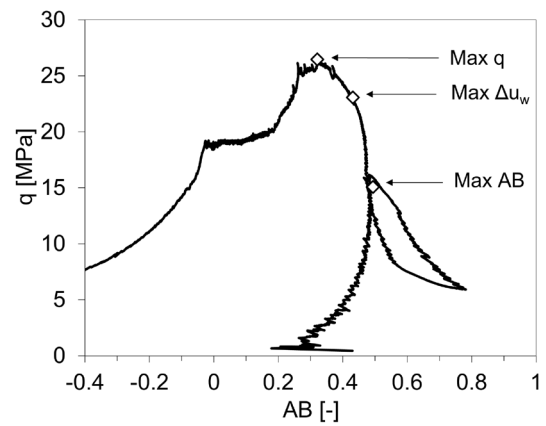
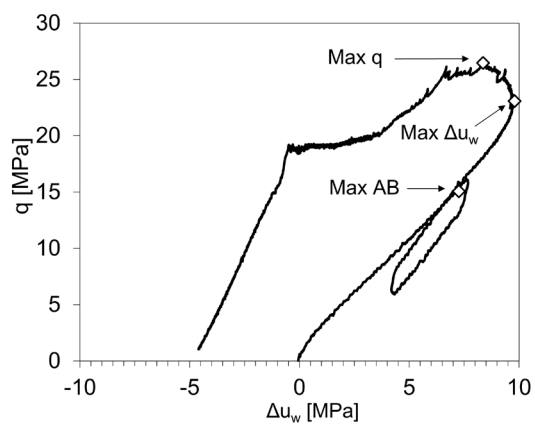
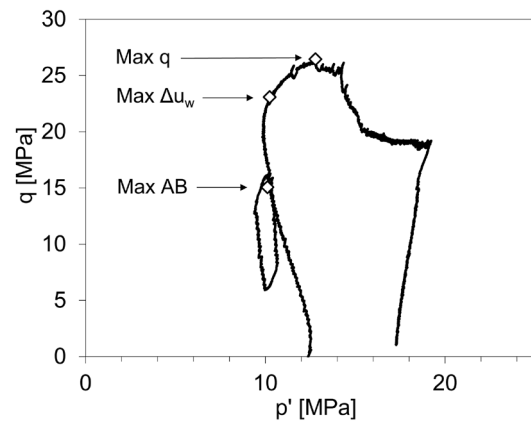
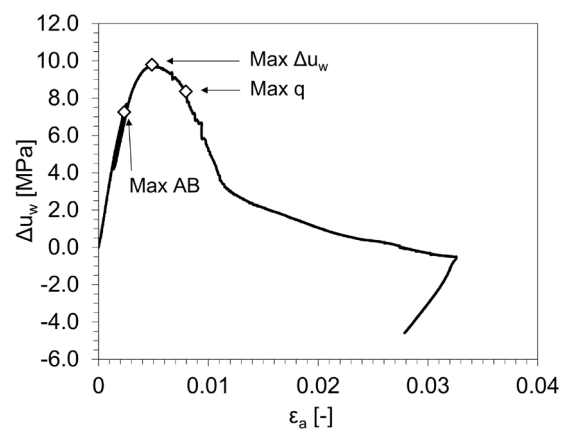
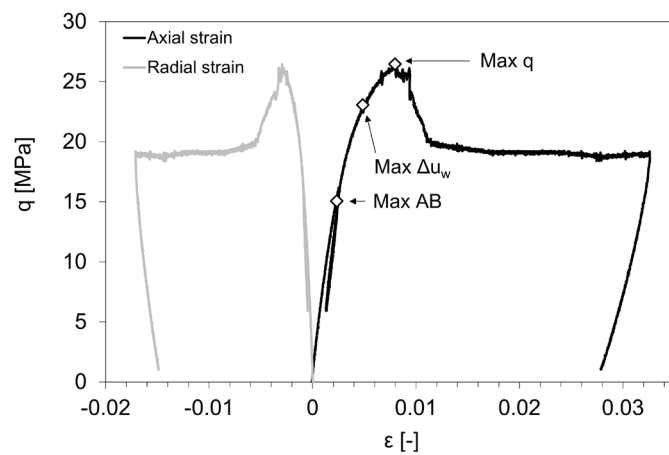
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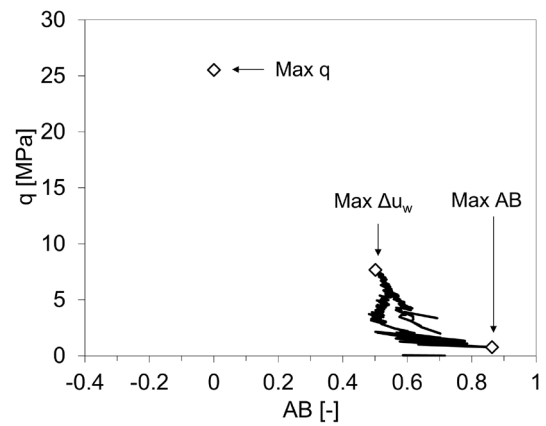
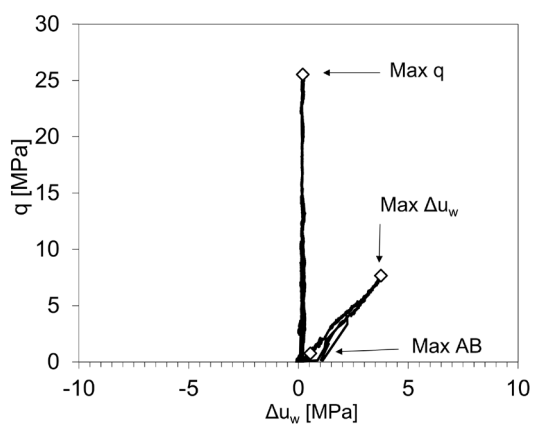
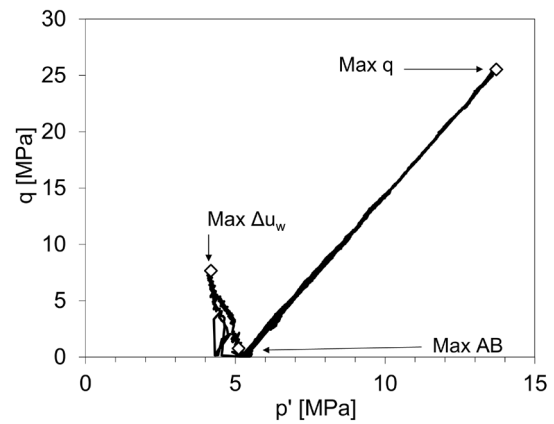
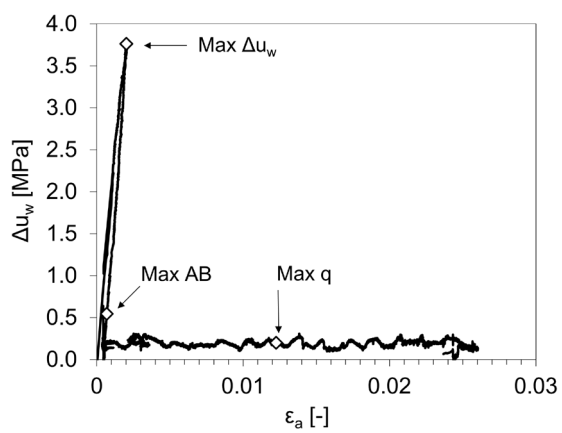
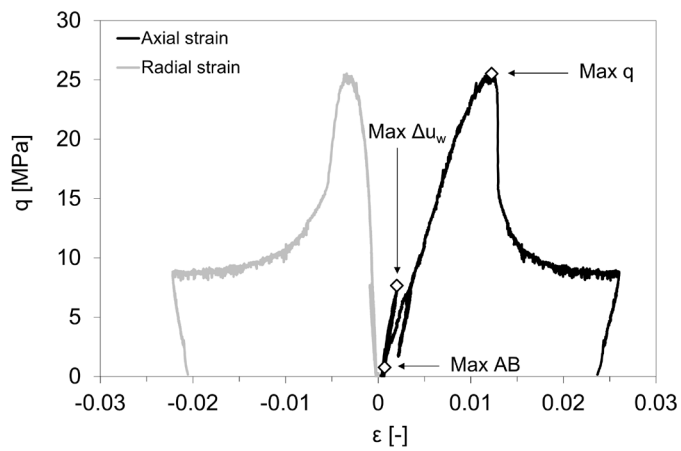
BUL1_1_915_33S9TC_C



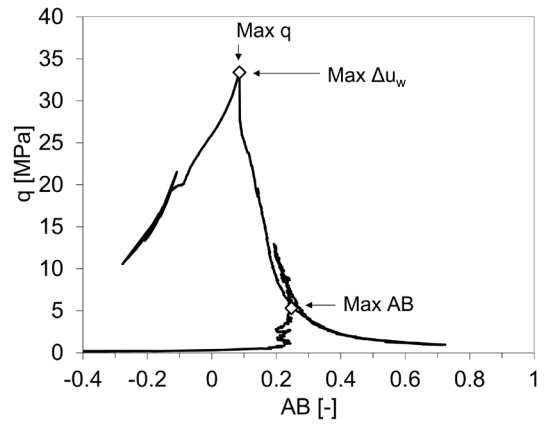
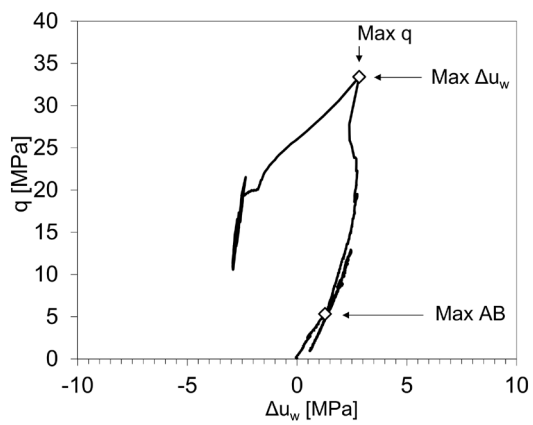
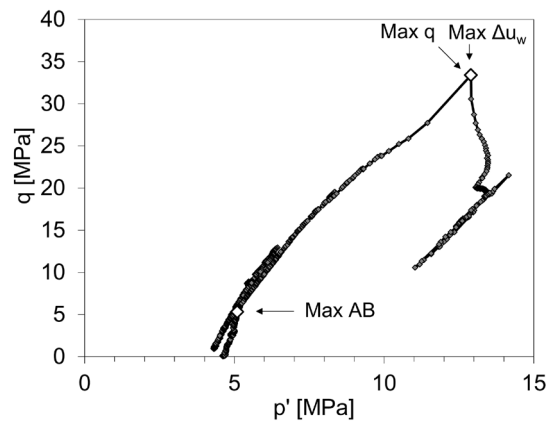
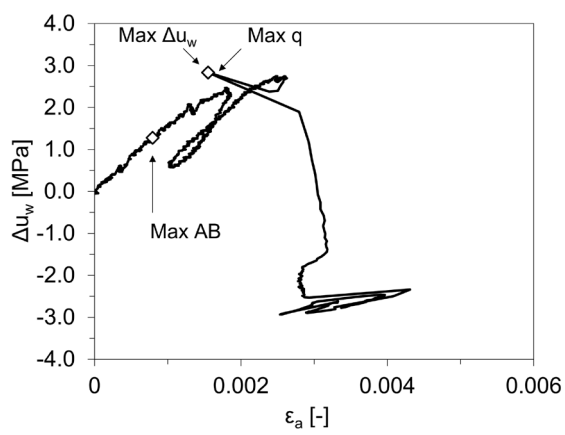
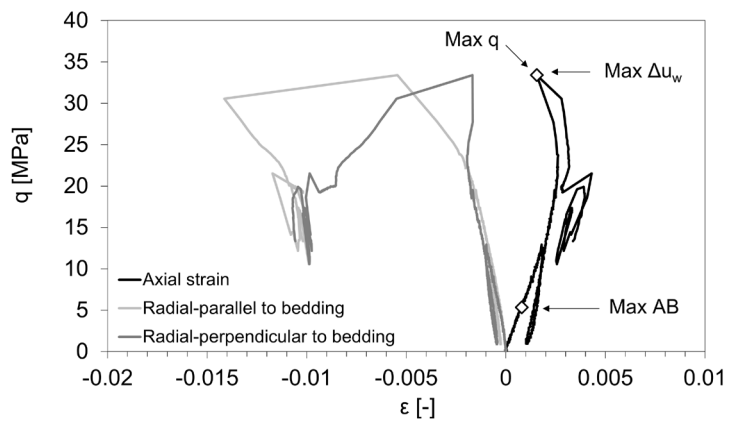
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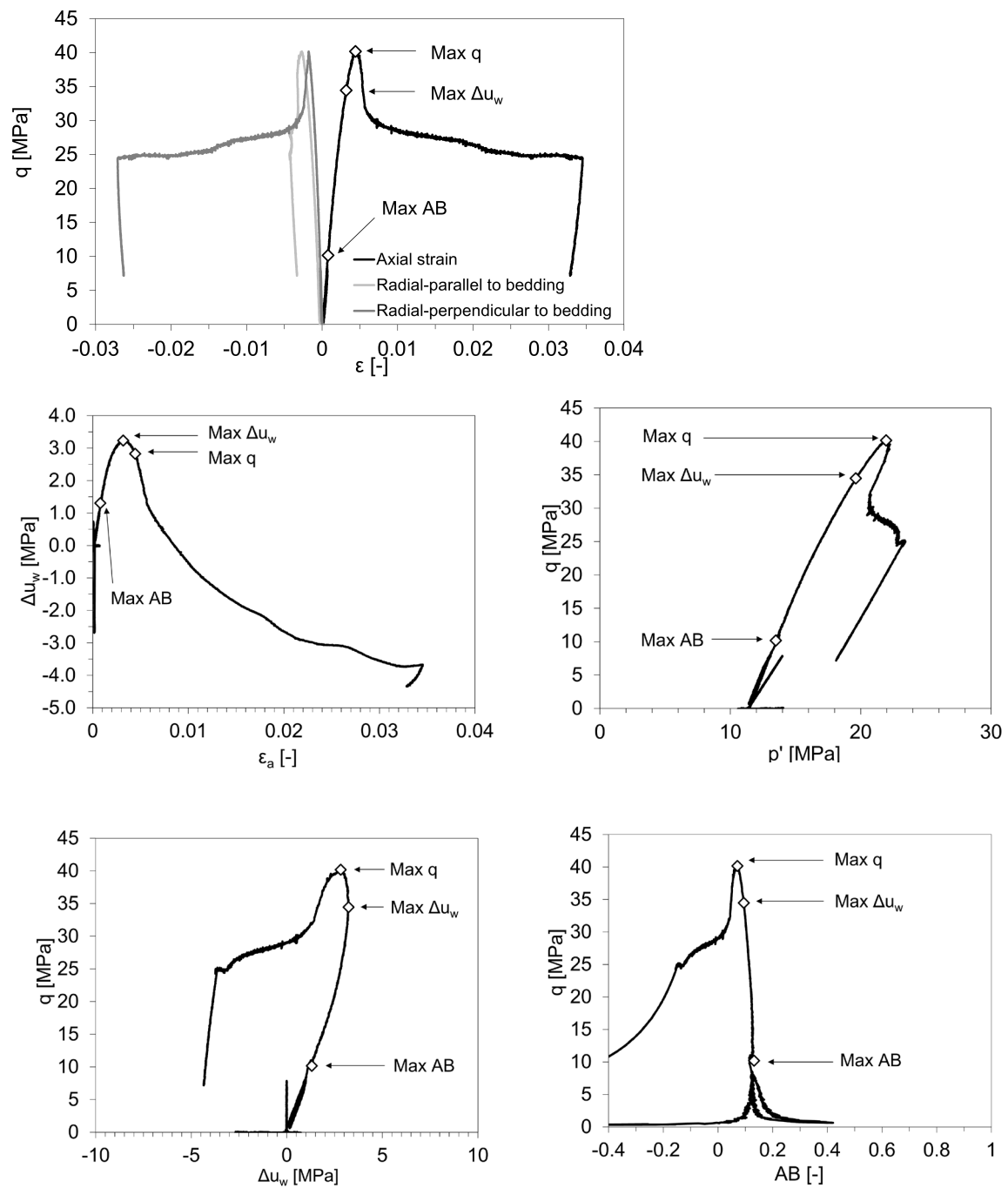
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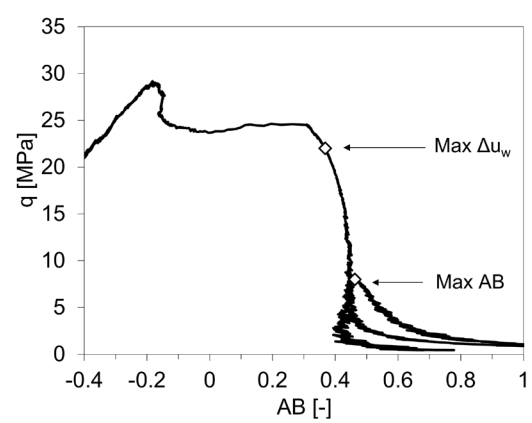
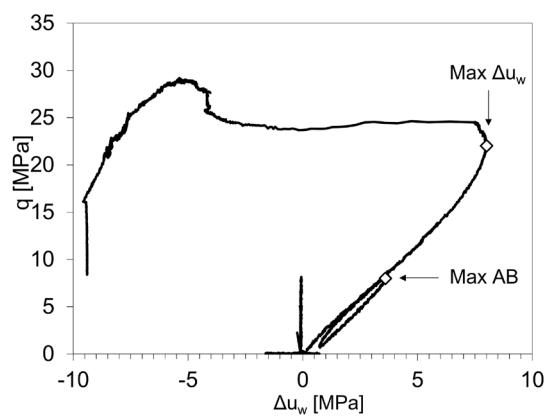
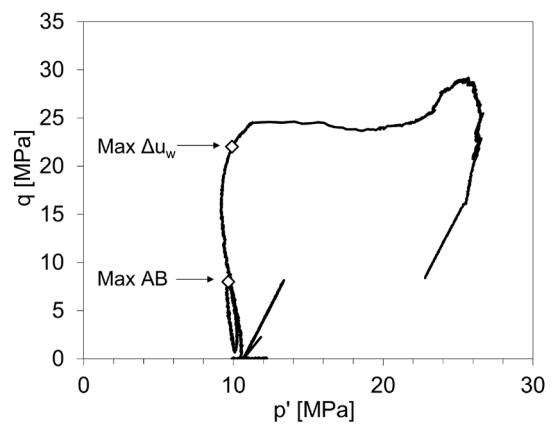
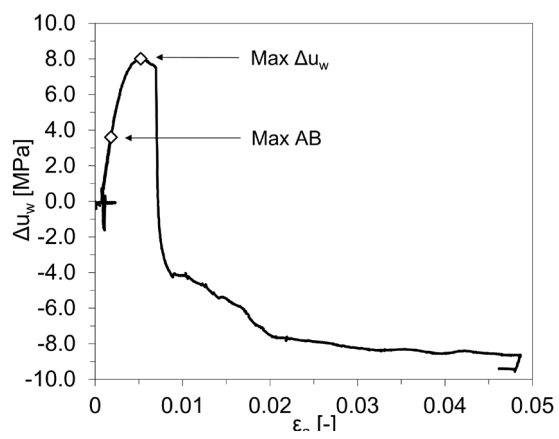
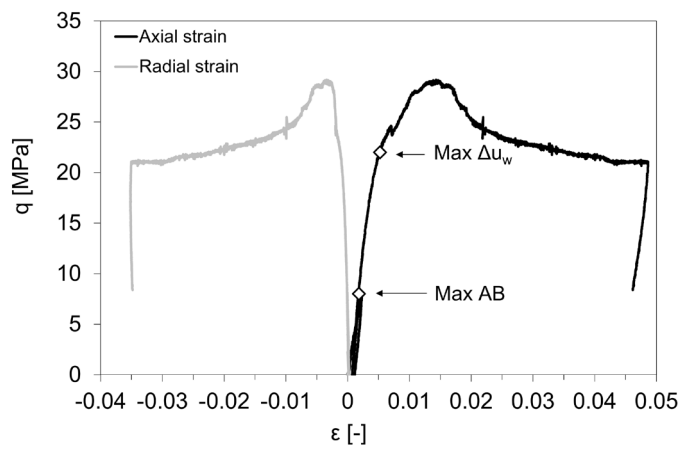
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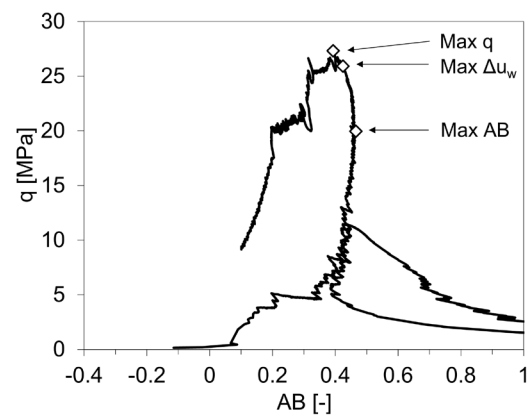
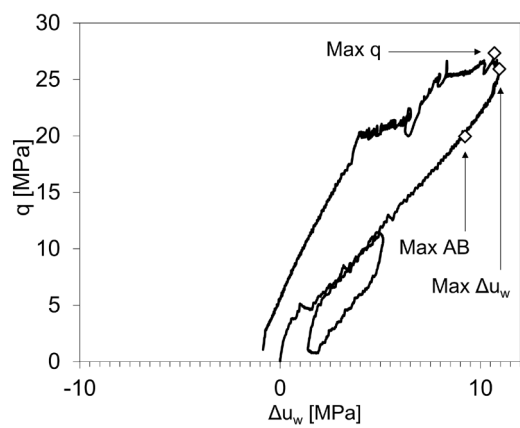
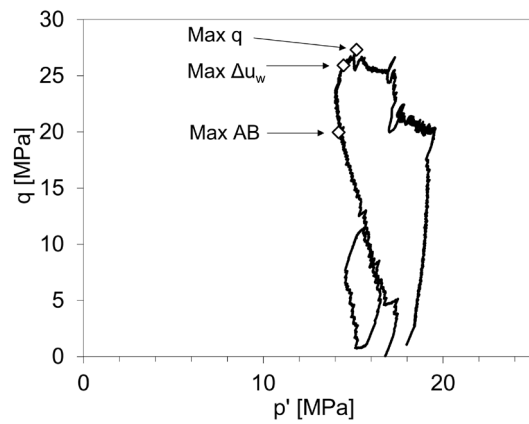
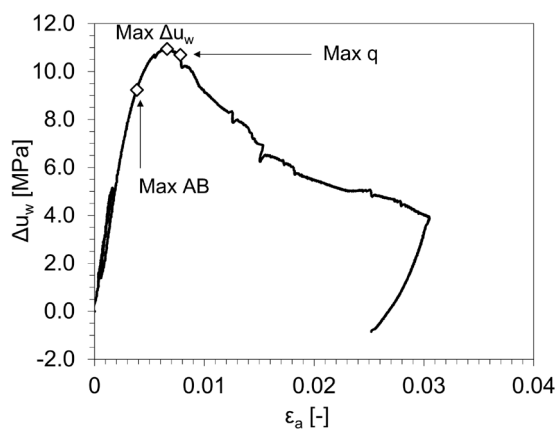
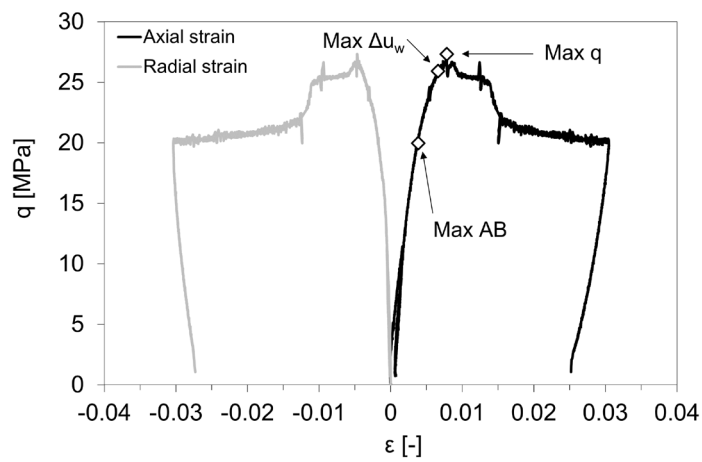
BUL1_1_915_32_P11CTC_U_C



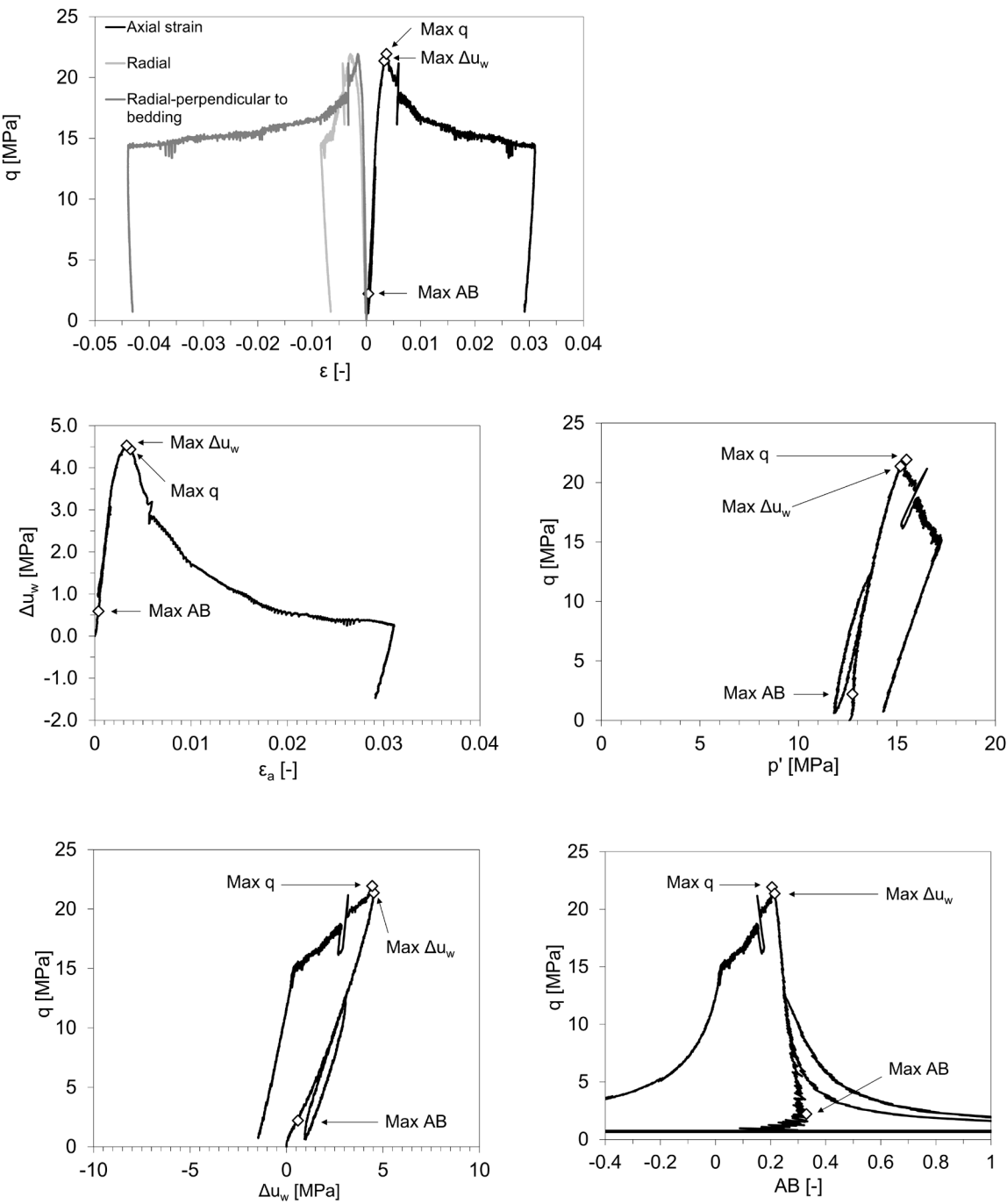
BUL1_1_941_20S10CTCU_C



BUL1_1_941_2S17CTCU_C



BUL1_1_915_38Z13CTCU_C



Appendix D: Mineralogical analysis of the tested samples

Mineralogical analysis and grain density performed by University of Bern on a series of specimens previously mechanically tested in triaxial or oedometric conditions

Nagra ID	S [wt.-%]	C(inorg) [wt.-%]	C(org) [wt.-%]	Quartz [wt.-%]	K-feldspar [wt.-%]	Plagioclase [wt.-%]	Calcite [wt.-%]	Dolomite/Ankerite [wt.-%]	Siderite [wt.-%]	Pyrite [wt.-%]	Clay minerals [wt.-%]	Grain density [g/cm ³]	Grain density STDEV [10 ⁻⁴ g/cm ³]
BUL1_1_896_60S13CTCU_B01	0.43	1.41	0.94	42	7	3	12			0.8	34	2.686	3
BUL1-1_896_66P11CTCU_B06	0.42	1.42	0.99	41	6	3	7	3	1	0.8	36	2.689	3
BUL1_1_878_67S4CTCU_C01	0.09	0.89	0.71	33	6	4	5		2	0.2	48	2.707	3
BUL1_1_878_6S13_CTCU_C02	0.99	0.91	0.65	34	6	4	8			1.9	46	2.696	3
BUL1_1_855_89S5CTCU_C03	0.47	3.63	0.69	41	6	3	29	1		0.9	18	2.691	4
BUL1_1_915_33S10TC_C06-C10	0.52	0.93	1.30	25	6	3	6		2	1.0	55	2.683	7
BUL1_1_892_84S13CTCU_A01	0.72	0.89	0.98	37	6	3	6		2	1.3	44	2.689	4
BUL1_1_892_71S5CTCU_A02	0.11	1.19	0.90	40	6	3	8		2	0.2	39	2.695	3
BUL1_1_892_53S13CTCU_A03	0.00	1.90	0.81	40	6	3	12		4	0.0	33	2.709	4
BUL1_1_970_78S13CTCU_A05	0.07	1.17	0.94	26	6	4	7		3	0.1	53	2.694	4
BUL1_1_892_50_S_OED_A1	0.59	1.56	0.80	42	5	3	10		3	1.1	34	2.700	3
BUL1_1_857_39_S_OED_A2	0.26	2.93	0.62	48	7	4	23	1		0.5	16	2.693	6
BUL1_1_857_39_P_OED_A3	0.25	2.97	0.63	39	6	3	25	< 1		0.5	25	2.704	2
BUL1_1_950_62S5CTC_B10	0.10	1.31	1.12	19	5	3	5		7	0.2	60	2.685	4
BUL1_1_857_49S5CTCU_A7	0.69	2.67	0.70	37	7	3	22			1.3	28	2.692	5
BUL1_1_857_49P5CTCU_A8	0.72	2.31	0.79	36	6	4	19			1.3	32	2.695	5
BUL1_1_892_53S5CTCU_A4	0.13	1.08	0.88	40	6	3	7	2		0.2	41	2.685	2
BUL1_1_948_01P13CTC_U_A10	0.57	1.22	1.12	23	5	3	7		3	1.1	56	2.690	3
BUL1_1_948_08P8CTCU_A11	0.06	0.74	1.31	19	5	3	4		2	0.1	65	2.671	5
BUL1_1_1016_12_P_OED_A4	0.25	1.07	0.91	38	7	3	6	2		0.5	42	2.683	4
BUL1_1_915_33S12CTCU_C7	0.68	0.86	1.32	25	5	3	6		2	1.3	57	2.677	5
BUL1_1_950_71P5CTC_U_B11	0.05	1.06	1.04	19	6	4	9	0	0	0.1	61	2.685	3
BUL1_1_950_86_Z13CTCU_B12	0.06	1.01	1.04	17	6	3	4	0	5	0.1	63	2.684	2
D07OED_BUL11	0.96	6.14	3.83	8	3	2	51	tr	tr	1.8	31	2.576	2
D01OED_BUL11	0.52	2.51	0.53	30	4	2	21	0	0	1.0	41	2.696	5
D03OED_BUL11	0.58	3.06	0.73	23	3	3	23	0	3	1.1	43	2.704	4
D04OED_BUL11	0.48	1.46	0.89	24	6	4	12	0	0	0.9	52	2.669	19
D06OED_BUL11	0.25	0.84	1.04	21	6	4	7	0	0	0.5	61	2.672	7
A05OED_BUL11	0.26	1.52	0.76	27	5	3	8	0	5	0.5	50	2.709	4

Appendix E: Triaxial test results

Tab. E-1: Wave velocities recorded at the beginning of the shearing phase

Nagra ID, sample geometry, mean effective and deviatoric stress (if applicable) at which the measurements were performed, P and S waves in the cylinder axis direction (Vp, Vs), P wave in the direction perpendicular to the axis direction (Vph).

Nagra ID	Sample geometry	p' [MPa]	q [MPa]	Vp [m/s]	Vs [m/s]	Vph [m/s]
BUL1_1_855_89S5CTCU_C	S	3.3	2.3	4'113	2'202	
BUL1_1_855_91S11TC_C	S	9.9	2.5	4'088	2'349	
BUL1_1_855_91S14CTCU_C	S	11.3	4.8	4'280	2'403	
BUL1_1_857_49P5CTCU_A	P	5		3'869	1'453	
BUL1_1_857_49S5CTCU_A	S	5		3'338	1'424	
BUL1_1_878_67S4CTCU_C	S	2.3	1.9	3'267	1'730	
BUL1_1_878_6S13_CTCU_C	S	10.2	2.7	3'481	1'846	
BUL1_1_892_53S13CTCU_A	S	13		3'521	1'870	
BUL1_1_892_53S5CTCU_A	S	5		3'618	1'884	
BUL1_1_892_71S5CTCU_A	S	5		3'482	1'748	
BUL1_1_892_84S13CTCU_A	S	13		3'202	1'451	
BUL1_1_896_60S13CTCU_B	S	13		3'767		4'093
BUL1_1_896_60S13CTCUT_B	S	13.1		3'677		
BUL1_1_896_60S5CTCU_B	S	5.1		3'674		4'321
BUL1_1_896_60S5CTCUT_B	S	5.1		3'580		
BUL1_1_896_66P11CTCU_B	P	11.1		4'104		3'623
BUL1_1_896_66P5CTCU_B	P	5.1		4'010		3'462
BUL1_1_896_80S5TCU_B	S	5.1		3'519		3'949
BUL1_1_915_32_P11CTCU_C	P	11.8	1.2	4'318	2'267	
BUL1_1_915_32Z7CTCU_C	Z	6.1	2.5	3'601	1'677	
BUL1_1_915_33P5CTCU_C	P	3.9	1.8	3'908	1'824	
BUL1_1_915_33S10TC_C	S	5.4	5.3	3'325	1'764	
BUL1_1_915_33S12CTCU_C	S	8.5	3.7	3'301	1'665	
BUL1_1_915_33S5CTCU_C	S	6.5	4	3'252	1'719	
BUL1_1_915_38Z13CTCU_C	Z	11.4	2.9	4'096	1'945	
BUL1_1_915_38Z5CTCU_C	Z	3.6	2.3	3'699	1'779	
BUL1_1_941_20S10CTCU_C	S	11.2	1.4	3'273	2'118	
BUL1_1_941_2S17CTCU_C	S	14.6	2.7	3'389	1'873	
BUL1_1_948_01P13CTC_U_A	P	13		4'150	2'311	
BUL1_1_948_08P8CTCU_A	P	8		3'810	1'466	
BUL1_1_948_08P9TC_A	P	9		3'961	2'157	
BUL1_1_950_62S13CTCU_B	S	13.1		3'070		4'141
BUL1_1_950_62S5CTC_B	S	5.1		2'952		3'949
BUL1_1_950_62S9CTCU_B	S	9.1		3'020		4'069
BUL1_1_950_71P5CTC_U_B	P	5.1		3'835		3'063
BUL1_1_950_86_Z13CTCU_B	Z	13.1		3'576		3'289
BUL1_1_950_86Z8CTCU_B	Z	8.1		3'510		3'248
BUL1_1_970_78S13CTCU_A	S	13		3'346	1'686	
BUL1_1_970_78S5CTCU_A	S	5		3'065	1'520	
BUL1_1_1016_02S11CTCU_A	S	11		3'527	1'582	

Sample ID and corresponding initial characteristics are reported in Tab. 6-2, along with the results from the triaxial tests from the pre-shearing phases. Tab. E-2 includes:

- Sample ID
- Core depth
- Formation
- Sample geometry
- Bulk modulus ρ_{bulk} [g/cm³]
- Initial void ratio, *computed using a reference solid density of 2.70 g/cm³
- Initial and final water content
- Results of the Skempton's B check (B value and average mean effective stress at which it was obtained)
- Swelling pressure obtained at the end of the saturation (radial and axial)
- Consolidation coefficient, obtained during the consolidation phase, and corresponding mean effective stress

Tab. E-3 presents the results from the shearing phases, including:

- Undrained elastic properties: Elastic modulus [GPa], Small Strain elastic modulus [GPa], Ratio of radial to axial strain [-], Ratio of radial (parallel to bedding) to axial strain [-] (only for P samples), Mean effective stress at which the elastic properties were computed [MPa]
- Drained elastic properties: Young's modulus E [GPa], Poisson's ratios $\nu_{\perp}$ (ratio of radial to axial strain) [-] and $\nu_{\parallel}$ (ratio of radial (parallel to bedding) to axial strain) [-], and mean effective stress at which the elastic properties were computed
- Peak strength: Mean effective stress and deviatoric stress at peak
- Maximum AB value: Mean effective stress and deviatoric stress at the maximum AB value
- Maximum pore pressure: Mean effective stress and deviatoric stress attained at the maximum pore pressure
- Post peak strength: Mean effective stress and deviatoric stress

Tab. E-2: Triaxial test results (part 1)

Nagra ID	Sample geometry	ρ_{bulk} [g/cm ³]	e_0 [-]	Water content		B check		Swelling pressure		Consolidation phase		Strain rate $\dot{\epsilon}$ [s ⁻¹]	Stress path
				Initial [%]	Final [%]	p' [MPa]	B [-]	Axial [MPa]	Radial [MPa]	p' [MPa]	c_v [mm ² /s]		
BUL1_1_855_89S5CTCU_C	S	2.59	0.072	2.7	2.8	5.0	0.79					1E-07	CTCU
BUL1_1_855_91S11TC_C	S	2.59	0.076	3.0	3.4	7.0	0.76					2E-08	TC drained
BUL1_1_855_91S14CTCU_C	S	2.59	0.070	2.6	2.2	14.0	0.54					1E-07	CTCU
BUL1_1_857_49P5CTCU_A	P	2.53	0.106	3.8	4.9	21.0	0.64	20.5	11.7	17.0	4.0E-04	2E-08, 1E-07	CTC + CTCU
BUL1_1_857_49S5CTCU_A	S	2.53	0.110	4.0	4.9	21.0	0.64	21.0	8.0	5.0	8.5E-03	9E-08	CTCU
BUL1_1_878_67S4CTCU_C	S	2.52	0.126	5.2	4.8	4.4	0.94					1E-07	CTCU
BUL1_1_878_6S13_CTCU_C	S	2.53	0.117	4.8	4.2	11.5	0.82					1E-07	CTCU
BUL1_1_892_53S13CTCU_A	S	2.56	0.090	3.4	3.6	14.3	0.55	16.8	5.0	13.0	2.1E-02	1E-07	CTCU
BUL1_1_892_53S5CTCU_A	S	2.58	0.080	3.2	4.0	16.0	0.61	16.1	4.8	5.0	7.0E-03	1E-07	CTCU
BUL1_1_892_71S5CTCU_A	S	2.53	0.108	3.8	4.1	20.3	0.56	20.5	9.2	5.0	5.9E-03	1E-07	CTCU
BUL1_1_892_84S13CTCU_A	S	2.51	0.123	4.4	5.4	13.0	0.61	21.4	7.9			1E-07	CTCU
BUL1_1_896_60S13CTCU_B	S	2.55	0.099	3.9	4.1	14.5	0.87	14.6	9.8	13.1	2.5E-03	9E-08	CTCU
BUL1_1_896_60S13CTCUT_B	S	2.55	0.100	3.8	3.8	15.8	0.91	15.8	9.8	13.0	2.3E-03	9E-08	CTCU
BUL1_1_896_60S5CTCU_B	S	2.56	0.095	3.8	4.1	14.9	0.87	14.8	9.8	5.0	1.8E-03	9E-08	CTCU
BUL1_1_896_60S5CTCUT_B	S	2.56	0.092	3.6	3.9	13.1	1.20	13.3	8.0	5.0	1.8E-03	9E-08	CTCU
BUL1_1_896_66P11CTCU_B	P	2.56	0.099	4.2	4.0	15.0	0.66	15.3	13.0	11.0	1.9E-03	9E-08	CTCU
BUL1_1_896_66P5CTCU_B	P	2.56	0.096	3.7	4.0	16.0	0.81	16.1	13.2	5.0	1.2E-03	1E-07	CTCU
BUL1_1_896_80S5TCU_B	S	2.54	0.104	3.9	4.2	16.7	0.78	16.7	11.2	5.0	1.2E-03	9E-08	TCU
BUL1_1_915_32_P11CTCU_C	P	2.53	0.117	4.6	4.7	10.0	0.69					1E-07	CTC + CTCU
BUL1_1_915_32Z7CTCU_C	Z (60)	2.55	0.109	4.8	5.0	11.0	0.80					1E-07	CTCU
BUL1_1_915_33P5CTCU_C	P	2.53	0.113	4.4	3.2	3.5	0.92					1E-07	CTCU
BUL1_1_915_33S10TC_C	S	2.52	0.123	5.0	5.0	7.6	0.87			7.7	1.6E-04	2E-08	TC drained
BUL1_1_915_33S12CTCU_C	S	2.52	0.124	5.0	4.9	12.0	0.82					1E-07	CTCU
BUL1_1_915_33S5CTCU_C	S	2.53	0.118	4.8	5.0	3.5	0.83					2E-08, 1E-07	CTCU + CTC
BUL1_1_915_38Z13CTCU_C	Z (60)	2.53	0.112	4.3	4.3	6.0	0.82			12.6	6.5E-05	9E-08	CTCU
BUL1_1_915_38Z5CTCU_C	Z (60)	2.53	0.124	5.6	4.9	2.0	0.94			4.4	1.3E-04	1E-07	CTCU
BUL1_1_941_20S10CTCU_C	S	2.50	0.133	4.9	4.4	9.0	0.84					2E-08, 1E-07	CTC + CTCU
BUL1_1_941_2S17CTCU_C	S	2.50	0.136	5.1	4.5	12.0	0.89			16.7	1.8E-04	1E-07	CTCU
BUL1_1_948_01P13CTC_U_A	P	2.51	0.127	4.7	5.4	12.7	0.80	13.4	4.7	13.0	1.7E-02	2E-08, 1E-07	CTC + CTCU
BUL1_1_948_08P8CTCU_A	P	2.46	0.156	5.5	6.1	33.0	0.67	33.0	21.0	8.0	3.8E-03	1E-07	CTCU
BUL1_1_948_08P9TC_A	P	2.47	0.153	5.5	6.6	34.0	0.80	34.0	9.0	9.0	5.9E-03	2E-08	TC drained
BUL1_1_950_62S13CTCU_B	S	2.53	0.122	5.3	5.6	25.0	0.79	26.0	17.4	13.0	1.1E-03	9E-08	CTCU
BUL1_1_950_62S5CTC_B	S	2.54	0.122	5.6	5.8	26.0	0.78	25.9	17.9	5.1	6.9E-04	2E-08, 1E-07	CTCU + CTC
BUL1_1_950_62S9CTCU_B	S	2.53	0.126	5.4	5.5	22.5	0.74	22.3	16.6	9.0	1.4E-03	9E-08	CTCU
BUL1_1_950_71P5CTC_U_B	P	2.51	0.130	5.1	5.5	23.0	0.74	22.5	22.5	5.0	4.5E-04	2E-08, 1E-07	CTC + CTCU
BUL1_1_950_86_Z13CTCU_B	Z (60)	2.51	0.133	5.3	5.5	25.0	0.79	25.0	23.0	13.1	9.7E-04	1E-07	CTCU
BUL1_1_950_86Z8CTCU_B	Z (60)	2.53	0.126	5.4	5.5	25.0	0.74	25.0	22.3	8.0	7.5E-04	9E-08	CTCU
BUL1_1_970_78S13CTCU_A	S	2.52	0.118	4.5	5.1	27.0	0.54	25.9	13.6	13.0	4.1E-03	1E-07	CTCU
BUL1_1_970_78S5CTCU_A	S	2.53	0.111	4.2	5.0	26.0	0.57	26.0	12.0	10.0	1.1E-03	1E-07	CTCU
BUL1_1_1016_02S11CTCU_A	S	2.60	0.075	3.4	4.5	34.0	0.61	34.9	16.2	11.0	1.0E-02	1E-07	CTCU

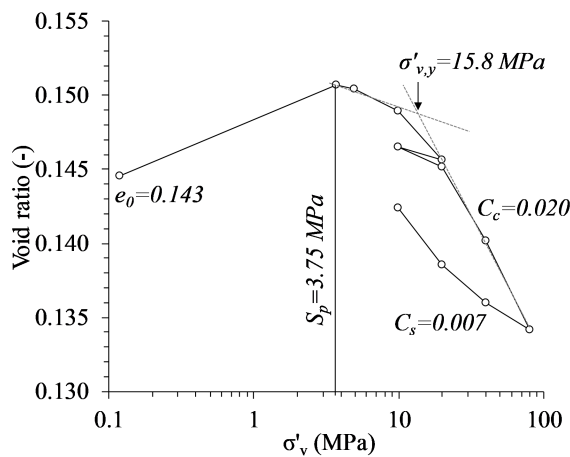
Tab. E-3: Triaxial test results (part 2)

^dDrained elastic modulus

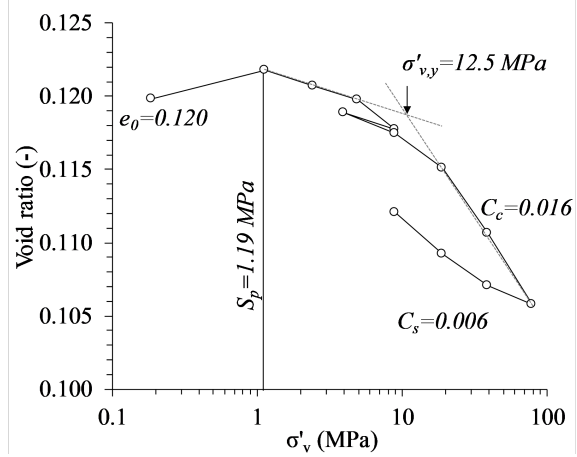
Nagra ID	Undrained elastic properties					Drained elastic properties				Peak strength [MPa]		Max. AB value [MPa]		Max. pore pressure [MPa]		Post peak strength [MPa]		Post peak elastic properties	
	p' [MPa]	Elastic modulus [GPa]	Small Strain modulus [GPa]	Ratio radial to axial strain [-]	Ratio radial (parallel to bedding) to axial strain [-]	p' [MPa]	E [GPa]	ν_L [-]	$\nu_{//}$ [-]	q [MPa]	p' [MPa]	q [MPa]	p' [MPa]	q [MPa]	p' [MPa]	q [MPa]	p' [MPa]	p' [MPa]	E [GPa]
BUL1_1_855_89S5CTCU_C	9.2	14.1	18	0.27						57.4	30.7	1.2	4.9	19.8	7.1	50.5	30.8	30.7	12.3
BUL1_1_855_91S11TC_C										35.1	11.5					18.8	11.9	11.8	6.9 ^d
BUL1_1_855_91S14CTCU_C	22	20	24.6	0.54						76.0	40.9	11.5	14.0	40.4	19.9	47.0	37.0	37.0	12.5
BUL1_1_857_49P5CTCU_A	6.4	9.9	11.8	0.547	0.346					36.6	16.9	3.6	5.5	22.6	10.3	21.2	17.2	16.0	7.3
BUL1_1_857_49S5CTCU_A	5.4	7.6	11.6	0.415						29.6	16.2	4.4	5.3	17.0	7.8	22.4	16.8	18.7	6.2
BUL1_1_878_67S4CTCU_C	3.6	4.22	5.3	0.43						13.4	6.6	1.4	3.9	10.0	4.1	11.4	9.4	9.4	4.2
BUL1_1_878_6S13_CTCU_C	12.4	7.2	12.3	0.41						34.5	20.6	3.8	12.2	22.9	12.6	21.1	21.0	20.9	3.9
BUL1_1_892_53S13CTCU_A	15.5	11.1	14.4	0.255						44.7	24.6	22.7	15.2	33.9	17.8	36.3	28.9	29.0	11.7
BUL1_1_892_53S5CTCU_A	5.6	13.9	16.8	0.5						43.1	23.3	6.3	5.5	22.6	9.2	30.1	23.9	23.9	7.1
BUL1_1_892_71S5CTCU_A	6.1	8.5	9.6	0.38						32.5	17.0	5.2	5.2	19.0	7.7	20.2	17.4	17.4	6.1
BUL1_1_892_84S13CTCU_A	15.6	10	14.98	0.35						31.6	18.8	21.6	14.5	27.1	15.5	25.7	24.3	24.3	6.0
BUL1_1_896_60S13CTCU_B	15.2	15.4	16.9	0.38						51.3	32.9	15.0	14.6	31.6	17.9	33.1	29.7	29.2	7.6
BUL1_1_896_60S13CTCUT_B	14.0	13.6	16.3	0.25						48.5	30.4	6.2	13.6	29.9	17.7	33.8	28.9	28.9	8.9
BUL1_1_896_60S5CTCU_B	5.6	9.1	12.6	0.34						42.9	24.7	3.9	5.4	20.6	8.9	33.4	25.0	25.0	9.1
BUL1_1_896_60S5CTCUT_B	5.3	8.9	12.8	0.25						43.8	25.7	2.9	5.2	18.8	8.2	25.5	21.1	21.1	6.7
BUL1_1_896_66P11CTCU_B	13.4	17.1	23.5	0.32	0.24					52.7	29.6	1.0	11.3	31.5	19.2	33.4	26.7	26.6	12.4
BUL1_1_896_66P5CTCU_B	7.4	13.7	19.5	0.36	0.29					45.5	23.3	4.2	6.1	20.6	10.7	25.2	19.0	19.0	9.7
BUL1_1_896_80S5CTCU_B										28.9	12.1	2.9	5.1	0.2	5.1				
BUL1_1_915_32_P11CTCU_C	13.1	11.2	24	0.267	0.395	14	10.2	0.4	0.38	40.2	21.9	10.2	13.5	34.5	19.6	24.7	23.4	23.2	11.2
BUL1_1_915_32Z7CTCU_C										12.3	8.9	2.8	7.2	11.2	8.3	9.5	11.1		
BUL1_1_915_33P5CTCU_C	6.3	11.8	36	0.86	0.98					33.4	12.9	5.3	5.1	33.4	12.9	0.0	0.0		
BUL1_1_915_33S10TC_C										21.2	8.9	0.1	8.9	7.4	8.9	8.8	8.9	3.3	9.0 ^d
BUL1_1_915_33S12CTCU_C	10.1	8.1	10.5	0.32						26.5	12.8	15.1	10.1	23.1	10.2	19.1	18.8	19.2	4.8
BUL1_1_915_33S5CTCU_C	5.3	5.1	5.7	0.39		7.7	2.83	0.26		25.5	13.7	0.8	5.1	7.7	4.2	8.4	8.1	7.7	2.8 ^d
BUL1_1_915_38Z13CTCU_C										21.9	15.5	2.3	12.7	21.4	15.2	14.6	17.2		
BUL1_1_915_38Z5CTCU_C15										10.8	5.9	0.8	4.4	9.4	5.4	7.9	6.9		
BUL1_1_941_20S10CTCU_C	9.7	7.2	9.1	0.23		13.4	6.57	0.17		23.8	10.8	8.0	9.7	22.0	9.9			26.2	6.3
BUL1_1_941_2S17CTCU_C	15.4	9.3	10.7	0.38						27.3	15.2	20.0	14.2	25.9	14.5	20.0	19.5	19.3	3.7
BUL1_1_948_01P13CTC_U_A	14.6	15.6	15.7	0.42	0.28	15.4	13.9	0.28	0.21	40.4	23.4	20.8	17.4	35.6	21.2	19.1	20.2	20.3	8.1
BUL1_1_948_08P8CTCU_A	10.0	16.7	19.2	0.57	0.15					36.0	18.7	7.3	9.9	26.9	15.5	21.4	17.5	18.5	6.4 ^d
BUL1_1_948_08P9TC_A										23.7	9.0	0.1	9.0	0.1	9.0	9.7	9.0		
BUL1_1_950_62S13CTCU_B	12.3	9.3	12.3	0.3						30.9	14.8	9.6	12.3	28.3	12.8	19.7	18.8	18.8	5.2
BUL1_1_950_62S5CTC_B	4.7	4.6	6.06	0.30		9.4	4.6	0.36		31.8	16.1	0.5	4.8	4.9	4.7	7.4	7.9	8.0	1.8 ^d
BUL1_1_950_62S9CTCU_B	8.2	8.2	9.4	0.26						28.2	11.6	6.2	8.5	25.5	9.9	17.1	13.9	13.7	5.0
BUL1_1_950_71P5CTC_U_B	6.8	12.6	17.4	0.51	0.23	7.3	11.8	0.7	0.3	36.2	15.8	6.9	6.7	28.4	12.7	14.3	11.4	11.4	9.9
BUL1_1_950_86_Z13CTCU_B										28.6	18.0	8.7	14.3	28.3	17.9	14.9	15.7		
BUL1_1_950_86Z8CTCU_B										22.2	12.2	6.7	8.8	21.2	11.4	12.4	11.9		
BUL1_1_970_78S13CTCU_A	14.2	11.8	13.3	0.32						33.8	17.8	22.4	13.7	30.4	14.8	22.0	20.4	20.4	6.7
BUL1_1_970_78S5CTCU_A	4.7	6.9	9.4	0.41						21.5	8.1	5.4	4.7	19.5	6.9	15.0	11.5	13.0	3.4
BUL1_1_1016_02S11CTCU_A	11.9	13	19.3	0.27						49.6	27.5	7.1	11.6	33.4	16.2	31.0	28.9	29.0	7.9

Appendix F: Test results in oedometric conditions

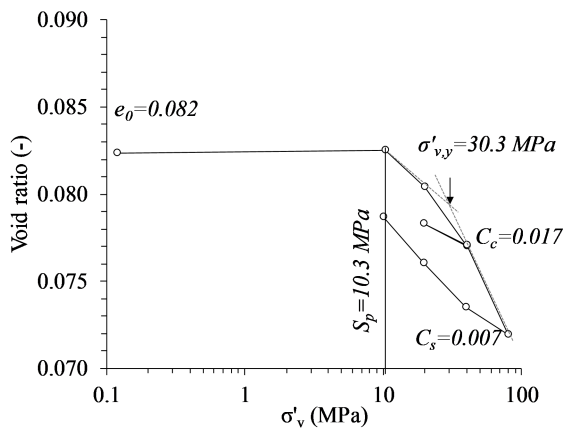
Results of the oedometric test results are here reported in terms of oedometric curves (void ratio at the end of the primary consolidation) obtained for each test. In each graph the void ratio (e) versus the vertical effective stress (σ'_v) are reported. In each graph, it is highlighted the initial void ratio, the swelling pressure attained at the end of the resaturation phase (S_p), the yield stress ($\sigma'_{v,y}$) obtained through the Casagrande's construction, the compressibility (C_c) and the unloading (C_s) indexes.



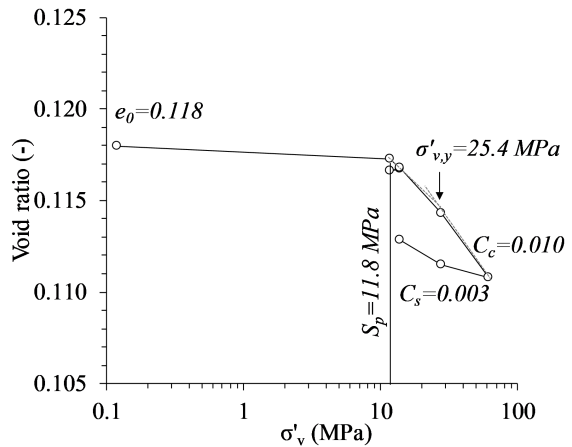
D01OED_BUL1_1



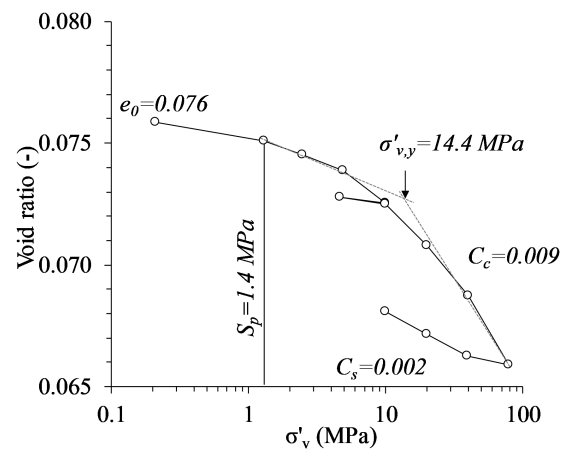
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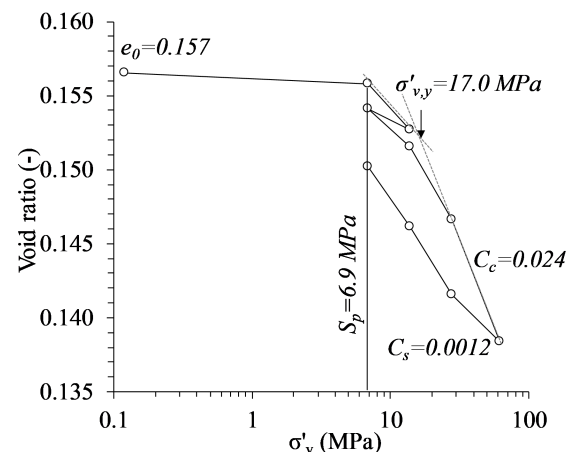
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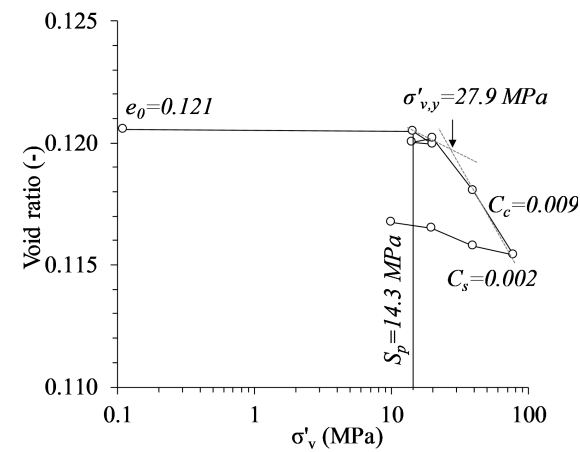
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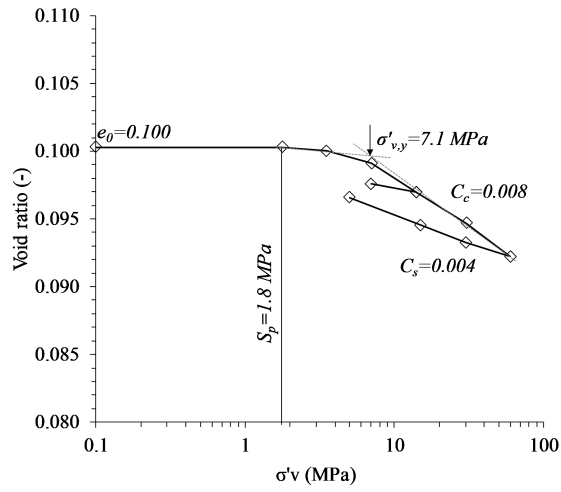
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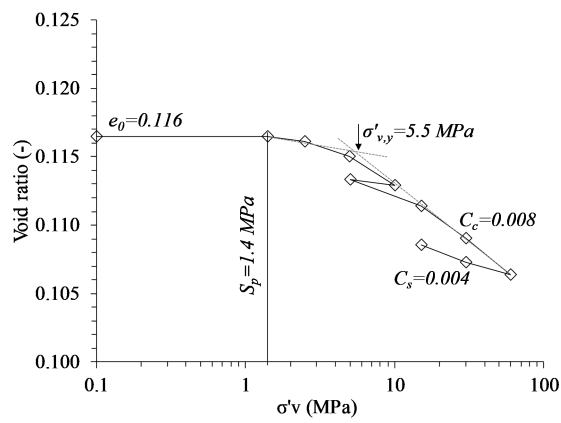
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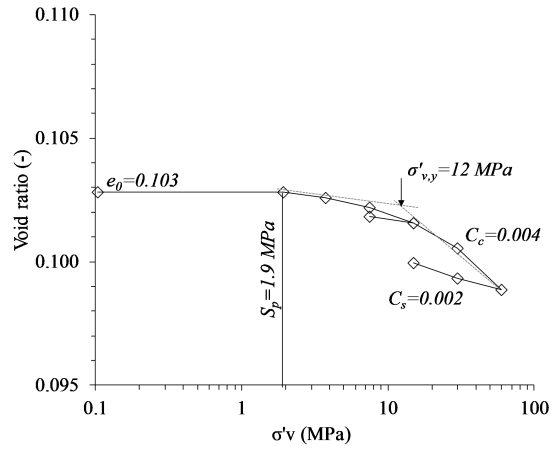
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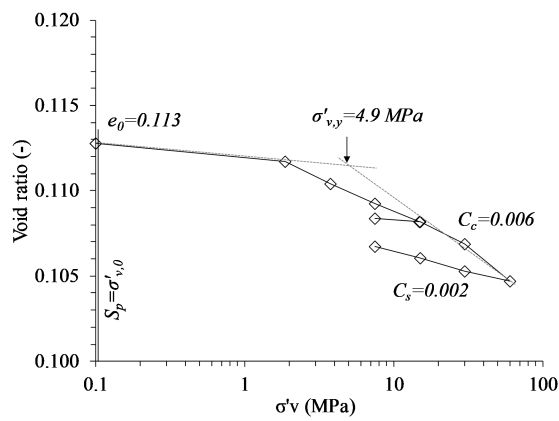
A01OED_BUL1_1



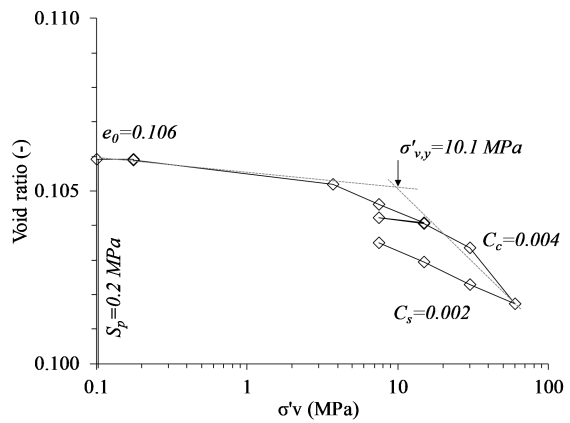
A02OED_BUL1_1



A03OED_BUL1_1

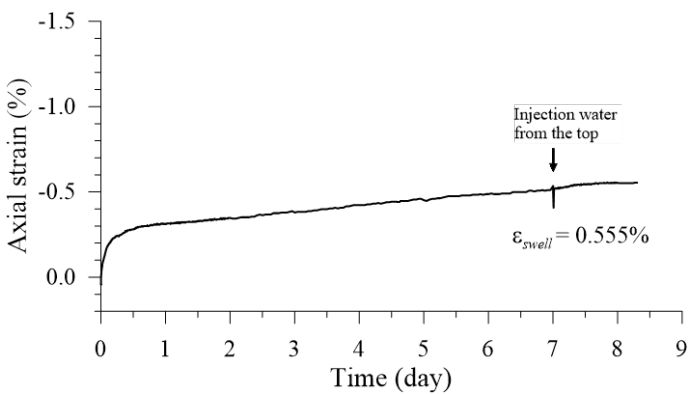


A04OED_BUL1_1

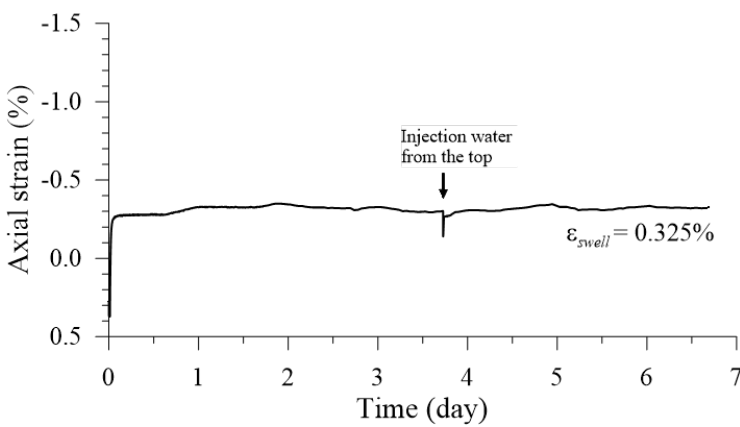


A05OED_BUL1_1

The results of the one-dimensional swelling tests are reported below. Lab D performed three tests, applying a small constant load on the sample, and injecting pressure at 50 kPa. Lab A reported issues in the first two tests: a leakage was reported, not allowing to reach constant pore pressure, and in the first phases the pore fluid pressure was equivalent to the axial stress applied, inducing a movement of the piston. Therefore, the strains reported in the initial phases have been affected and may be not representative of the real material response. A lower pore pressure was used for tests A03 and A04. However, test 4 reported a non-expected pore pressure fluctuation.

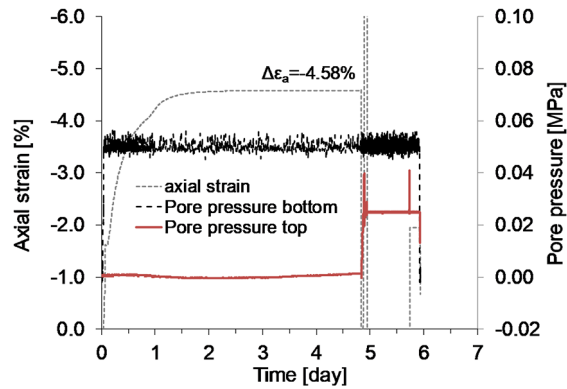


D02OS_BUL1_1

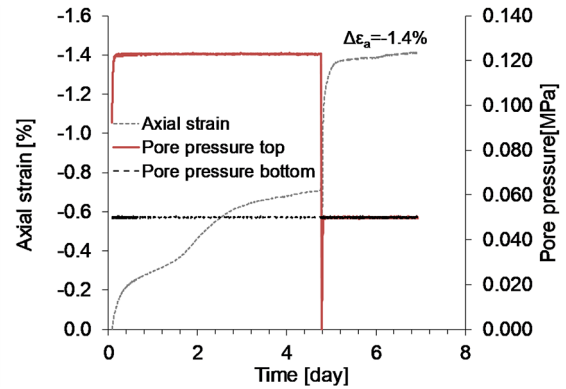


D03OS_BUL1_1

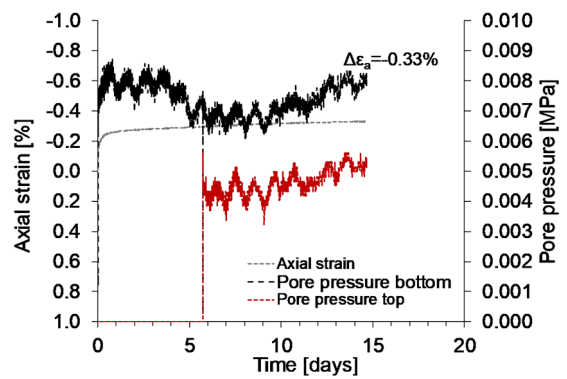
For the one-dimensional swelling tests from lab A, the pore pressure evolution is reported together with the strain evolution.



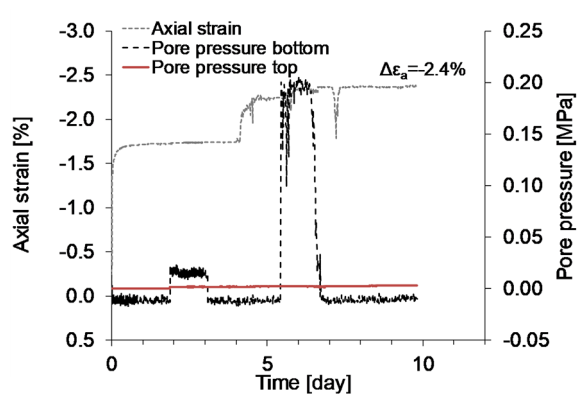
A01OS_BUL1_1



A02OS_BUL1_1



A03OS_BUL1_1



A04OS_BUL1_1