

Arbeitsbericht NAB 21-20

**TBO Marthalen-1-1:
Data Report**

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

September 2021

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

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www.nagra.ch

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

September 2021

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geomechanics, geomechanical laboratory testing,
triaxial tests, oedometric tests

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Nagra Arbeitsberichte ("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 (test coordination, formal analysis, QC and writing)

S. Giger (project administration, conceptualisation and writing)

L. Laloui (review and supervision)

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

Editorial work: P. Blaser and M. Unger

The Dossier has greatly benefitted from technical discussions with external and internal experts. Their input and work are very much appreciated.

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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 ("Tiefbohrungen", TBO) 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 Marthalen-1-1 borehole.

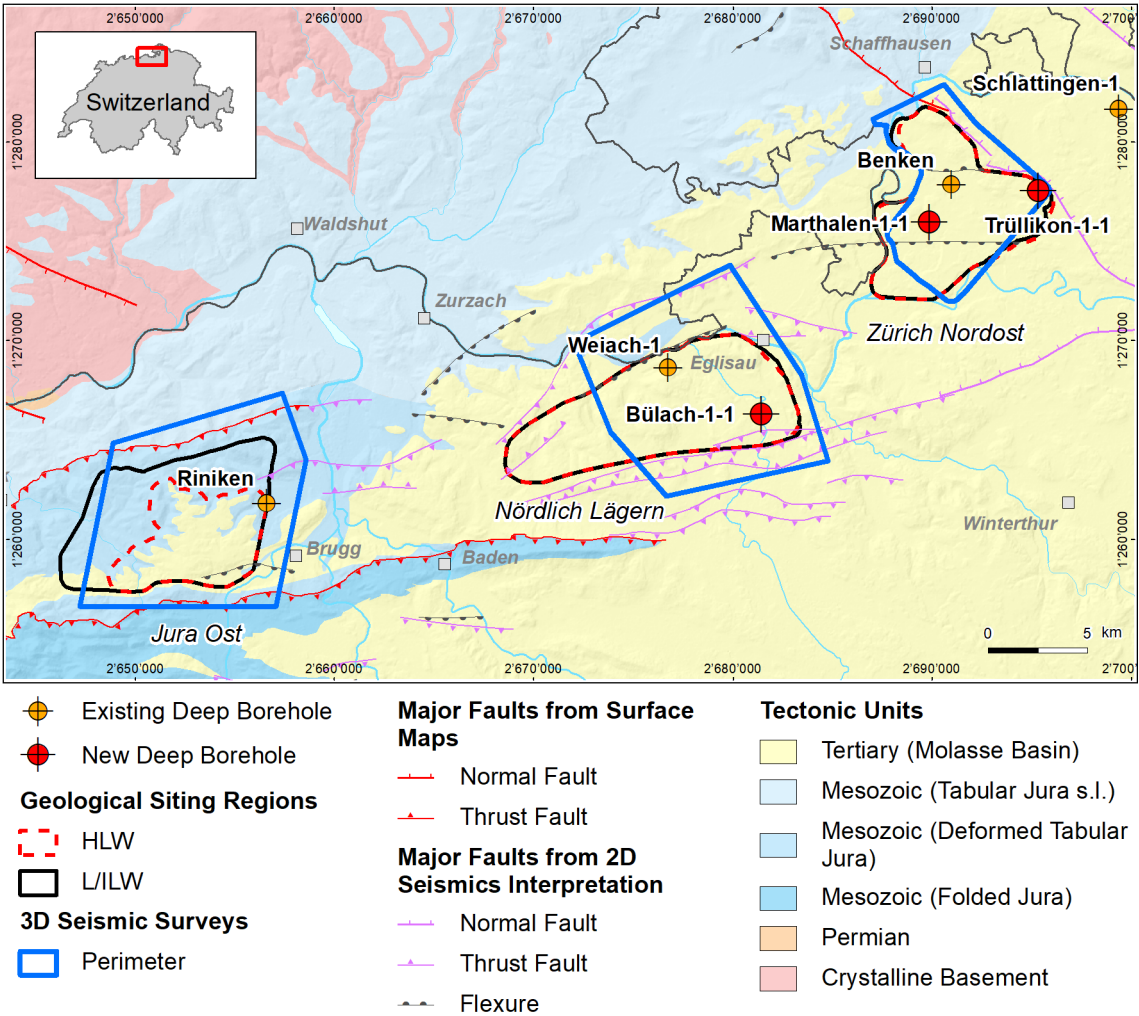


Fig. 1-1: Tectonic overview map with the three siting regions under investigation

1.2 Location and specifications of the borehole

The Marthalen-1-1 (MAR1-1) exploratory borehole is the third borehole drilled within the framework of the TBO project. The drill site is located in the western part of the Zürich Nordost siting region (Fig. 1-2). The vertical borehole reached a final depth of 1'099.25 m (MD)¹. The borehole specifications are provided in Tab. 1-1.

Tab. 1-1: General information about the MAR1-1 borehole

Siting region	Zürich Nordost
Municipality	Marthalen (Canton Zürich / ZH), Switzerland
Drill site	Marthalen-1 (MAR1)
Borehole	Marthalen-1-1 (MAR1-1)
Coordinates	LV95: 2'689'889.946 / 1'275'956.932
Elevation	Ground level = top of rig cellar: 399.48 m above sea level (asl)
Borehole depth	1'099.25 m measured depth (MD) below ground level (bgl)
Drilling period	9th February 2020 – 14th July 2020 (spud date to end of rig release)
Drilling company	Daldrup & Söhne AG
Drilling rig	Wirth B 152t
Drilling fluid	Water-based mud with various amounts of different components such as ² : 55 – 460 m: Bentonite & polymers 460 – 881 m: Potassium silicate & polymers 881 – 961 m: Sodium silicate & polymers 961 – 1'099.25 m: Sodium chloride & polymers

The lithostratigraphic profile and the casing scheme are shown in Fig. 1-3. The comparison of the core versus log depth³ of the main lithostratigraphic boundaries in the MAR1-1 borehole is shown in Tab. 1-2.

¹ Measured depth (MD) refers to the position along the borehole trajectory, starting at ground level, which for this borehole is the top of the rig cellar. For a perfectly vertical borehole, MD below ground level (bgl) and true vertical depth (TVD) are the same. In all Dossiers depth refers to MD unless stated otherwise.

² For detailed information see Dossier I.

³ Core depth refers to the depth marked on the drill cores. Log depth results from the depth observed during geophysical wireline logging. Note that the petrophysical logs have not been shifted to core depth, hence log depth differs from core depth.

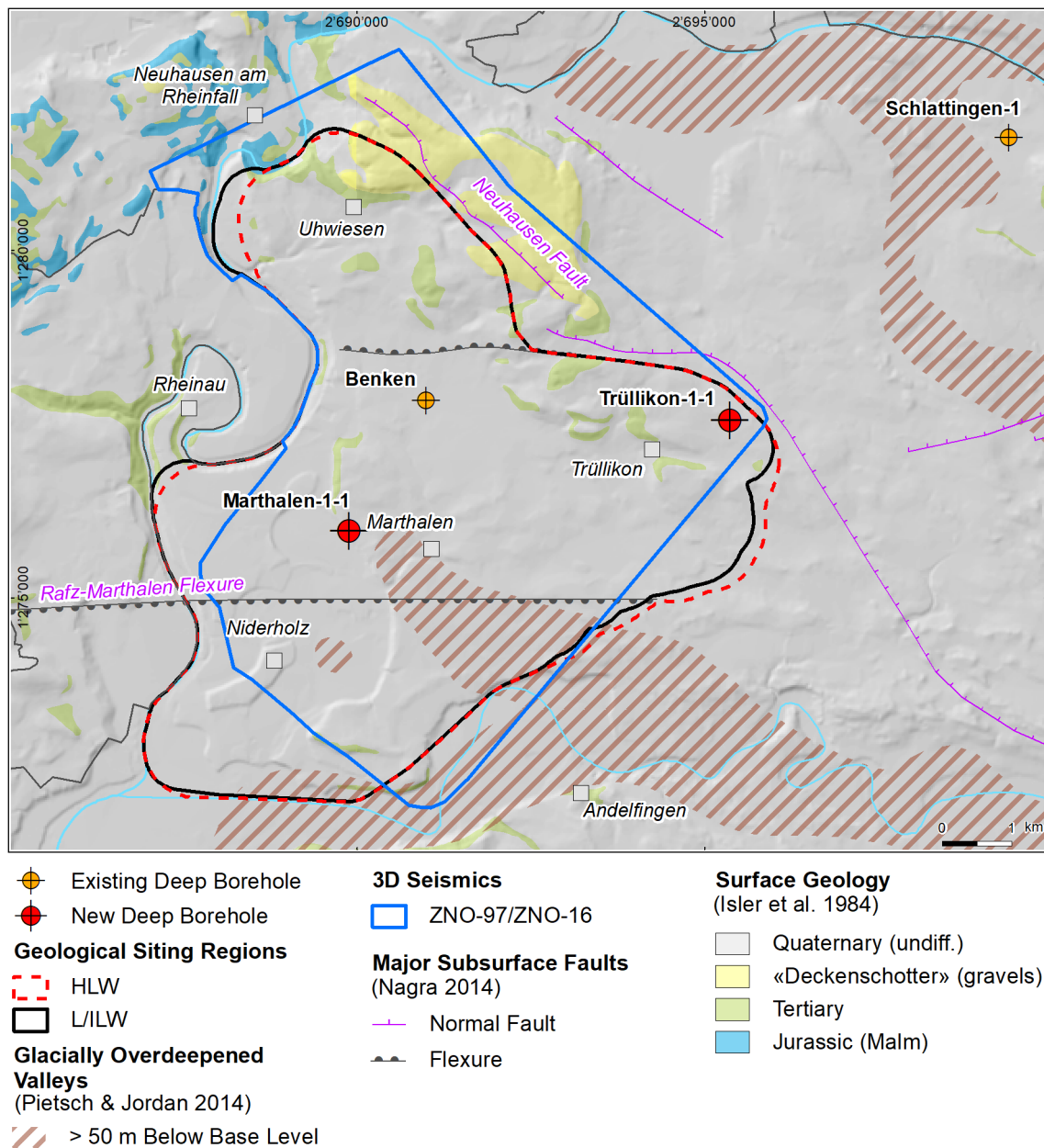


Fig. 1-2: Overview map of the investigation area in the Zürich Nordost siting region with the location of the MAR1-1 borehole in relation to the boreholes Benken, Schlattigen-1 and TRU1-1

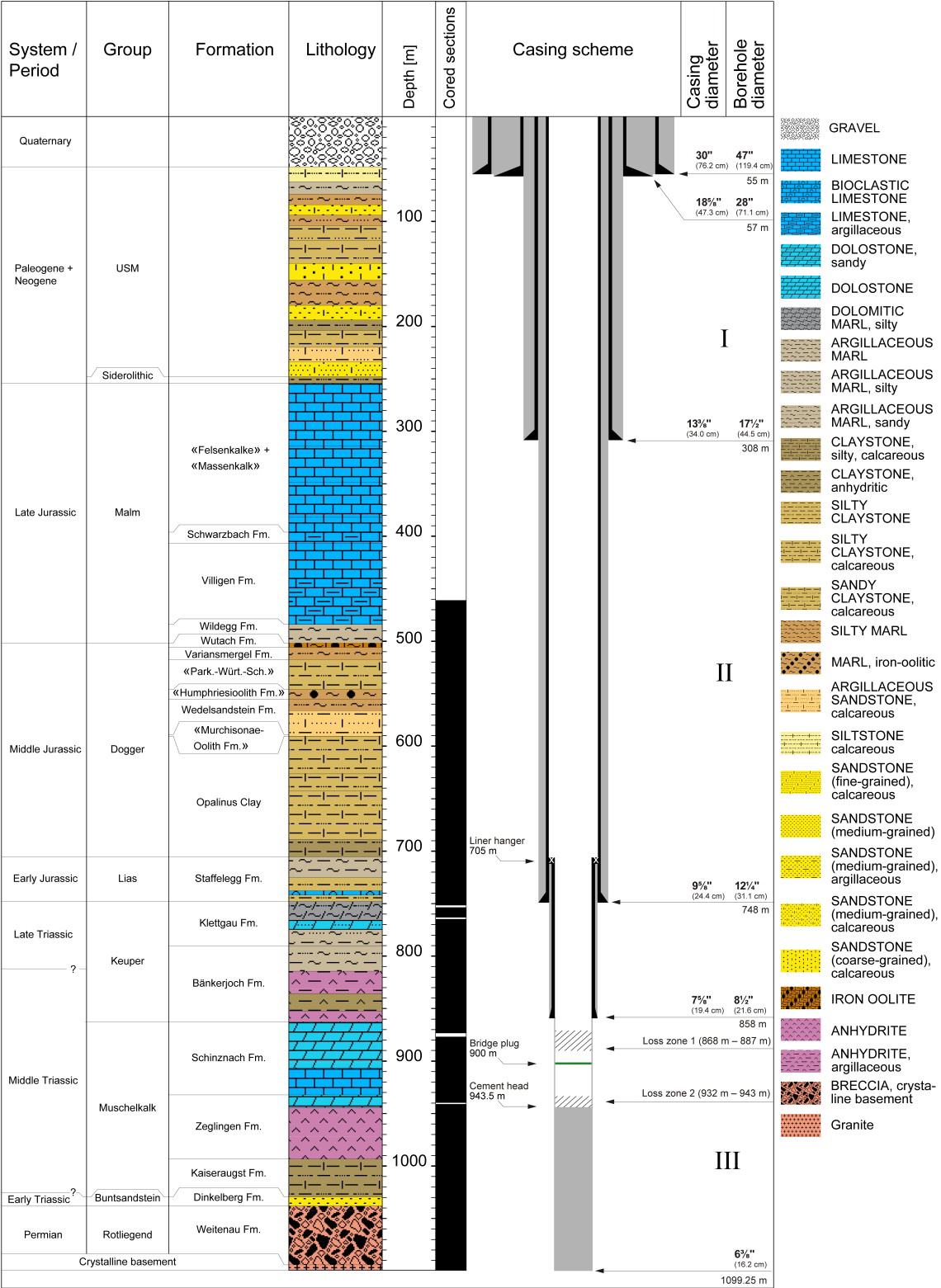


Fig. 1-3: Lithostratigraphic profile and casing scheme for the MAR1-1 borehole⁴

⁴ For detailed information see Dossier I and III.

Tab. 1-2: Core and log depth for the main lithostratigraphic boundaries in the MAR1-1 borehole⁵

System / Period	Group	Formation	Core depth in m (MD)	Log
Quaternary			48	—
Paleogene + Neogene	USM		248	—
	Siderolithic		254	—
		«Felsenkalke» + «Massenkalk»	396.1	—
	Malm	Schwarzbach Formation	406.9	—
		Villigen Formation	484.02	484.22 —
		Wildeggen Formation	501.80	501.92 —
		Wutach Formation	505.75	506.03 —
		Variansmergel Formation	517.43	517.57 —
Jurassic		«Parkinsoni-Württembergica-Schichten»	545.93	546.12 —
	Dogger	«Humphriesioolith Formation»	555.23	555.47 —
		Wedelsandstein Formation	589.17	589.19 —
		«Murchisonae-Oolith Formation»	590.35	590.37 —
		Opalinus Clay	705.40	705.52 —
	Lias	Staffellegg Formation	747.83	747.89 —
	Keuper	Klettgau Formation	790.12	790.34 —
		Bänkerjoch Formation	862.52	862.77 —
		Schinznach Formation	932.24	932.47 —
Triassic	Muschelkalk	Zeglingen Formation	993.50	993.97 —
		Kaiseraugst Formation	1029.45	1029.95 —
	Buntsandstein	Dinkelberg Formation	1037.98	1038.23 —
Permian	Rotliegend	Weitenau Formation	1094.08	1094.08 —
		Crystalline Basement	1099.25	final depth

⁵ For details regarding lithostratigraphic boundaries see Dossier III and IV; for details about depth shifts (core gonio-metry) see Dossier V.

1.3 Documentation structure for the MAR1-1 borehole

NAB 21-20 documents the majority of the investigations carried out in the MAR1-1 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. 1-3).

This documentation aims at early publication of the data collected in the MAR1-1 borehole. It includes most 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 results 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. 1-3: List of dossiers included in NAB 21-20

Black indicates the dossier at hand.

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

1.4 Structure of Dossier IX

The dossier at hand is organised as follows:

- Chapter 2 presents the aims of the laboratory testing programmes 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 MAR1-1 borehole comprised the following testing methods:

- Indirect tensile strength (Brazilian)
- Uniaxial compressive strength (UCS)
- Triaxial compressive strength (TRX)

The following laboratory was contracted:

- Gesteinslabor Dr. Eberhard Jahns (Germany). Main contacts were Hansjörg Baumgartner and Sarah Louis.

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 MAR1-1 borehole:

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

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.

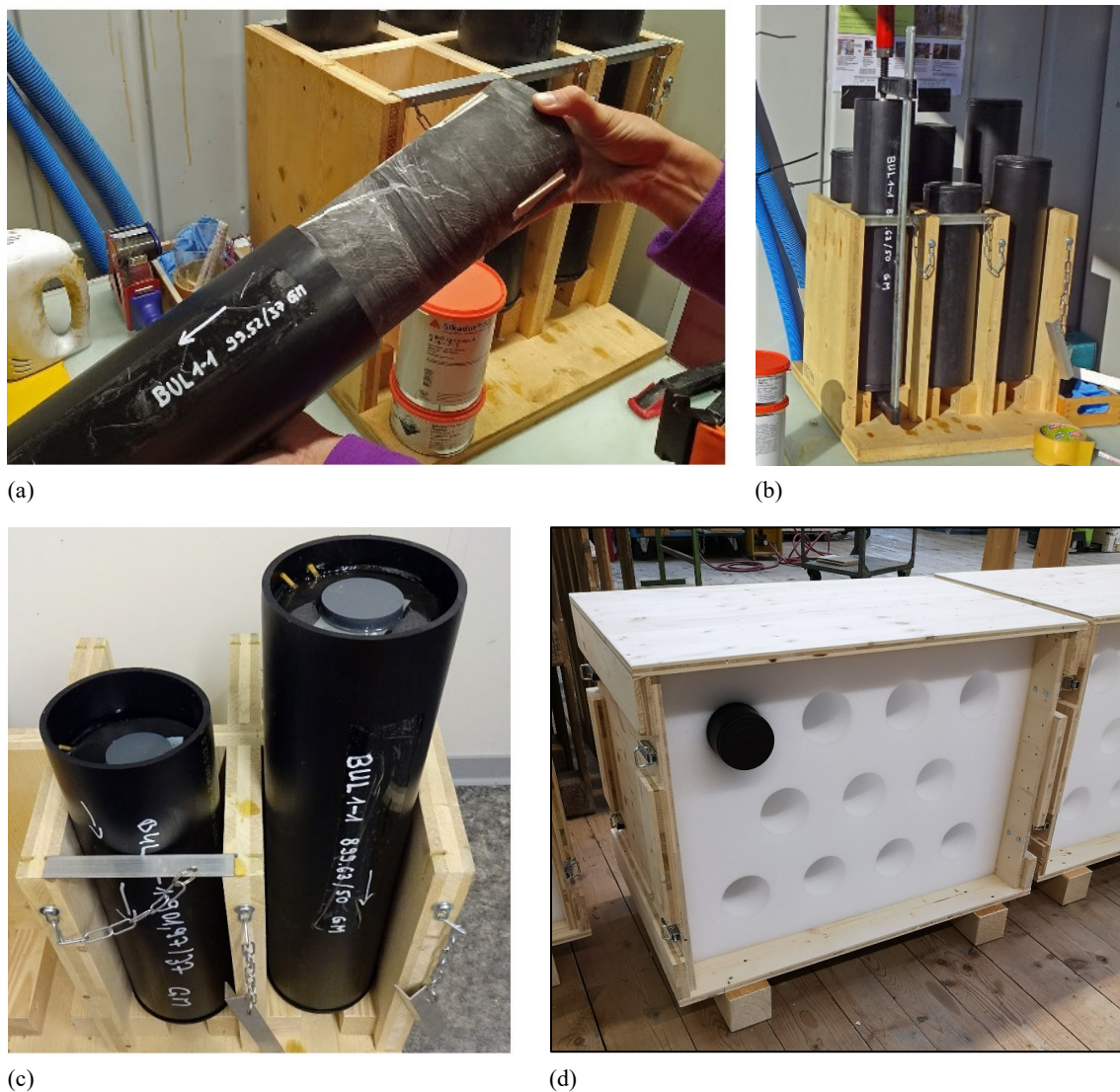


Fig. 3-1: Sampling and storage workflow of GM cores

(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. Example pictures are from borehole Bülach-1-1 (Nagra 2021).

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 to be very small (nominal 3.2 mm) to ensure that no excessive temperature from curing of the epoxy develops⁶. Further details of the conditioning procedure are provided in the field manual (Rufer 2019).

A total of 88 GM cores with a cumulative length of 38 metres were conditioned in this way. Additional 10 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 (Chapter 6 and Appendix B).

⁶ Maximum measured temperature in the laboratory was <27 °C.

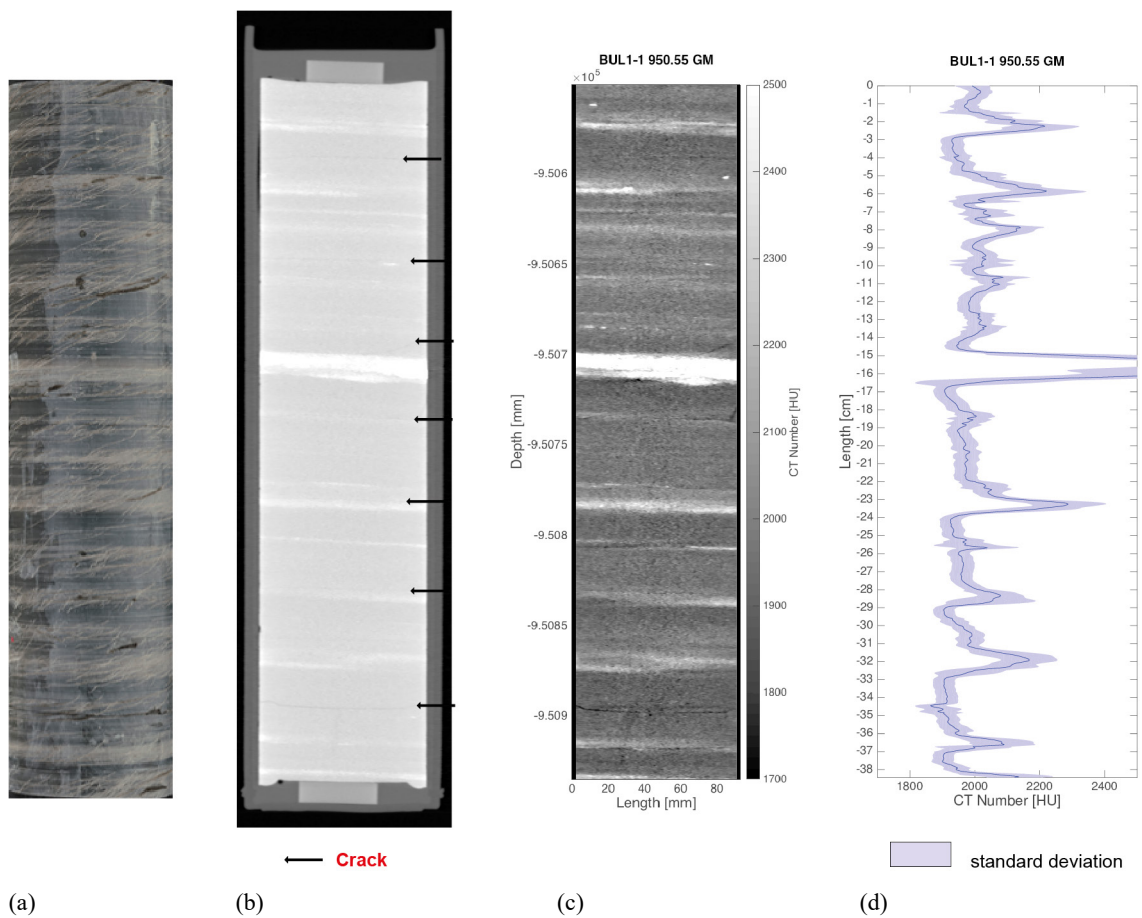


Fig. 3-2: Cross-section of XCT stack

(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). Example pictures are from borehole Bülach-1-1 (Nagra 2021).

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 bedding (Fig. 4-1).

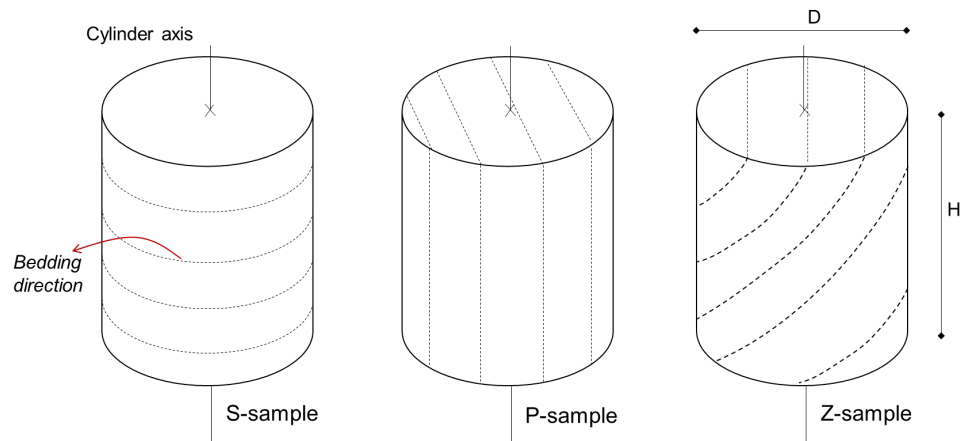


Fig. 4-1: Terminology of specimens in relation to bedding orientation

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 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 Illocut 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. 4-2). 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).

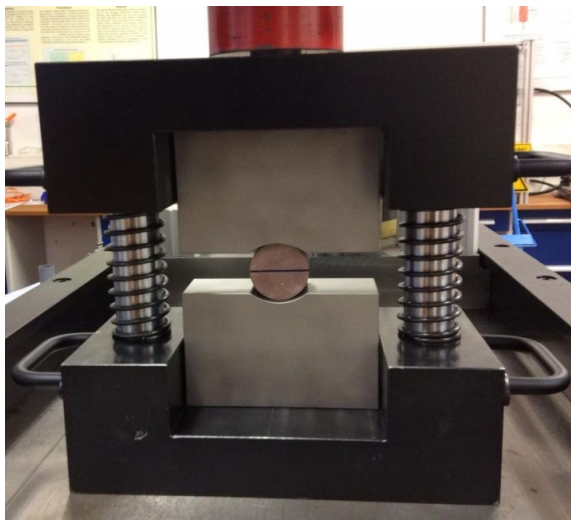


Fig. 4-2: Brazilian testing configuration with curved jaws
Plug diameter is 30 mm.

4.2.2 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 (ASTM 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 specimen 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. Measurements were either done directly at the specimen or corrected for load frame deformation. Radial deformation of the specimen is measured diametric with a transverse extensometer (Fig. 4-3).

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

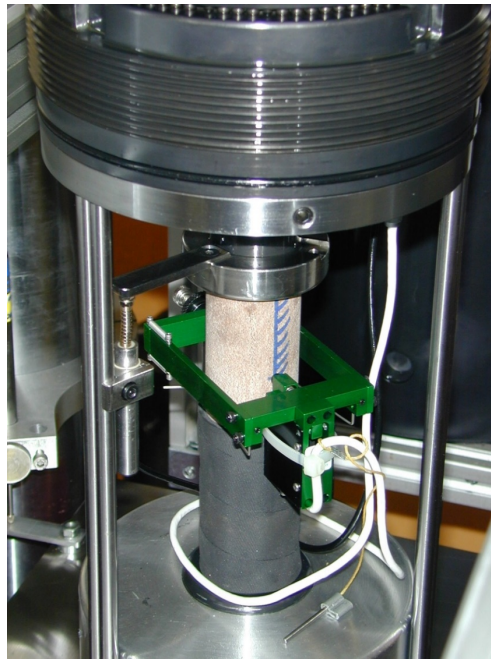


Fig. 4-3: UCS testing configuration

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

4.2.3 Triaxial deformation test

Triaxial deformation tests (TRX) 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 4.2.2). 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 specimen (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. 4-4). 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 specimen distortions that may 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. Exceptions were the anhydrite specimens in tests MAR1-1-857TRX_S1 and MAR-962TRX_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.

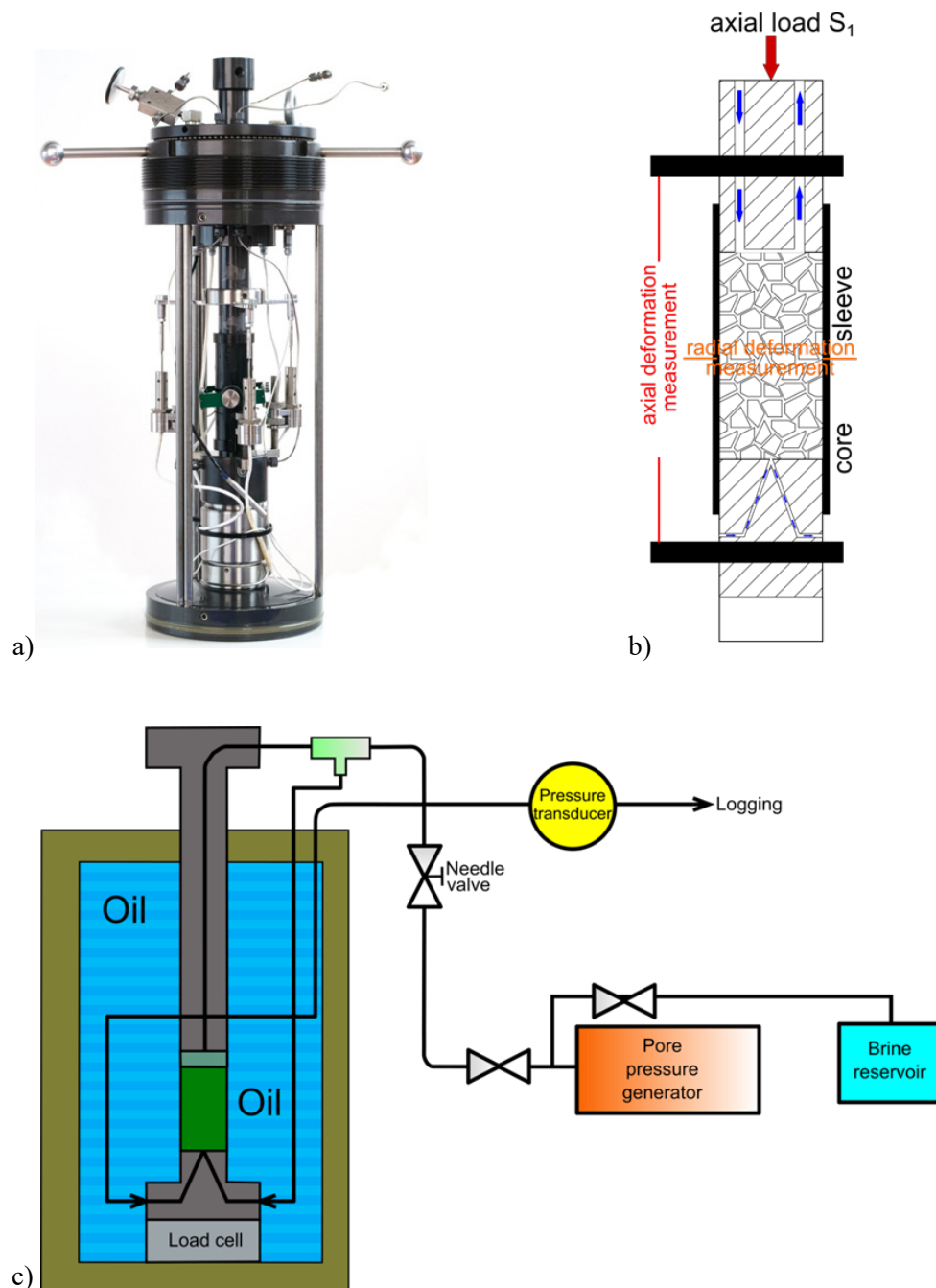


Fig. 4-4: Triaxial testing configuration used for rock-mechanical testing

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. 4-5 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 shear path 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. 4-5. 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 illustrations in Fig. 4-5 are the stress paths for total stress only, so the tests can be conducted in either drained or undrained conditions. This means that the submitted loading conditions may differ based on the porewater pressure generation during undrained testing. For the triaxial tests of the rock-mechanical testing programme, CTC and TC stress paths were executed.

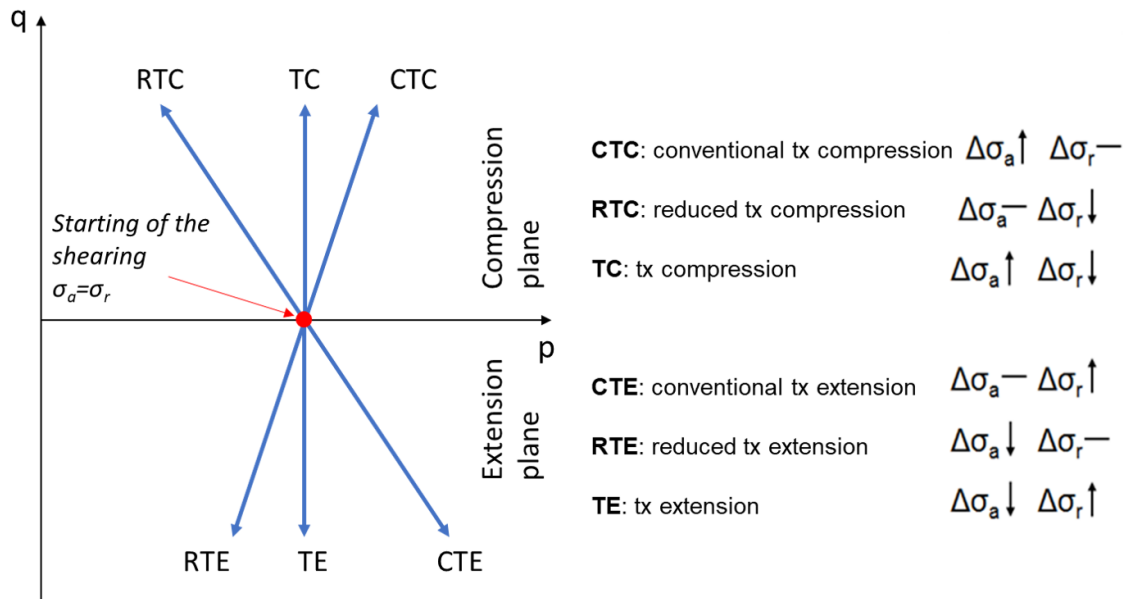


Fig. 4-5: Summary of possible stress paths that can be used to perform triaxial tests

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 specimen 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 specimen and the deformations in time are recorded and analysed. The deformation evolves according to the dissipation of the porewater 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 specimens 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} (2 TRX tests), 10^{-6} s^{-1} (4 TRX tests) and 10^{-7} s^{-1} (4 TRX tests). Strain rate was chosen based on lithological criteria (*cf.* Nagra 2021). Generally, clay-rich rocks, such as clay- and siltstones or marls were tested at lower strain rates than limestones, sandstones and anhydrites.

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 Schlattigen 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 MAR1-1 core material (*cf.* Dossier VIII). These investigations confirm the general Na-Cl water type. The salinities in the Dogger section are somewhat higher than in the APW; e.g. chloride concentrations are in the range of 6 – 9 g/L (APW: 6.7 g/L). Towards the base of the Staffelegg Formation, the chloride contents decrease to about 2 g/L. In addition to saturation with carbonate minerals, the investigations suggest saturation with respect to the sulphate mineral celestite. 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 slightly higher ionic strength in the porewater found in the MAR1-1 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 the 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 was 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. 4-6a):

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 specimen disturbance (i.e. fissure opening during resaturation); 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 specimens 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 specimen 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 is adopted by Lab A and Lab B. A different procedure was used by Lab C and is referred to as «alternative procedure» (Fig. 4-6b). 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 diameter (D) to height (H) ratio $D:H \approx 0.5$. In particular, the specimen nominal diameters are:

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

All the labs used two 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). Water content after testing was determined by oven-drying at 105 °C for several days until constant weight.

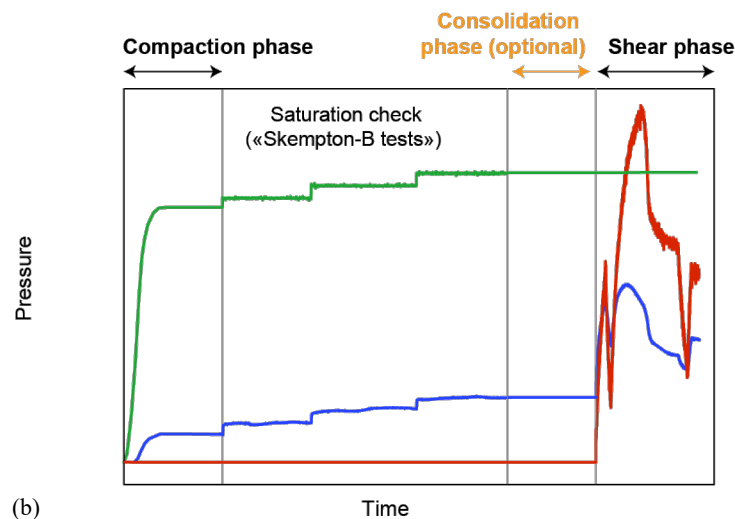
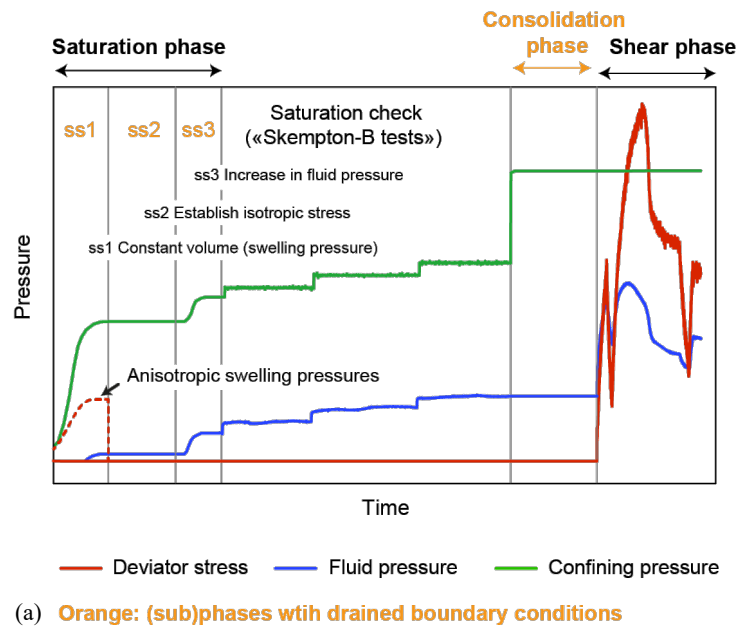


Fig. 4-6: Pressure – time sequences of triaxial deformation tests (specialised protocols)
 (a) conventional and (b) alternative testing procedures for undrained boundary conditions during the shear phase.

4.3.3 Oedometric test (high-pressure, specialised protocol)

Specimens for oedometric testing are of cylindrical shape, with a 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 handled 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 specimens 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 specimen and the laboratory atmosphere. After the final specimen diameter was reached, the specimen 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 resaturated 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 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 an axial stress of 10 – 15 MPa. For selected tests, a constant-head permeability test (PERM) is performed at this stage, increasing the pore pressure at the bottom side and measuring the fluid flow through the specimen. 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 an 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 (oven-drying at 105° C for several days until constant weight).

4.3.4 Complementary tests

In this section, complementary tests performed within the context of the geomechanical testing campaign are described: (i) one-dimensional swelling (OS), either allowing axial swelling in free stress conditions or at constant stress, and (ii) 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 as concluded. After completion of the test, the oedometer cell is dismantled, and the specimen is dried in the oven to measure the water content.

Specimens for one-dimensional swelling under constant load are placed in the oedometric cell. First, the prescribed load is applied, then the sample is put in contact with the synthetic pore fluid at low pressure (< 50 kPa) to saturate the specimen as in a one-dimensional swelling test (described above). In some cases, the test is continued as a high-pressure oedometric test.

Constant-head permeability tests were performed on 5 of the specimens tested at predefined stages during one-dimensional compression tests, at constant vertical total stress.

A pore pressure gradient between the 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 Marthalen-1-1: Lithostratigraphy).

- **Villigen Formation:** Mostly well bedded, micritic limestone with fossil fragments.
- **Wildeggen Formation:** Calcareous marl with limestone interbeds of the Effingen Member. At MAR1-1, the marly sections are argillaceous rather than calcareous.
- **Variansmergel Formation:** Argillaceous marl to claystone, some siderite nodules and abundant fossils and bioturbation (burrows).
- **«Parkinsoni-Württembergica-Schichten»:** A uniform succession of silty claystone to argillaceous marl with interbeds of calcareous marl and limestone.
- **«Humphriesoolith Formation»:** Marly interval with limestone beds which is sandwiched between two iron-oolitic beds at the top and at the base, all being very fossiliferous. The cores used for testing are from the marly interval without iron-oolids.
- **Wedelsandstein Formation:** silty to sandy marl with interbeds of bioclasts and sandy limestone. Tested intervals are from predominantly argillaceous sandstones and siltstones.

- **Opalinus Clay:** Silty claystone with silty to sandy lenses. The following subdivision was specified:
 - **590.35 – 626.41 m Sub-Unit 1: «Subunit with silty calcareous beds» (36.06 m):** Claystone to silty claystone with discrete heterogeneities, in particular silty calcareous beds and nodules.
 - **626.41 – 642.77 m Sub-Unit 2: «Upper silty subunit» (16.36 m):** Claystone to silty claystone with frequent lenses, typically centimetres wide and several millimetres thick (flaser structure).
 - **642.77 – 688.88 m Sub-Unit 3: «Mixed clay-silt-carbonate subunit» (46.11 m):** Silty claystone, devoid of marked characteristics typical for the other subunits.
 - **688.88 – 705.40 m Sub-Unit 4: «Clay-rich subunit» (16.52 m):** Homogeneous claystone with high clay-mineral content (mostly > 60 wt.-%).
- **Staffellegg Formation:** Heterogeneous succession, dominated by marl and claystone, some bituminous, and including various thin limestone horizons, some of them iron-oolitic. The tested intervals belong to the bituminous shale of the Rietheim Member (720 m) and the silty claystone of the Rietheim Member (732 m).
- **Klettgau Formation:** Heterogeneous succession of silty shale and dolocretes. The tested intervals are from the Seebi Member and consist of dolomite (772 m) and argillaceous marl (775 m).
- **Bänkerjoch Formation:** Evaporitic succession of sulphate sediments, interbedded by dolomitic marls, dolostones and sandstones. The tested intervals are claystones (793 m), thin-layered anhydrite and claystone sequences (838 – 845 m) anhydrites, and banded massive anhydrite / dolomites (857 m).
- **Schinznach Formation:** Succession of dolomites and dolostones, partly dolomitised limestones and micritic limestones, and bituminous claystones. The tested intervals are predominantly dolostones and limestones.
- **Zeglingen Formation:** Evaporation sequence but without salt layer (at MAR1-1). Tested interval is from massive to banded anhydrites with interbeds of claystone.
- **Kaiseraugst Formation:** Calcareous marl with abundant fossils.

4.4.2 Test overview

A total of 128⁷ rock-mechanical and geomechanical tests were conducted on a total of 30 cores from borehole MAR1-1 (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. This 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.

⁷ The constant-head permeability tests (PERM) are performed as part of the oedometric tests (OED+) and are therefore not counted separately in the total number of tests, but they are listed as such in Tab. 4-2 for overview.

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), TRX+ = TRX with specialised protocols, OED+ = Oedometric test with specialised protocols (high-pressure), OS = one-dimensional swelling test, PERM = constant-head permeability tests in OED+ tests).

Core-ID	Formation	Top	Length	Bottom		Rock-mechanical testing programme			Geomechanical testing programme			
						Standard rock-mechanical tests			Special protocols		Other	
MAR1-1-		[m]	[cm]	[m]	PVC	TRX	UCS	TS	TRX+	OED+	OS	PERM
472.81/42	Villigen Fm.	472.81	42	473.23	1	1	2	2				
483.94/50	Wildegge Fm.	483.94	50	484.44	1		2	2				
487.09/41		487.09	41	487.50	1	1	2	2				
506.78/40	Variansmergel Fm.	506.78	40	507.18	1					1	1	1
521.41/50	«Parkinsoni-Württ.-Sch.»	521.41	50	521.91	1	1	2	2				
535.04/40		535.04	40	535.44	1					1		1
551.88/50	«Humphriesoolith»	551.88	50	552.38	1					1		1
553.06/45		553.06	45	553.51	1	1	3	2				
572.98/50	Wedelsandstein Fm.	572.98	50	573.48	1	1	2	2				
580.15/50		580.15	50	580.65	1		2	2				
608.11/50	Opalinus Clay	608.11	50	608.61	1				4			
617.22/50		617.22	50	617.72	1				6			
629.67/40		629.67	40	630.07	1					2	2	
647.71/36		647.71	36	648.07	1				8			
663.37/50		663.37	50	663.87	1				2			
676.96/49		676.96	49	677.45	1					2	2	1
701.62/51		701.62	51	702.13	1					4	3	1
720.47/43	Staffellegg Fm.	720.47	43	720.90	1	1	2	2				
732.15/50		732.15	50	732.65	1		2	2				
772.15/45	Klettgau Fm..	772.15	45	772.60	1		2	2				
775.44/50		775.44	50	775.94	1	1	2	2				
793.50/50	Bänkerjoch Fm.	793.50	50	794.00	1		1	2				
838.87/47		838.87	47	839.34	1		3	2				
845.30/37		845.30	37	845.67	1		2	2				
857.62/50		857.62	50	858.12	1	1	2	2				
896.89/50	Schinznach Fm.	896.89	50	897.39	1		1	2				
902.70/32		902.70	32	903.02	1		1	1				
917.48/36		917.48	36	917.84	1	1	2	2				
962.57/48	Zeglingen Fm.	962.57	48	963.05	1	1	2	2				
1010.41/50	Kaiseraugst Fm.	1'010.41	50	1'010.91	1		3	2				
Total					30	10	40	39	20	11	8	5

5 Rock-mechanical testing

5.1 Indirect tensile strength test (Brazilian)

A total of 39 Brazilian tests were performed from two laboratories (Tab. 5-1). The splitting tensile strength (MPa) is calculated according to the following formula (curved jaws):

$$S_z = \frac{1272 * F}{\pi * d * l} \quad (5-1)$$

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. 5-1: Test results from indirect tensile strength tests

For specimen geometry see Fig. 4-1.

Test-ID	Specimen geometry	Average depth	Bulk density	Water content	Tensile strength
		[m]	[g/cm ³]	[wt.-%]	[MPa]
MAR1-1_472TS_S1	S	472.82	2.677	0.60	-7.42
MAR1-1_472TS_P1	P	472.86	2.673	0.73	-6.12
MAR1-1_483TS_S1	S	484.24	2.609	2.54	-2.93
MAR1-1_483TS_P1	P	484.28	2.614	2.18	-2.34
MAR1-1_487TS_S1	S	487.26	2.631	1.79	-2.73
MAR1-1_487TS_P1	P	487.29	2.615	2.54	-1.66
MAR1-1_521TS_P1	P	521.80	2.479	5.08	-1.59
MAR1-1_521TS_S1	S	521.83	2.499	5.36	-2.54
MAR1-1_553TS_S1	S	553.21	2.520	4.68	-2.35
MAR1-1_553TS_P1	P	553.26	2.526	4.56	-1.90
MAR1-1_573TS_P1	P	573.31	2.449	3.96	-1.88
MAR1-1_573TS_S1	S	573.35	2.463	4.50	-2.34
MAR1-1_580TS_S1	S	580.33	2.539	3.25	-3.20
MAR1-1_580TS_P1	P	580.36	2.568	2.82	-2.20
MAR1-1_720TS_S1	S	720.56	2.246	4.22	-3.87
MAR1-1_720TS_P1	P	720.60	2.255	4.16	-1.56
MAR1-1_732TS_P1	P	732.46	2.542	4.13	-1.91
MAR1-1_732TS_S1	S	732,50	2,521	4,49	-3,18
MAR1-1_772TS_P1	P	772.40	2.531	3.17	-2.39
MAR1-1_772TS_S1	S	772.45	2.515	3.05	-3.78
MAR1-1_775TS_P1	P	775.74	2.552	5.47	-1.54
MAR1-1_775TS_S1	S	775.78	2.554	5.39	-2.62
MAR1-1_793TS_S1	S	793.77	2.571	4.41	-3.79
MAR1-1_793TS_P1	P	793.81	2.554	4.73	-2.08
MAR1-1_838TS_P1	P	839.12	2.781	1.36	-3.47

Tab. 5-1: (continued)

Test-ID	Specimen geometry	Average depth	Bulk density	Water content	Tensile strength
		[m]	[g/cm ³]	[wt.-%]	[MPa]
MAR1-1_838TS_S1	S	839.16	2.838	0.80	-4.59
MAR1-1_845TS_S1	S	845.48	2.511	5.66	-2.02
MAR1-1_845TS_P1	P	845.52	2.560	5.42	-2.11
MAR1-1_857TS_P1	P	857.91	2.921	0.06	-5.42
MAR1-1_857TS_S1	S	857.95	2.860	0.22	-4.95
MAR1-1_896TS_S1	S	897.04	2.741	1.35	-5.96
MAR1-1_896TS_P1	P	897.08	2.750	1.38	-3.14
MAR1-1_902TS_P1	P	902.86	2.621	4.61	-2.55
MAR1-1_917TS_P1	P	917.68	2.657	2.19	-4.96
MAR1-1_917TS_S1	S	917.72	2.649	1.99	-6.32
MAR1-1_962TS_S1	S	962.74	2.906	0.26	-6.00
MAR1-1_962TS_P2	P	962.77	2.940	0.14	-2.91
MAR1-1_1010TS_P1	P	1'010.68	2.636	1.79	-2.09
MAR1-1_1010TS_S1	S	1'010.72	2.590	2.64	-3.98

The test results yielded values from -7.42 MPa to -2.02 MPa for S-samples and -6.12 MPa to -1.54 MPa for P-samples. Most TS specimens failed along primary central tensile fractures. Photographs of the specimens before and after testing can be found in Appendix A.

5.2 Unconfined compressive strength tests (UCS)

A total of 40 UCS tests were performed (Tab. 5-2). Photographs of the specimens before and after testing can be found in Appendix A.

At the linear portion of the axial load cycle, an unloading/reloading 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 $\Delta\varepsilon_a$ (dimensionless) according to the equation:

$$E = \frac{\Delta\sigma_a}{\Delta\varepsilon_a} \quad (5-2)$$

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

$$\nu = -\frac{\Delta\varepsilon_r}{\Delta\varepsilon_a} \quad (5-3)$$

An example of calculation of tangent E modulus is reported in Fig. 5-1.

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

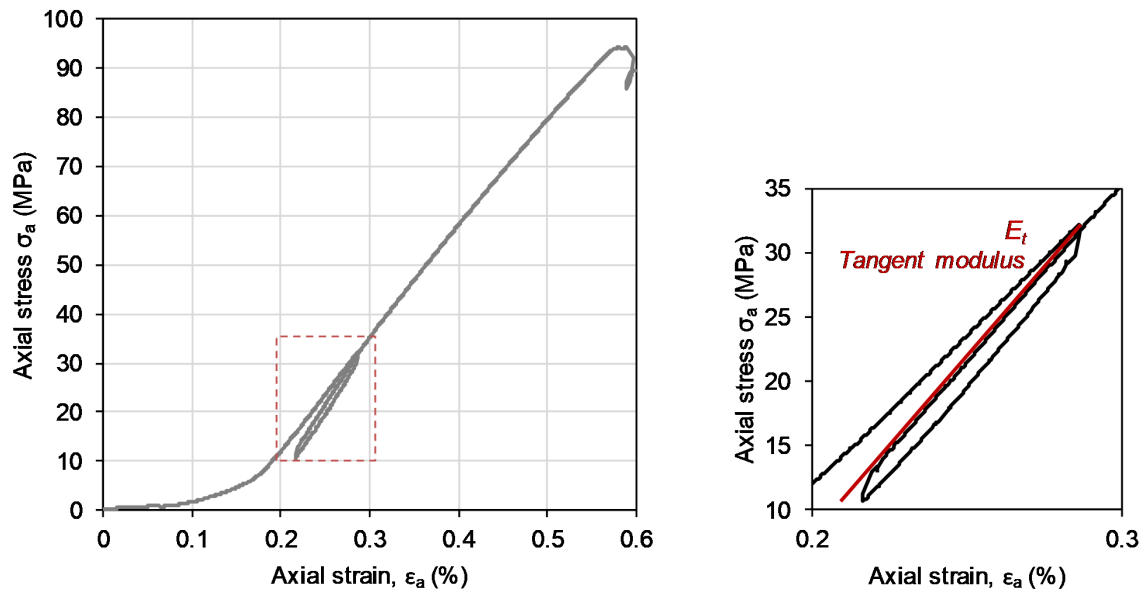


Fig. 5-1: Stress – strain diagram for UCS test

On the right zoom into the unloading/reloading loop, and definition of tangent and secant Young's moduli. Example from test BUL1-1_661UCS_S1 (Nagra 2021).

Tab. 5-2: Test results from UCS test

^a Samples failed unexpectedly before hysteresis No unloading and reloading was performed.
Young's modulus refers to E_{50} . For specimen geometry see Fig. 4-1.

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]	[%]
MAR1-1_472UCS_S1	S	472.96	2.71	0.51	21.23	0.227	63.87	0.55
MAR1-1_472UCS_P1	P	473.00	2.67	0.45	41.69	0.201	263.13	0.73
MAR1-1_483UCS_P1	P	484.15	2.62	2.64	21.29	0.228	64.03	0.55
MAR1-1_483UCS_S1	S	484.16	2.64	1.95	17.92	0.163	64.72	0.59
MAR1-1_487UCS_P1	P	487.23	2.62	1.78	23.90	0.204	72.91	0.38
MAR1-1_487UCS_S1	S	487.44	2.63	1.83	15.89	0.196	46.75	0.42
MAR1-1_521UCS_P1	P	521.68	2.51	4.77	14.50	0.607	44.17	0.50
MAR1-1_521UCS_S1	S	521.74	2.54	4.84	8.24	0.247	32.67	0.91
MAR1-1_553UCS_S1	S	553.13	2.54	4.32	10.16	0.181	39.57	0.81
MAR1-1_553UCS_P1	P	553.17	2.52	4.81	12.42	0.282	39.45	0.66
MAR1-1_553UCS_S2	S	553.43	2.64	2.14	20.24	0.149	53.67	0.45
MAR1-1_573UCS_P1	P	573.39	2.48	4.76	9.76	0.290	32.45	0.61
MAR1-1_573UCS_S1	S	573.44	2.46	4.69	7.59	0.255	33.04	0.80
MAR1-1_580UCS_P1	P	580.27	2.50	4.39	12.57	0.273	32.09	0.44
MAR1-1_580UCS_S1	S	580.28	2.51	4.19	9.38	0.264	29.53	0.56
MAR1-1_720UCS_S1	S	720.51	2.20	4.43	7.11	0.198	50.73	1.39
MAR1-1_720UCS_P1	P	720.51	2.28	4.90	12.97	0.377	48.93	0.55
MAR1-1_732UCS_S1	S	732.28	2.56	4.10	8.66	0.159	37.92	0.87
MAR1-1_732UCS_P1	P	732.55	2.55	4.48	15.88	0.329	48.06	0.57
MAR1-1_772UCS_S1	S	772.50	2.58	3.02	23.20	0.209	78.67	0.55
MAR1-1_772UCS_P1	P	772.50	2.58	3.39	21.18	0.179	68.96	0.49
MAR1-1_775UCS_P1	P	775.82	2.56	5.37	15.96	0.291	50.39	0.42
MAR1-1_775UCS_S1	S	775.88	2.54	5.82	9.55	0.223	38.56	0.72
MAR1-1_793UCS_P1	P	793.89	2.56	5.05	12.60	0.199	38.83	0.56
MAR1-1_838UCS_S1	S	839.02	2.64	3.37	9.10	0.146	24.11	0.57
MAR1-1_838UCS_S2	S	839.22	2.79	1.30	18.42	0.145	45.41	0.43
MAR1-1_838UCS_P2	P	839.22	2.73	2.03	20.58	0.178	27.43	0.27
MAR1-1_845UCS_S1 ^a	S	845.39	2.62	4.12	5.36	0.440	15.27	0.70
MAR1-1_845UCS_P1	P	845.42	2.59	4.97	9.16	0.284	9.40	0.18
MAR1-1_857UCS_S1 ^a	S	857.81	2.95	0.03	26.76	0.130	31.79	0.15
MAR1-1_857UCS_P1	P	857.81	2.95	0.04	49.86	0.195	88.76	0.28
MAR1-1_896UCS_S1	S	896.96	2.75	0.61	36.53	0.154	210.00	0.74
MAR1-1_902UCS_P1	P	902.78	2.65	2.63	23.47	0.349	49.61	0.40
MAR1-1_917UCS_S1	S	917.60	2.67	2.52	34.65	0.183	171.93	0.60
MAR1-1_917UCS_P1	P	917.64	2.68	2.52	25.64	0.104	146.21	0.60
MAR1-1_962UCS_S1	S	962.62	2.90	0.42	29.73	0.143	94.65	0.47
MAR1-1_962UCS_P1	P	962.67	2.92	0.24	17.01	0.099	31.54	0.31
MAR1-1_1010UCS_S1	S	1'010.45	2.66	1.60	15.79	0.130	66.07	0.59
MAR1-1_1010UCS_P1	P	1'010.49	2.63	2.90	23.08	0.373	51.21	0.30
MAR1-1_1010UCS_S2	S	1'010.77	2.69	1.23	26.11	0.160	71.37	0.48

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. Most UCS plugs failed by longitudinal splitting or due to spalling.

Due to the high lithological diversity, the rock-mechanical data scatter widely. A number of tests especially in limestones of the Malm were controlled by planes of relative weakness, e.g. healed fractures (MAR1-1_472UCS_S1, MAR1-1_838UCS_P1 and MAR1-1_962UCS_P1) or stylolites (MAR1-1_487UCS_S1 and MAR1-1_487UCS_P1).

5.3 Triaxial deformation tests

A total of 10 triaxial tests were performed. Photographs of the specimens before and after testing can be found in Appendix A.

Basic properties and test conditions are provided in Tab. 5-3. The water content reported refers to the final, post-test measurement. With some exceptions, tests were conducted undrained with an initial fluid pressure of 5 MPa. The anhydrite samples MAR1-1_857TRX_S1 and MAR1-1_962TRX_S1 were carried out "dry" without the pore-fluid system and at a strain rate of $1 \times 10^{-05} \text{ s}^{-1}$. Several tests were performed with slightly larger pore pressure (approximately 6 – 7 MPa) to allow monitoring of fluid pressure in undrained conditions where larger dilation occurs (see below). Confining pressure in all tests ranged from 10 to 17 MPa.

Ultrasonic velocities were recorded throughout the tests, and the tabulated values in Tab. 5-3 were constrained at the start of the shear phase. P-wave signals could not be determined for one sample (MAR1-1_775TRX_S1). S-wave signals could not be determined for two samples (MAR1-1_521TRX_S1 and MAR1-1_775TRX_S1).

Consolidation phases were performed in 4 tests (MAR1-1_521TRX_S1, MAR1-1_553TRX_S1, MAR1-1_720TRX_S1, MAR1-1_775TRX_S1) in the attempt to constrain consolidation coefficient values. But deformation during the consolidation tests were too low for robust determination of the consolidation index c_v (Fig. 5-2c). In these same tests, Skempton's B values were also constrained⁸. But in all four cases, B values did not fully equilibrate over successive steps.

In some tests, the pore pressure dropped to a value close to zero during the shearing phase (Figs. 5-2a, 5-2b and Tab. 5-4).

⁸ The tabulated values in Tab. 5-3 are not corrected for potential equipment compliance.

Tab. 5-3: Basic properties and triaxial test results from consolidation and saturation stage
For specimen geometry see Fig. 4-1. ^a B values increasing over successive steps.

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
MAR1-1		[m]	[g/cm ³]	[wt.-%]	[MPa]	[MPa]	[mm ² /h]	[-]	[m/s]	[m/s]
472TRX_S1	S	472.96	2.678	1.89	5.0	10.0			6'050	3'117
487TRX_S1	S	487.44	2.625	3.37	5.0	10.0			4'459	2'343
521TRX_S1	S	521.73	2.524	5.91	6.2	16.5	-	0.50 ^a	3'255	
553TRX_S1	S	553.13	2.531	4.99	6.5	16.5	-	0.45 ^a	3'621	2'048
573TRX_S1	S	573.44	2.465	7.88	5.0	10.0			3'501	1'857
720TRX_S1	S	720.66	2.456	5.71	6.3	16.3	-	0.52 ^a	3'042	1'855
775TRX_S1	S	775.88	2.534	6.67	7.0	17.0	-	0.68 ^a		
857TRX_S1	S	857.87	2.936	0.18		10.0			6'325	3'319
917TRX_S1	S	917.60	2.666	3.68	5.0	10.0			5'586	2'919
962TRX_S1	S	962.62	2.894	0.63		10.0			5'572	2'980

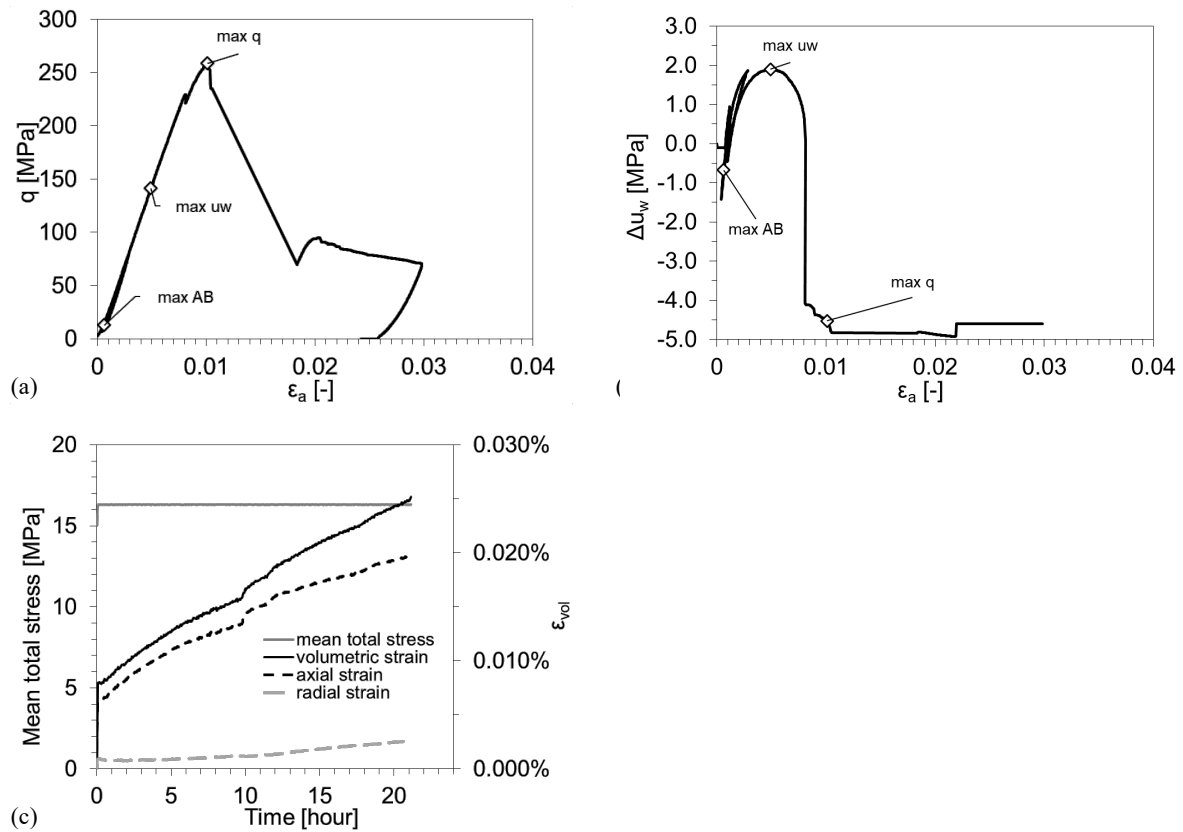


Fig. 5-2: Examples of problematic pore pressure and strain evolution during testing

(a), (b) Example test 472TRX_S1, with deviatoric stress (q , (a)) and generated pore fluid pressure (Δu_w , (b)). Pore fluid pressure dropped rapidly after pore pressure maximum value.

(c) Example test 720TRX_S1, consolidation phase in which consolidation coefficient could not be constrained. Strain evolves rather linearly, no consolidation could be detected.

Tab. 5-4: Triaxial test results from the shear phase

Test-ID	Stress path	Strain Rate	Young's modulus	Poisson's ratio	Deviatoric stress at peak ($q = \sigma_1 - \sigma_3$)	Radial total stress at peak (σ_3)	Pore pressure at peak	Axial strain at peak	Post-peak q	Pore pressure at post-peak
MAR1-1		[1/s]	[GPa]	[-]	[MPa]	[MPa]	[MPa]	[%]	[MPa]	[MPa]
472TRX_S1	CTCU	1×10^{-06}	32.9	0.20	258.8	10.0	0.6	1.08	71.1	0.5
487TRX_S1	CTCU	1×10^{-06}	11.6	0.46	90.1	10.0	5.7	1.23	40.8	0.4
521TRX_S1	CTCU	2×10^{-07}	5.9	0.39	26.0	16.5	10.6	0.85	17.5	6.8
553TRX_S1	CTCU	1×10^{-07}	7.9	0.37	39.7	16.5	7.3	1.46	28.7	1.9
573TRX_S1	CTCU	1×10^{-06}	5.5	0.37	42.3	10.0	2.0	1.73	28.5	0.5
720TRX_S1	CTCU	1×10^{-07}	7.8	0.21	27.4	16.3	14.0	0.87	15.1	8.5
775TRX_S1	CTCU	1×10^{-07}	7.6	0.35	28.6	17.0	13.0	0.68	17.9	8.3
857TRX_S1	CTC	1×10^{-05}	37.8	0.14	185.3	10.0		0.75	63.0	
917TRX_S1	CTCU	1×10^{-06}	31.3	0.22	204.3	10.0	0.3	0.88	74.9	0.0
962TRX_S1	CTC	1×10^{-05}	34.1	0.21	130.1	10.0		0.75	36.5	

6 Geomechanical testing programme

In the MAR1-1 geomechanical testing campaign, 20 triaxial tests were performed. Three geometries (Section 5.1) were selected: S-, P- and Z-samples. Z-samples were prepared in order to obtain specimen shearing along the bedding direction, i.e. to investigate the direction with potentially the minimum strength. The angle to bedding was decided based on the results from the previous testing campaigns and the first results with P- and S-samples of MAR1-1.

Z-samples were prepared with an angle of $55^\circ - 60^\circ$ between the bedding direction and the horizontal plane (i.e., $30^\circ - 35^\circ$ between bedding direction and the cylinder axis).

In addition, 11 one-dimensional compression (including 5 constant-head permeability tests) and 8 one-dimensional swelling tests were also 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 also determined basic properties 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 specimen, either (i) by injecting water in isochoric conditions, measuring the generated swelling pressure (conventional testing procedure), or (ii) by isotropically compressing the specimen until a positive pore pressure is developed (alternative testing procedure) (*cf.* Fig. 4-6).

When the conventional testing procedure is used, the water content before and after the tests generally differs, and in the case of undrained boundary conditions, the difference may primarily be attributed to resaturation in the rig. This is the case for results from Lab B. Using the alternative testing procedure (Lab C), no fluid is added to the specimen whilst in the rig. Therefore, the water content before and after the test is expected to be the same. For drained boundary conditions, volumetric deformation during testing may also alter the water content of the specimen (data points marked with *d* for drained, in Fig. 6-1). Additional sources of discrepancy between the water content before and after the tests can be attributed to the handling of the specimen when dismantling the tests (e.g., loss of solid mass while removing the specimen from the sleeve, drained unloading phases which allow further swelling and water content increase).

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 content⁹.

⁹ 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.

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 a psychrometer.

Results are reported in Fig. 6-2. All cores exhibit high Native activity, above 67%, with peaks at approximately 88%, indicating excellent preservation conditions.

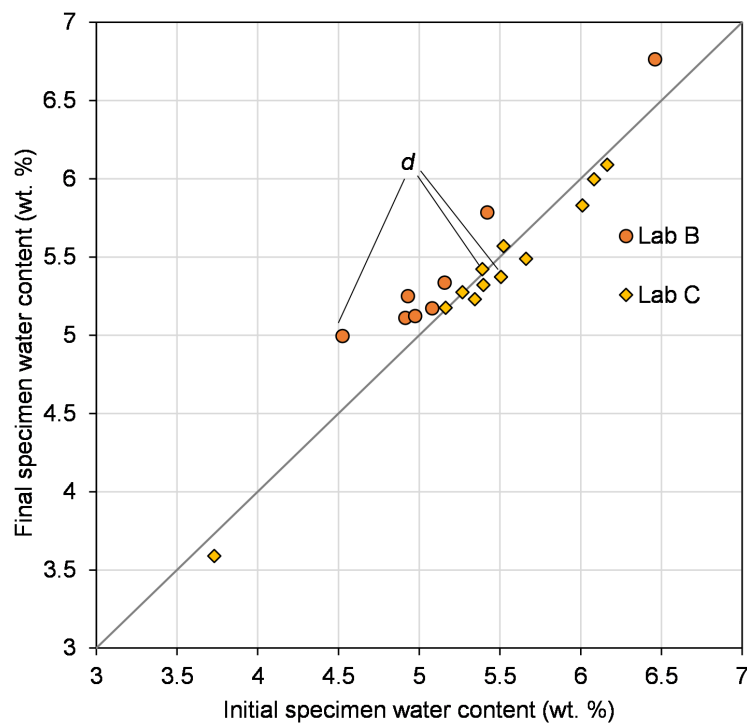


Fig. 6-1: Initial versus final water content of triaxial specimens
d indicates specimens from drained tests. Results grouped by laboratory.

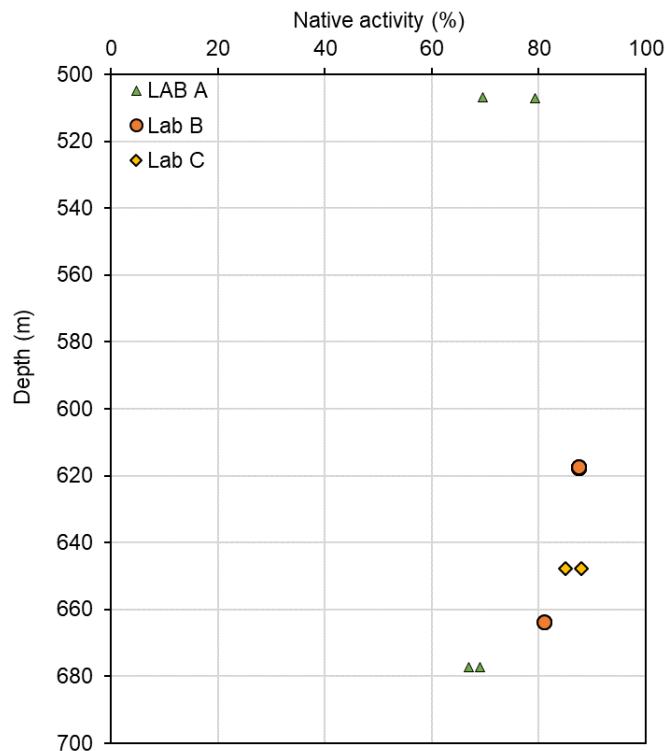


Fig. 6-2: Native activity of trimming material versus depth of collection
Results grouped by laboratory.

6.1.2 Atterberg limits, plasticity index, grain size distribution

In the context of the oedometric testing campaign Lab A and D performed geotechnical characterisation of the tested cores. They determined Atterberg limits and plasticity index, and Lab D additionally determined also the grain size distribution.

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, in line with Nagra's indications. The main aspects of the procedure of Lab D 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.

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, the results show a slightly lower clay fraction (29 – 33 wt.-%) with respect to the clay mineralogical content determined by XRD (41 – 52 wt.-%; Appendix D).

Granulometric fractions (sand, silt and clay) are distinguished (as by standard) on the dimension of the particles. It has to be pointed out that there could be particles in the silt granulometric fraction which are clay-mineral particles or aggregates with a larger size than the clay – silt threshold; on the other hand, there may be particles in the clay granulometric fraction which are minerals other than clays having fine particles dimension. All the previous considerations contribute to the difference between granulometric and mineralogical clay fractions.

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]	[-]	[-]	[-]	[%]	[%]	[%]	[%]
MAR1-1-535.04/40-GM	D	7 – 11	0.33	0.21	0.12	0	9.1	63.6	27.3
MAR1-1-551.88/50-GM	D	11 – 15	0.36	0.23	0.13	0.1	20.8	50.1	29.0
MAR1-1-629.67/40-GM	D	12 – 20	0.36	0.23	0.13	0.3	2.4	67.3	30.0
MAR1-1-701.62/51-GM	D	29 – 35	0.37	0.24	0.13	0	1.6	56.7	41.7
MAR1-1-676.96/49-GM	A	16 – 23	0.30	0.19	0.11	-	-	-	-
MAR1-1-506.78/40-GM	A	13 – 19	0.34	0.20	0.13	-	-	-	-
MAR1-1-506.78/40-GM	A	20 – 24	0.30	0.21	0.09	-	-	-	-

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 C8_MAR1_1_608_35P6CTCU indicates the laboratory that performed the test (C), a testing sequential number (8), the borehole name (MAR1-1), the specimen depth (608.35 m), the specimen geometry (P), effective confinement before shearing (6 MPa), the stress path (CTCU).

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

- Specimen ID, as described above
- Core depth [m]
- Geological formation, as presented in Section 4.4.1
- Specimen 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. 4-5

- Indication of shearing in drained conditions (d)
- Initial void ratio, computed using a reference solid density¹⁰ of 2.70 g/cm³
- Initial porosity, computed using a reference solid density of 2.70 g/cm³
- Initial and final water content.

The initial part of the ID (hereafter Specimen ID), including the laboratory and the sequential number is afterwards used as short code to quickly identify a specific test point in the entire set of results.

Tab. 6-2: Conducted triaxial tests

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

Specimen ID	Specimen location and geometry			Shearing phase characteristics					Specimen initial conditions			
	Core depth [m]	Formation	Specimen geometry	Total confining stress, p_0 [Mpa]	Pore pressure, u_{a0} [MPa]	Effective confining stress p'_0 [MPa]	Stress path	Drained shearing (d)	Initial void ratio, e_0^* [-]	Initial porosity, n_0 [-]	Initial water content, w_0 [%]	Final water content, w_f [%]
C8_MAR1_1_608_35P6CTCU	608.345	Opalinus Clay	P	15	9	6	CTCU		0.158	0.137	6.08	6.00
C6_MAR1_1_608_34P13CTCU	608.345		P	22	9	13	CTCU		0.160	0.138	6.16	6.09
C5_MAR1_1_608_34S13CTCU	608.345		S	22	9	13	CTCU		0.156	0.135	5.52	5.57
C7_MAR1_1_608_34S7CTCU	608.345		S	16	9	7	CTCU		0.157	0.136	6.01	5.83
B5_MAR1_1_617_28Z13CTCU	617.28		Z	22	9	13	CTCU		0.132	0.117	4.98	5.12
B1_MAR1_1_617_42S7CTCU	617.42		S	16	9	7	CTCU		0.144	0.126	6.46	6.76
B2_MAR1_1_617_42S5CTC	617.62		S	14	9	5	CTC	d	0.119	0.106	4.53	4.99
B6_MAR1_1_617_55Z5CTCU	617.55		Z	14	9	5	CTCU		0.128	0.113	4.91	5.11
B7_MAR1_1_617_55Z9CTCU	617.55		Z	18	9	9	CTCU		0.131	0.116	5.08	5.17
B8_MAR1_1_617_68P5CTCU	617.675		P	14	9	5	CTCU		0.143	0.125	5.42	5.78
C2_MAR1_1_647_84P13CTCU	647.84		P	22	9	13	CTCU		0.131	0.116	5.40	5.32
C12_MAR1_1_647_84P5CTC	647.84		P	15	10	5	CTC	d	0.141	0.123	5.39	5.42
C4_MAR1_1_647_84P6CTCU	647.84		P	15	9	6	CTCU		0.115	0.103	3.73	3.59
C3_MAR1_1_647_84S6CTCU	647.84		S	15	9	6	CTCU		0.129	0.114	5.27	5.27
C11_MAR1_1_647_84S5CTC	647.84		S	15	10	5	CTC	d	0.147	0.128	5.51	5.37
C1_MAR1_1_647_84S13CTCU	647.84		S	22	9	13	CTCU		0.130	0.115	5.16	5.18
C9_MAR1_1_647_84Z6CTCU	647.84		Z	15	9	6	CTCU		0.144	0.126	5.66	5.49
C10_MAR1_1_647_84Z13CTCU	647.84		Z	22	9	13	CTCU		0.136	0.120	5.34	5.23
B3_MAR1_1_663_46P5CTCU	663.46		P	14	9	5	CTCU		0.142	0.125	4.93	5.25
B4_MAR1_1_664_15P9CTCU	664.15		P	18	9	9	CTCU		0.133	0.118	5.16	5.34

¹⁰ Solid density measurements were conducted on specimens used for mineralogical analysis, and are available in Appendix D.

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 specimen saturation was conducted imposing isochoric conditions 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 specimen initial water content (before saturation). For specimens with S geometry, the radial direction corresponds to the one of the bedding, while the axial direction is perpendicular to it. Swelling pressure was found to be higher in the axial direction. No clear trend between swelling pressure and water content is found for this set of tests. It has to be noted that the specimens' water content varied within a limited range (about 2 – 3 wt.-%), which may be not sufficient to infer a trend.

On the other hand, in P-samples, 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.

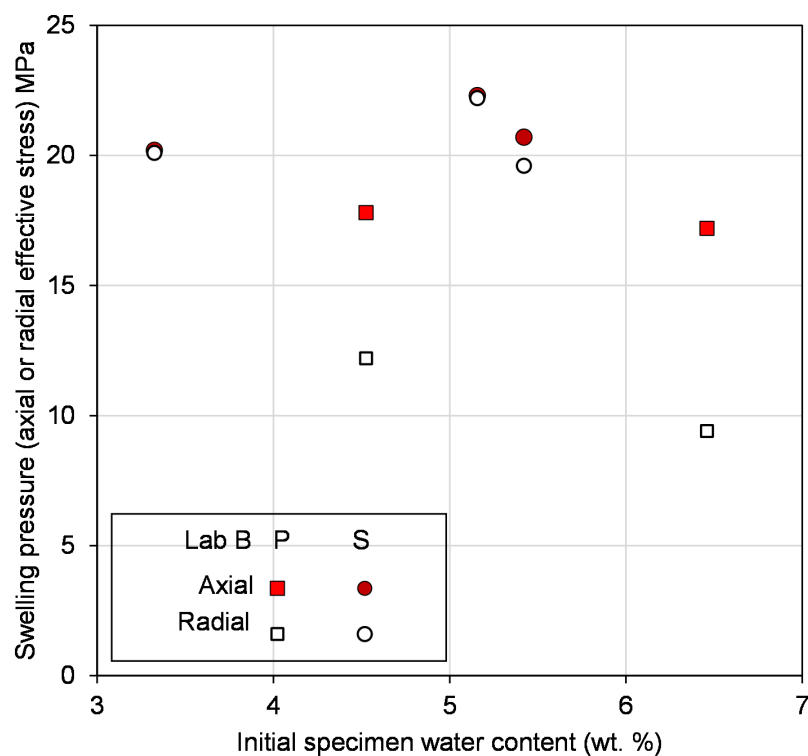


Fig. 6-3: Radial and axial swelling pressure versus initial specimen water content S-samples and P-samples.

6.2.1.2 B check tests

In each test, the specimen 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 measurements 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 porewater lines, V is the volume of the specimen, K is the bulk modulus of the specimen, K_f the bulk modulus of the pore fluid and K_s 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 Lab B, while the 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 the apparatus from Lab B is ~ 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 0.91 was calculated and marked as a dotted line in the graph.

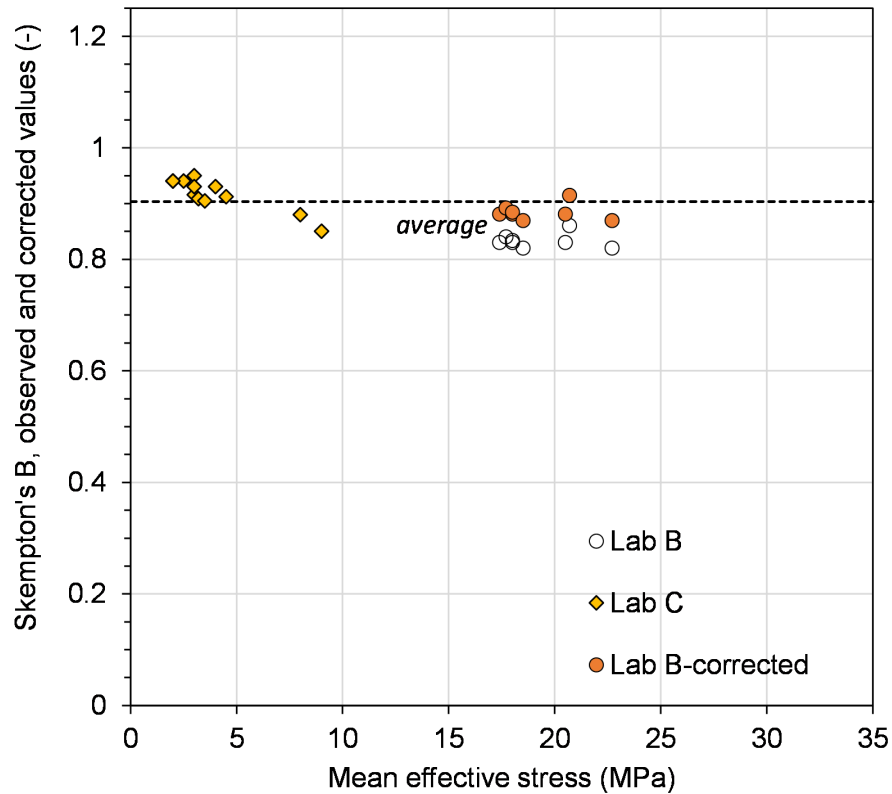


Fig. 6-4: Average Skempton's B values

B values versus the corresponding effective mean stress; data from MAR1-1 (Opalinus Clay): 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 specimens is reported in Fig. 6-5 versus the mean effective stress at the end of the consolidation phase. Lab C results yielded smaller values of the consolidation coefficient, which may be related to a higher clay content of the samples in the tested sections (mineralogical compositions in Appendix D).

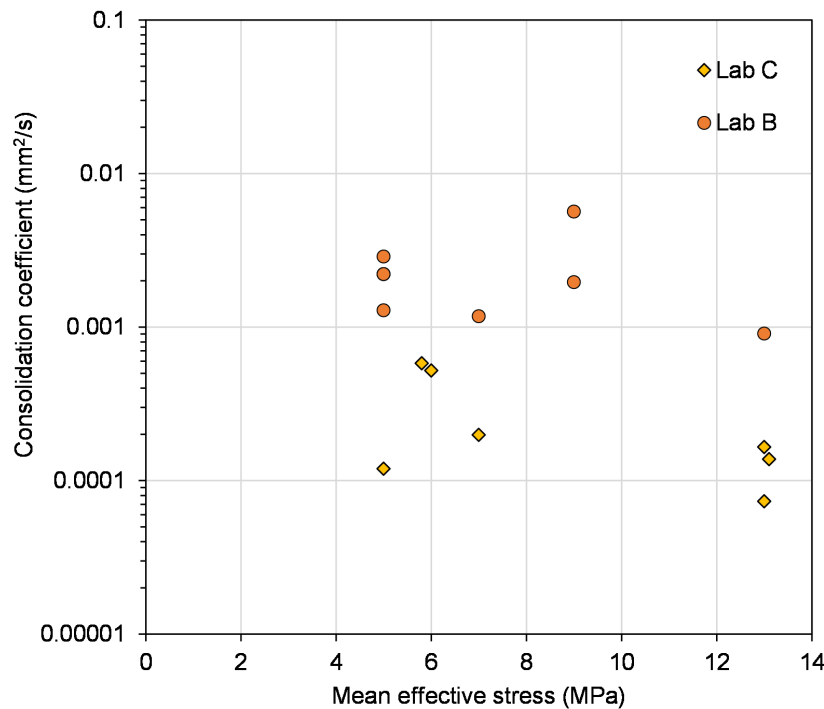


Fig. 6-5: Consolidation coefficient for Opalinus Clay specimens (all geometries)

6.2.2 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. 4-2) and p' is the mean effective stress, obtained by subtracting the porewater pressure u_w from the total stress (Eq. 4-1).

6.2.2.1 Stress paths of Opalinus Clay subunits

This section focuses on the stress paths obtained on Opalinus Clay specimens during the shearing phase of the tests. The behaviour of the tested specimens 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 porewater pressure (u_w) with the axial effective strain is observed among the tests (see Appendix B). The effective stress paths obtained on Opalinus Clay specimens are here grouped by the lithological sub-units (Fig. 6-6), as presented in Section 4.4.1. Results are further subdivided by the sample geometry. In tests C12_MAR1_1_647_84P5CTC and C11_MAR1_1_647_84S5CT, the drainage conditions were switched from drained to undrained at the end of the shearing, before the final unloading.

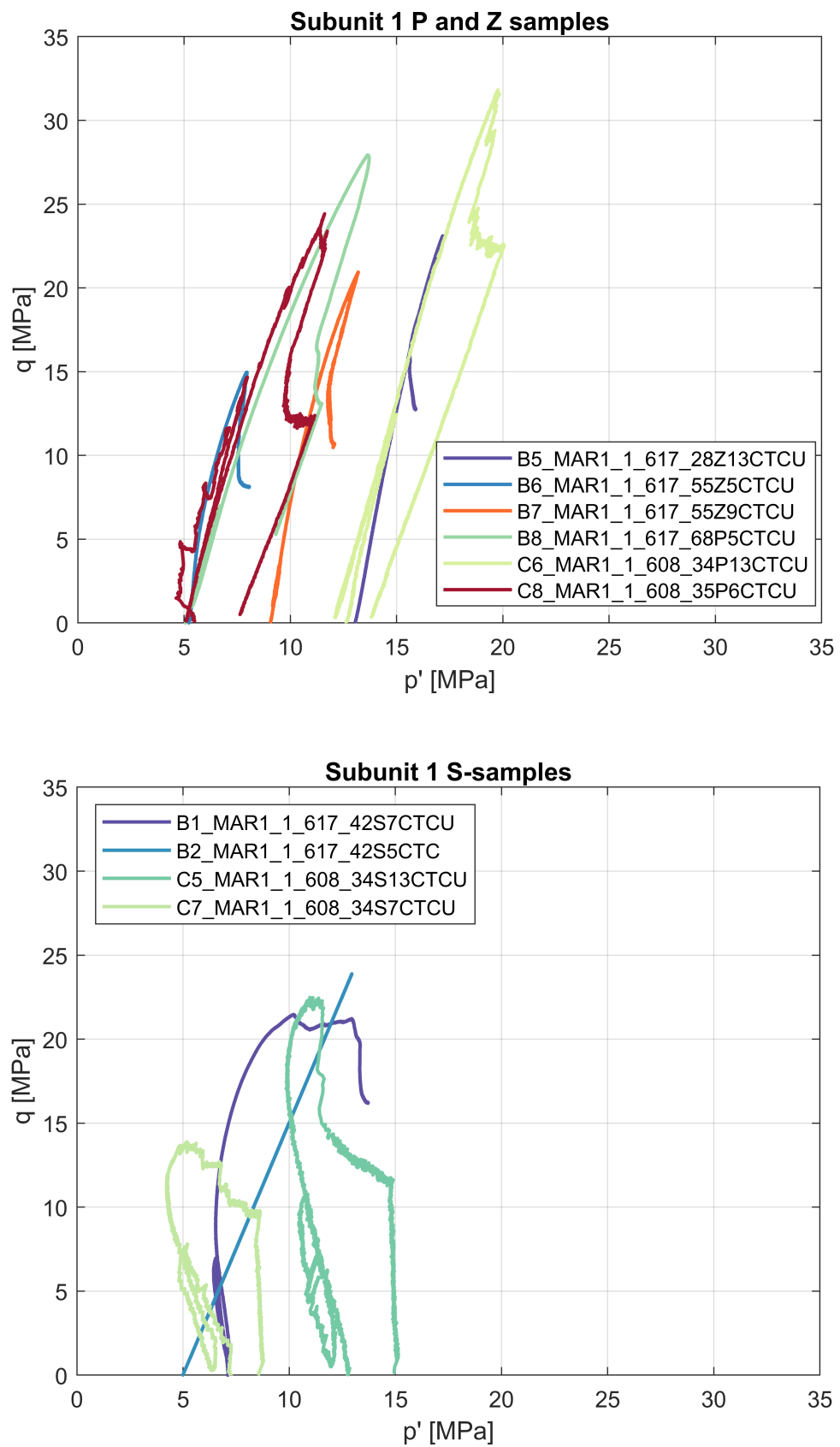


Fig. 6-6: Effective stress paths of tested specimens grouped by the lithological (sub-)unit

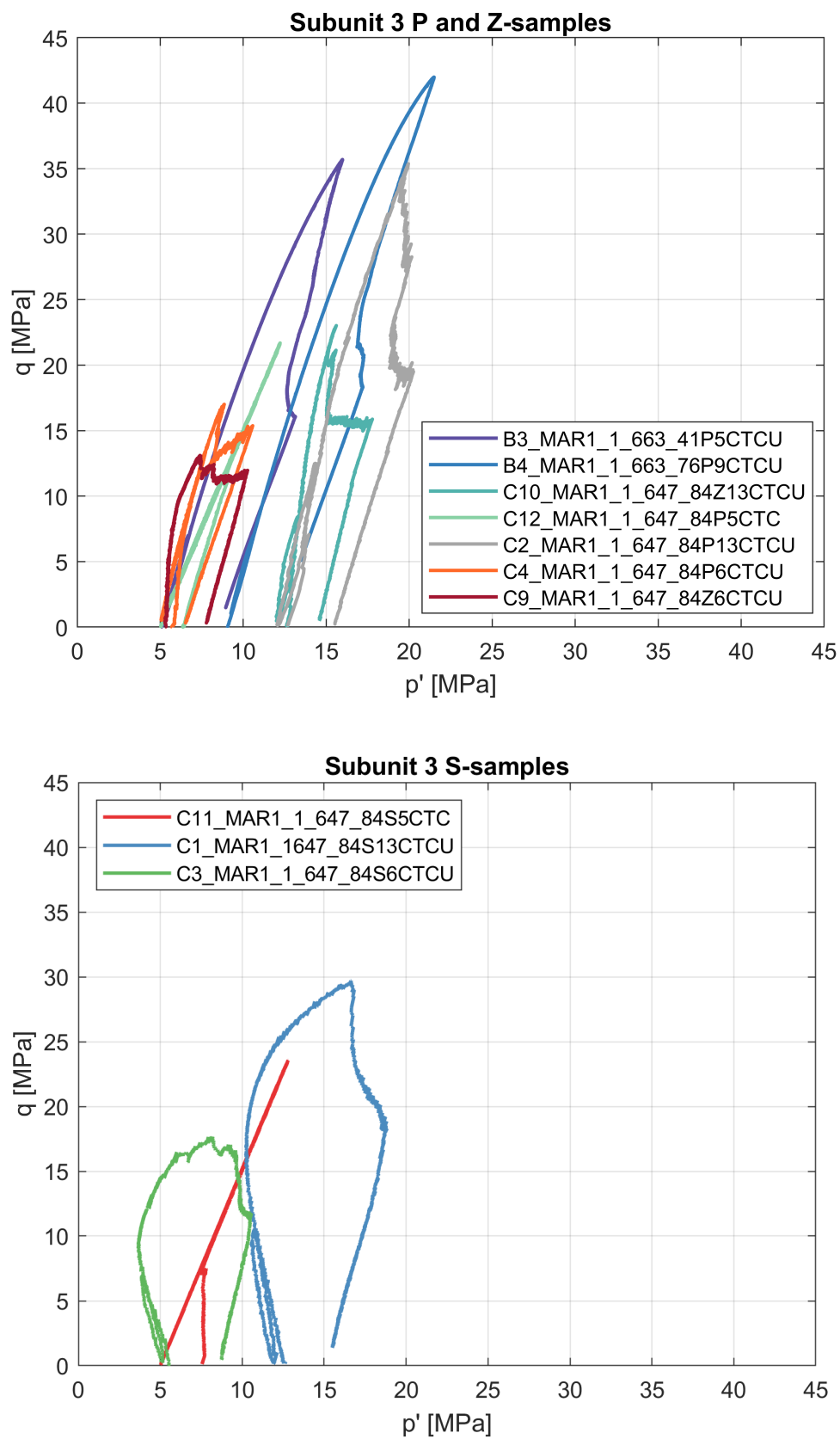


Fig. 6-6: (continued)

Some tests exhibited an unusual stress path evolution, or a lower peak shear strength compared to the majority of the results. These tests are analysed in more detail in Section 6.3.

6.2.2.2 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. 6-7). The strain calculation refers to the initial specimen 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. 6-7) according to the procedure presented in Giger et al. (2018).

An example of elastic moduli evaluation is reported in Fig. 6-7, with reference to test BUL1_1_950_62S9CTCU_B (BUL1-1 testing campaign, Nagra 2021). The top graph in Fig. 6-7 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. 6-8. 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. 6-9, sorted by specimen final water content. In the case of P-tests, higher values of the undrained elastic moduli with respect to S-tests are observed.

Results are reported in Fig. 6-10 (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. 6-10, divided by specimen 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 B and C. In particular, the 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 S and P waves in the axial (V_p), and of P waves in the radial (V_{ph}) direction. These configurations allowed the estimation of the wave velocities considering the anisotropic structure of the tested specimens (bedding orientation).

Results are shown in Fig. 6-11. 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 B measured wave velocities at the end of the consolidation phase. However, the effect of the difference in deviatoric load among laboratories seems negligible.

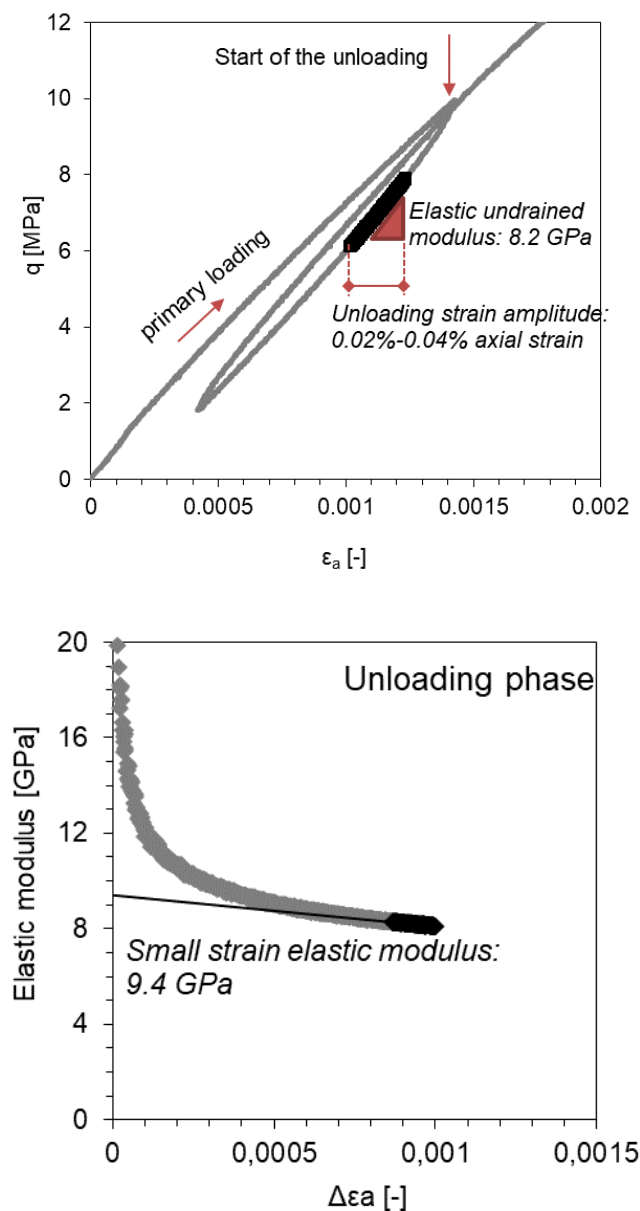


Fig. 6-7: Unloading-reloading cycle and small strain elastic modulus in triaxial tests

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, Nagra 2021).

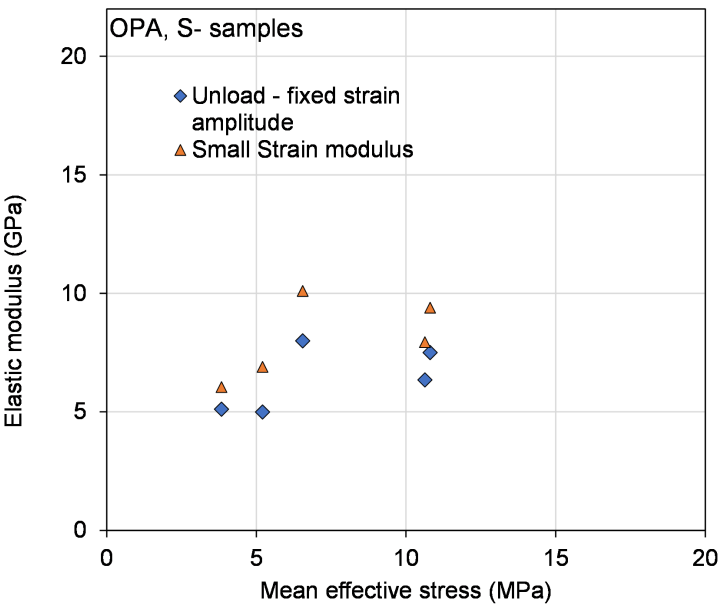


Fig. 6-8: Elastic undrained moduli calculated over an unloading (UL) path: modulus of a fixed strain interval and small strain extrapolation

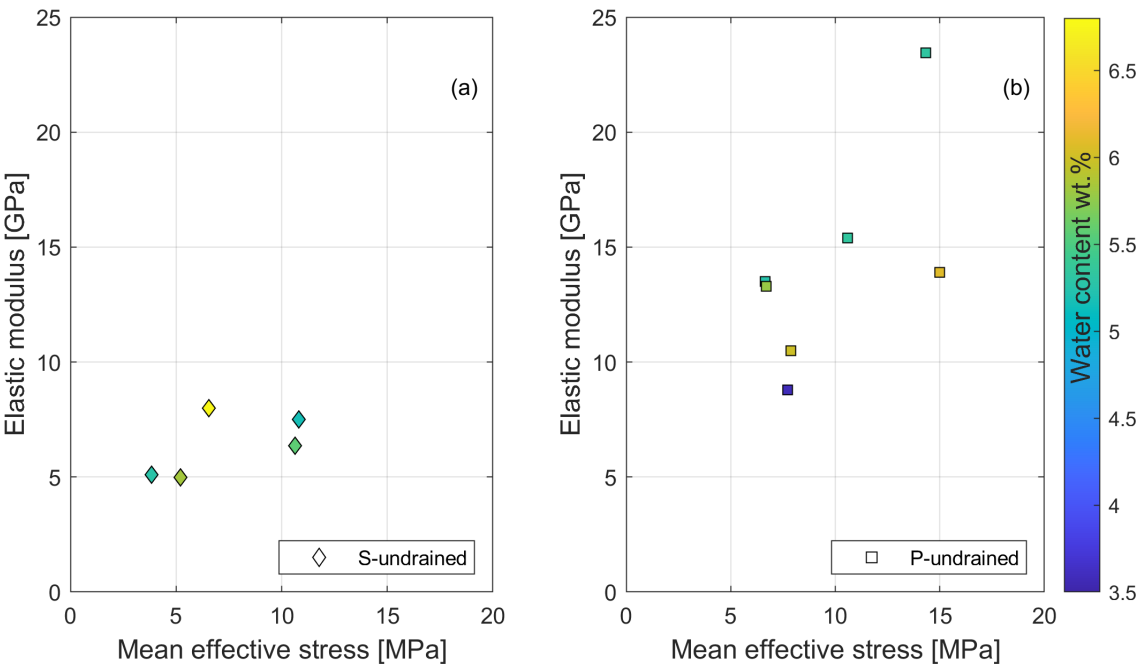


Fig. 6-9: Elastic undrained moduli, sorted by specimen final water content
Moduli are calculated over a fixed strain interval during an unloading (UL) path: P and S geometries, sorted by water content at the end of the test (final).

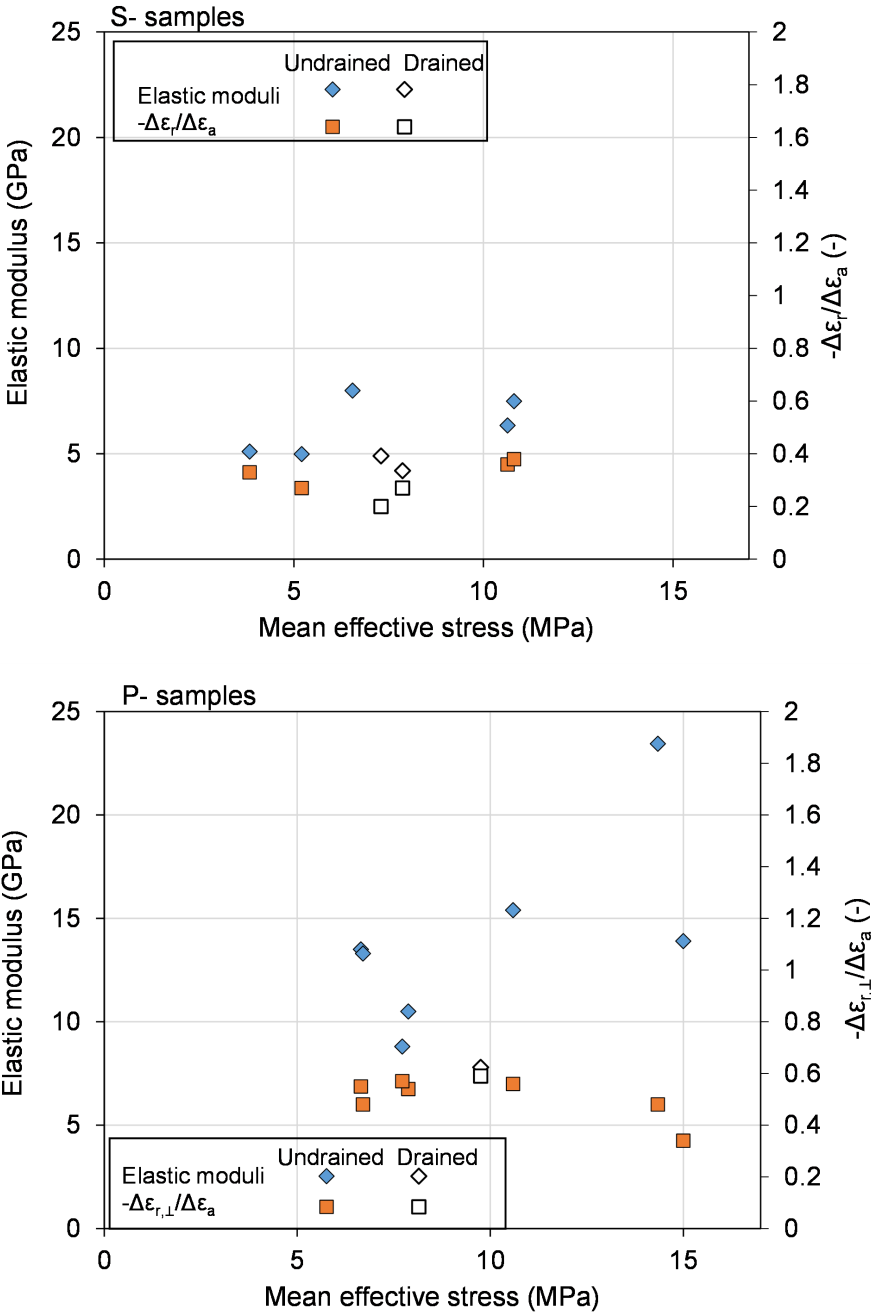


Fig. 6-10: Elastic drained and undrained properties

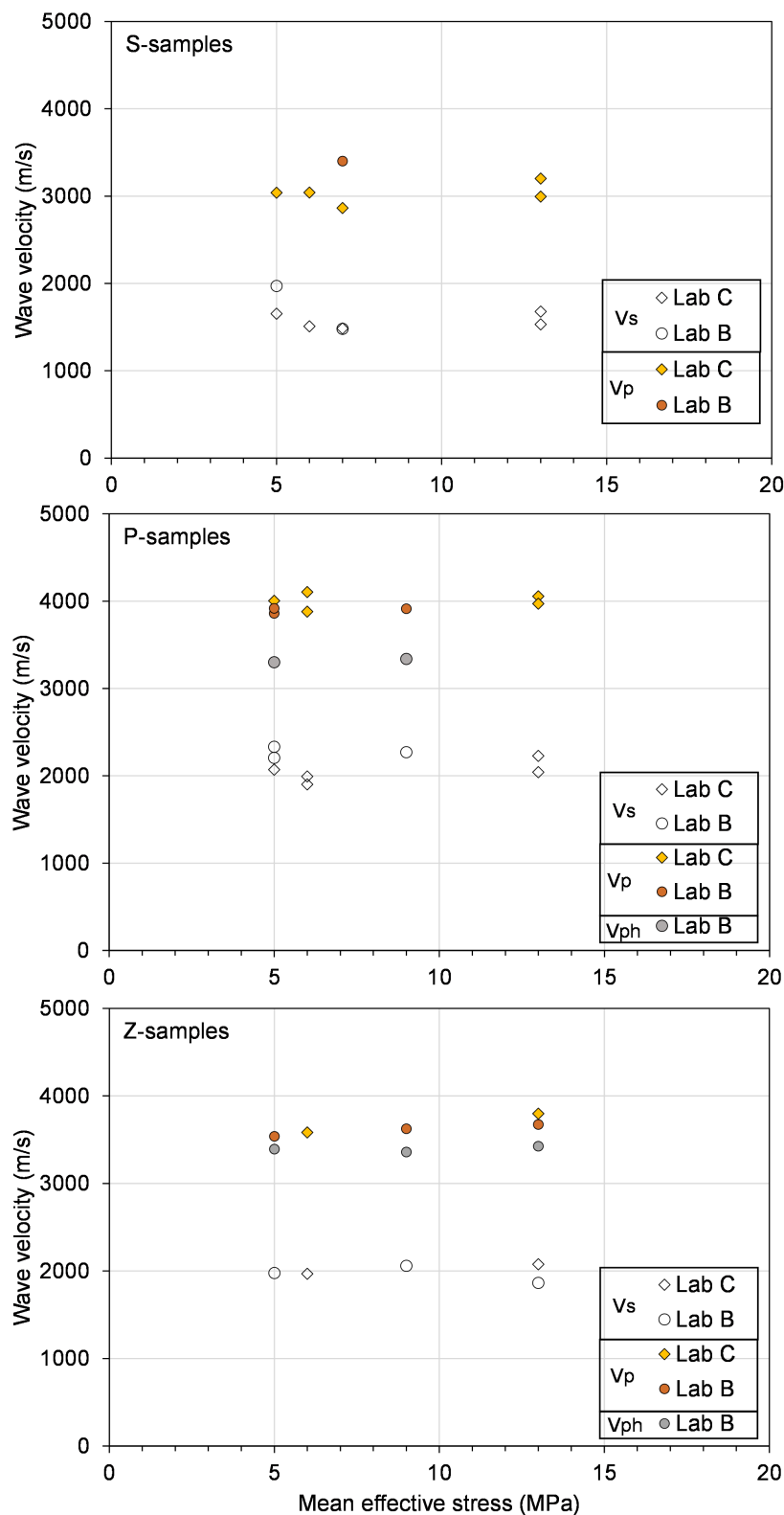


Fig. 6-11: Wave velocities at the beginning of the shearing phase

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

6.2.2.3 Pore pressure evolution

In Fig. 6-12, the stress levels ($p' - q$), at which the maximum value of the AB parameter and the maximum pore pressure u_w are 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 Δu_w and q . The results are divided by specimen geometry. Drained test results do not include values of Δu_w and AB as porewater pressure is maintained constant throughout shearing.

On each graph, the tests that showed an unexpected response (i.e. peak strength considerably lower than the majority of the tests or inconsistent stress path evolution) are highlighted. Those tests are indicated with a short code, being the initial part of the Specimen ID (Section 6.2), and are analysed in detail in the following sections (Sections 6.2.2.5 and 6.2.2.6).

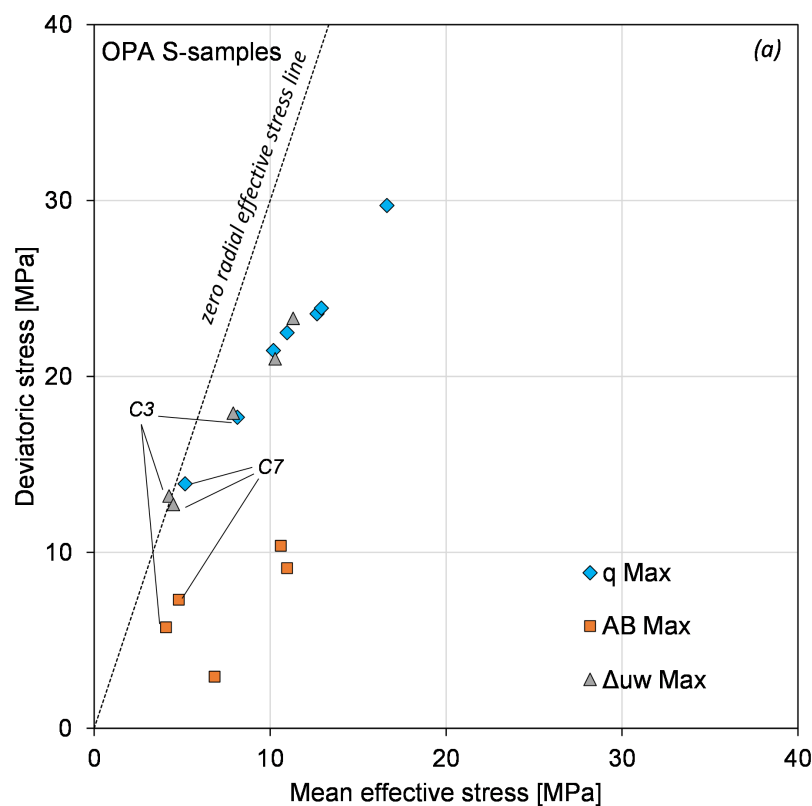


Fig. 6-12: Stress level at which the maximum deviatoric stress, the maximum value of pore pressure and the maximum value of the AB parameter are achieved

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

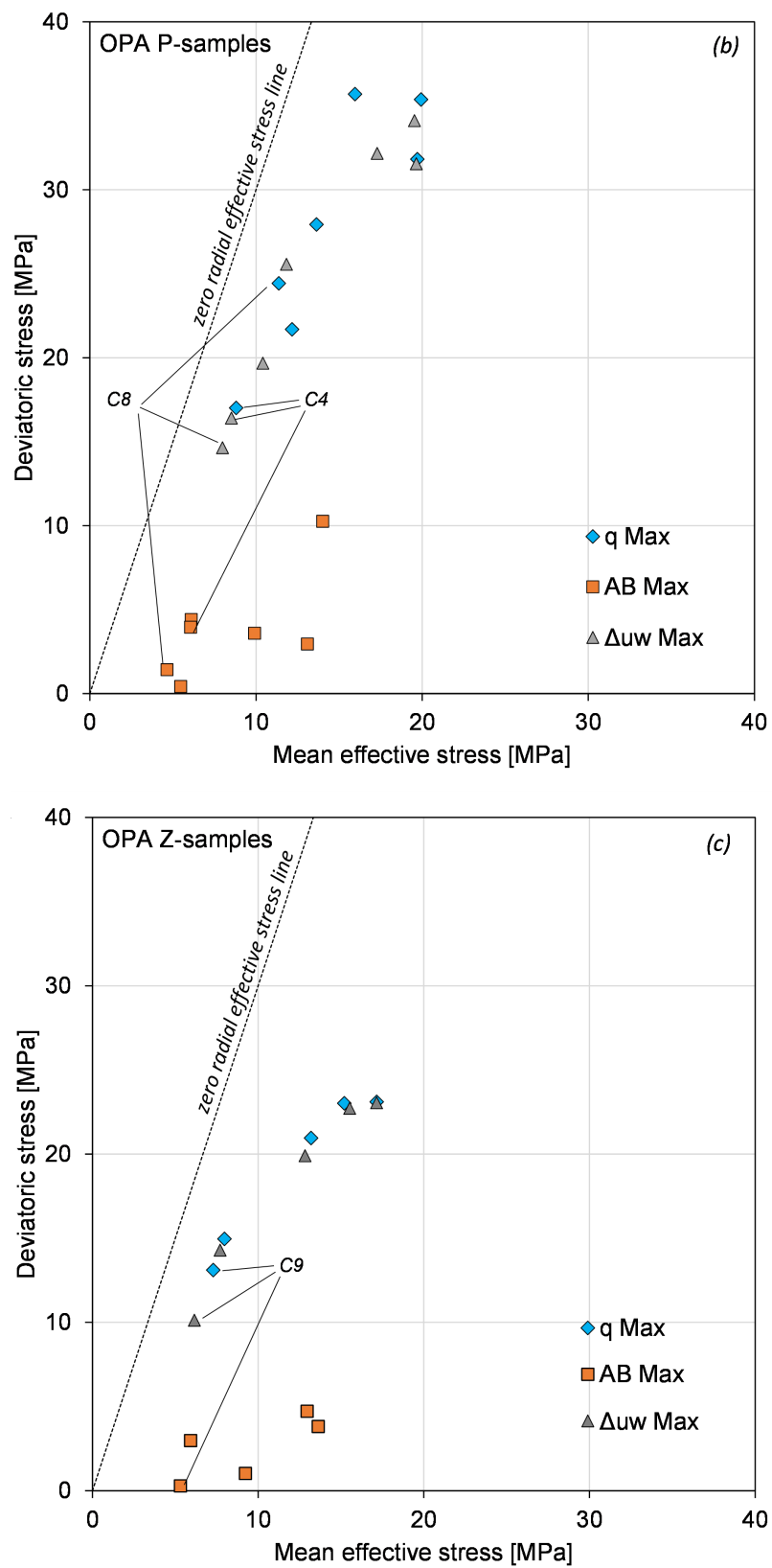


Fig. 6-12: (continued)

6.2.2.4 Shear strength

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

In Fig. 6-13, peak (a) and post-peak shear strength (b) are reported in the $p' - q$ plane. Only Opalinus Clay results are reported and grouped by specimen geometry. The dotted lines highlight the pair of $p' - q$ values, at which radial effective stress equals zero. This line would correspond to the same boundary conditions as a UCS test if pore fluid pressure was zero. The stress states on the left side of this line cannot be achieved (negative radial effective stress) in a triaxial apparatus.

In Fig. 6-14 the peak shear strength results are analysed, sorted by the water content (Fig. 6-14a) of the specimens (obtained at the end of the test) and by the clay mineral content (Fig. 6-14b), where available. The plot includes the results of S-samples of Opalinus Clay. The results are reported also in Fig. 6-15 in terms of radial versus axial effective stress.

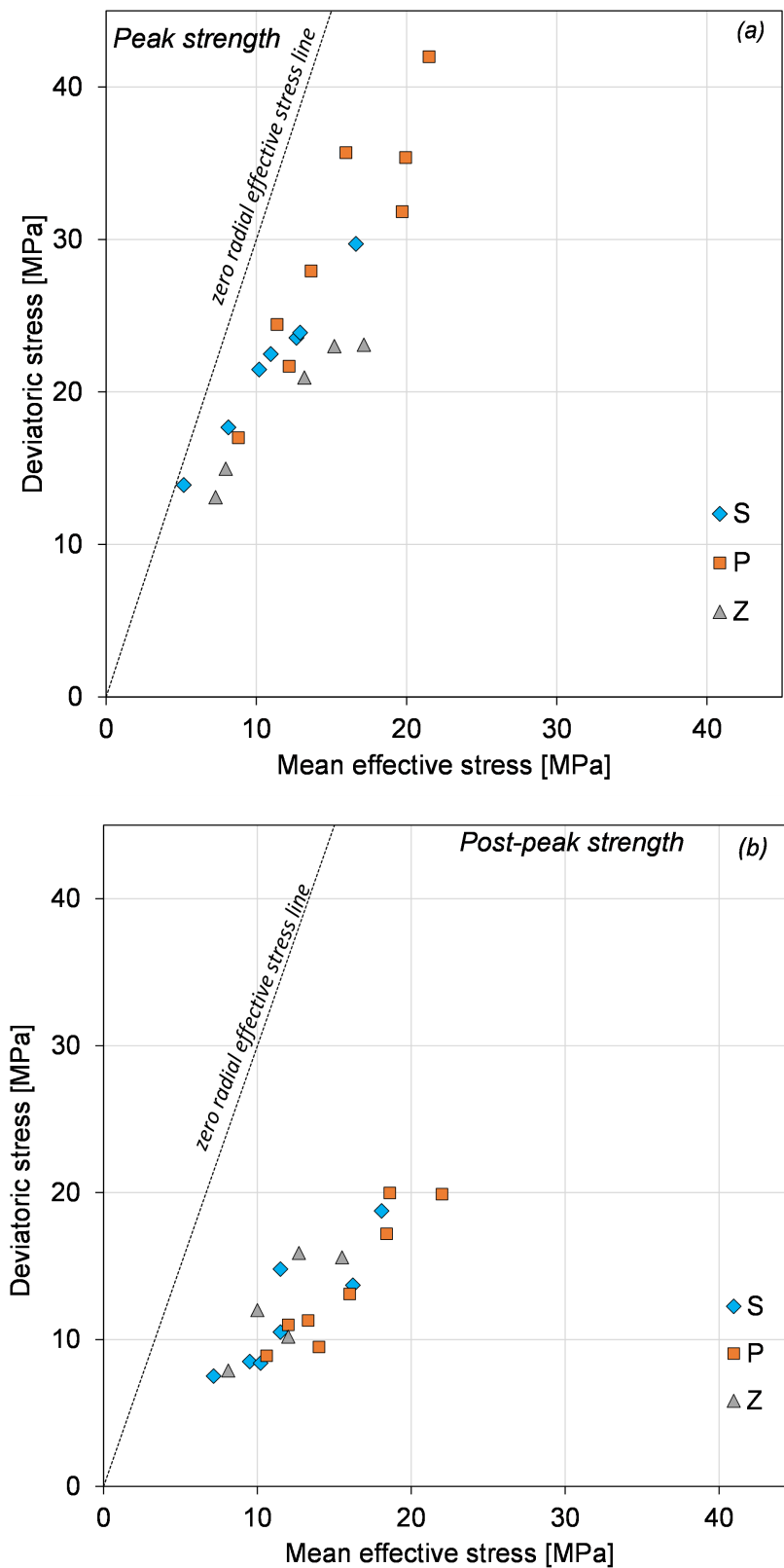


Fig. 6-13: Peak (a) and post-peak (b) shear strength of Opalinus Clay MAR1-1 specimens
Results divided by geometry.

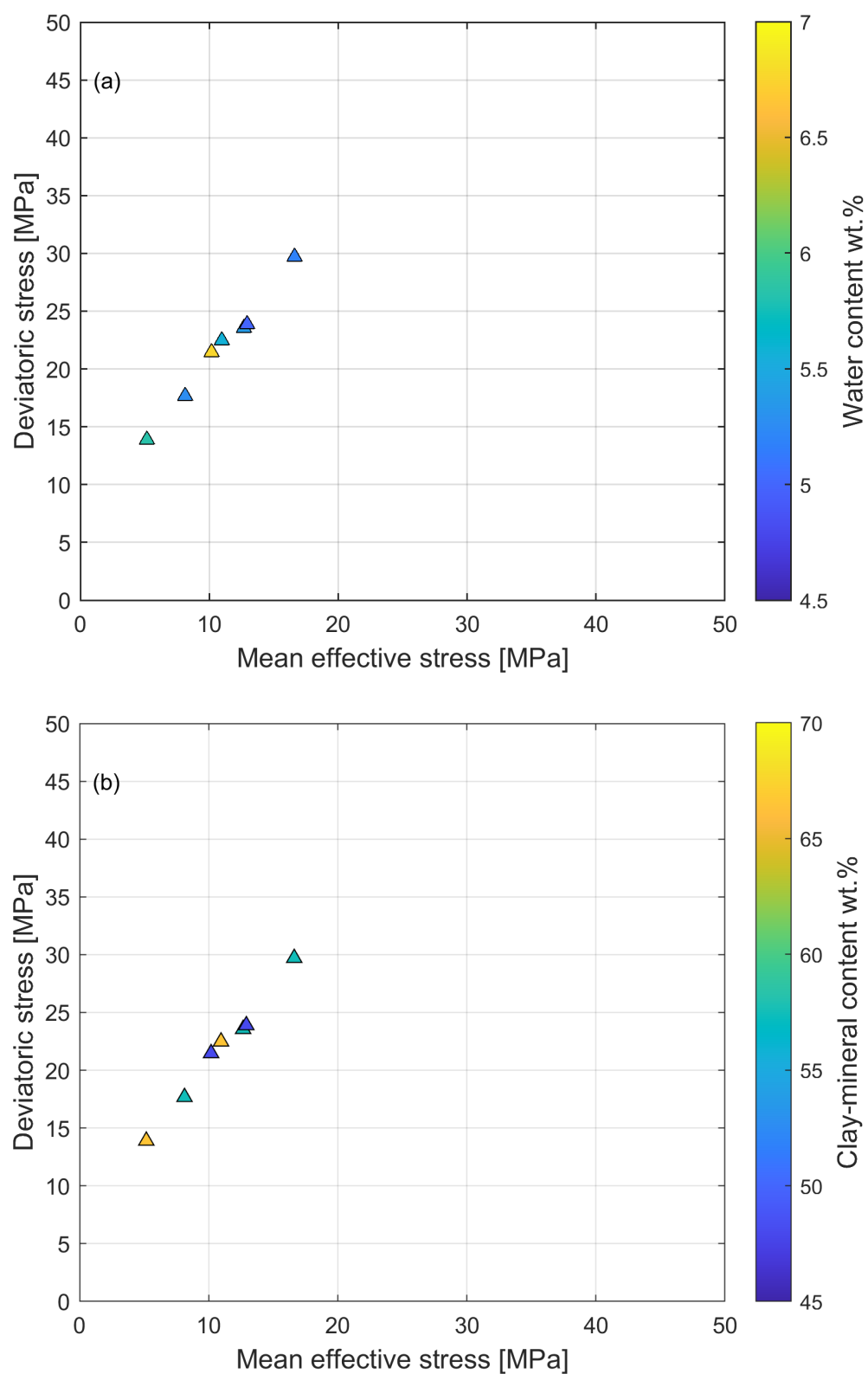


Fig. 6-14: Peak shear strength of specimens of Opalinus Clay

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

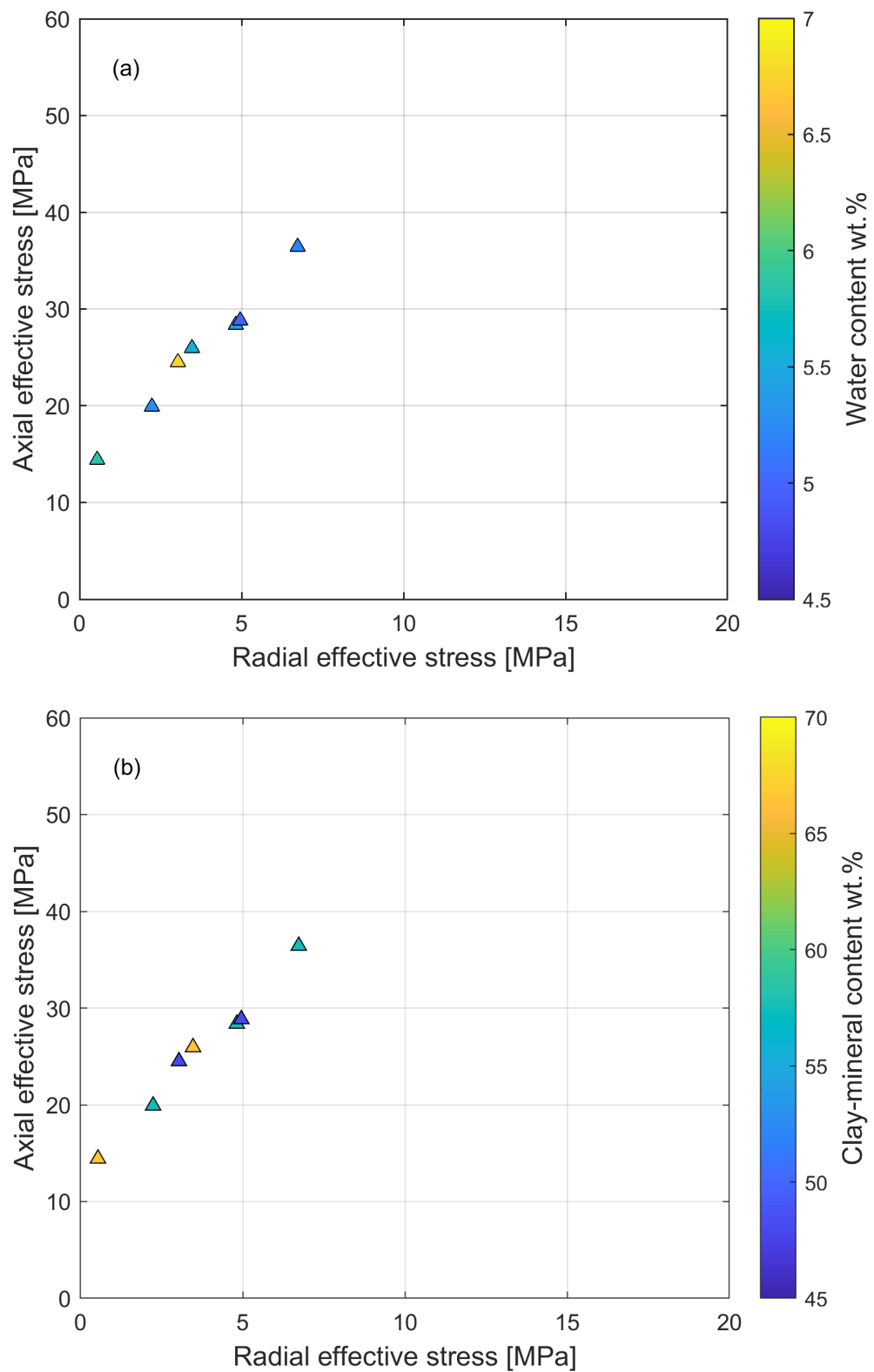


Fig. 6-15: Peak shear strength of specimens of Opalinus Clay

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

6.2.2.5 Analysis of the results of the Z-samples

Results of the five Z-tests (bedding oriented to 30° – 35° from cylinder axis and loading direction) are analysed in this section. Fig. 6-16 reports the axial strain (top) and stress path (bottom) during the shearing phase of the tests.

Among the Z-tests, C9_MAR1_1_647_84Z6CTCU did not exhibit a distinct softening response during shearing: the maximum shear strength was reached at axial strain of 0.006, about twice the axial strain at the peak of the other tests with the same geometry. The peak deviatoric strength is lower compared to the other Z-test results at similar confining effective stress, which can be an indication of a pre-damage of the samples. This geometry was selected 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.

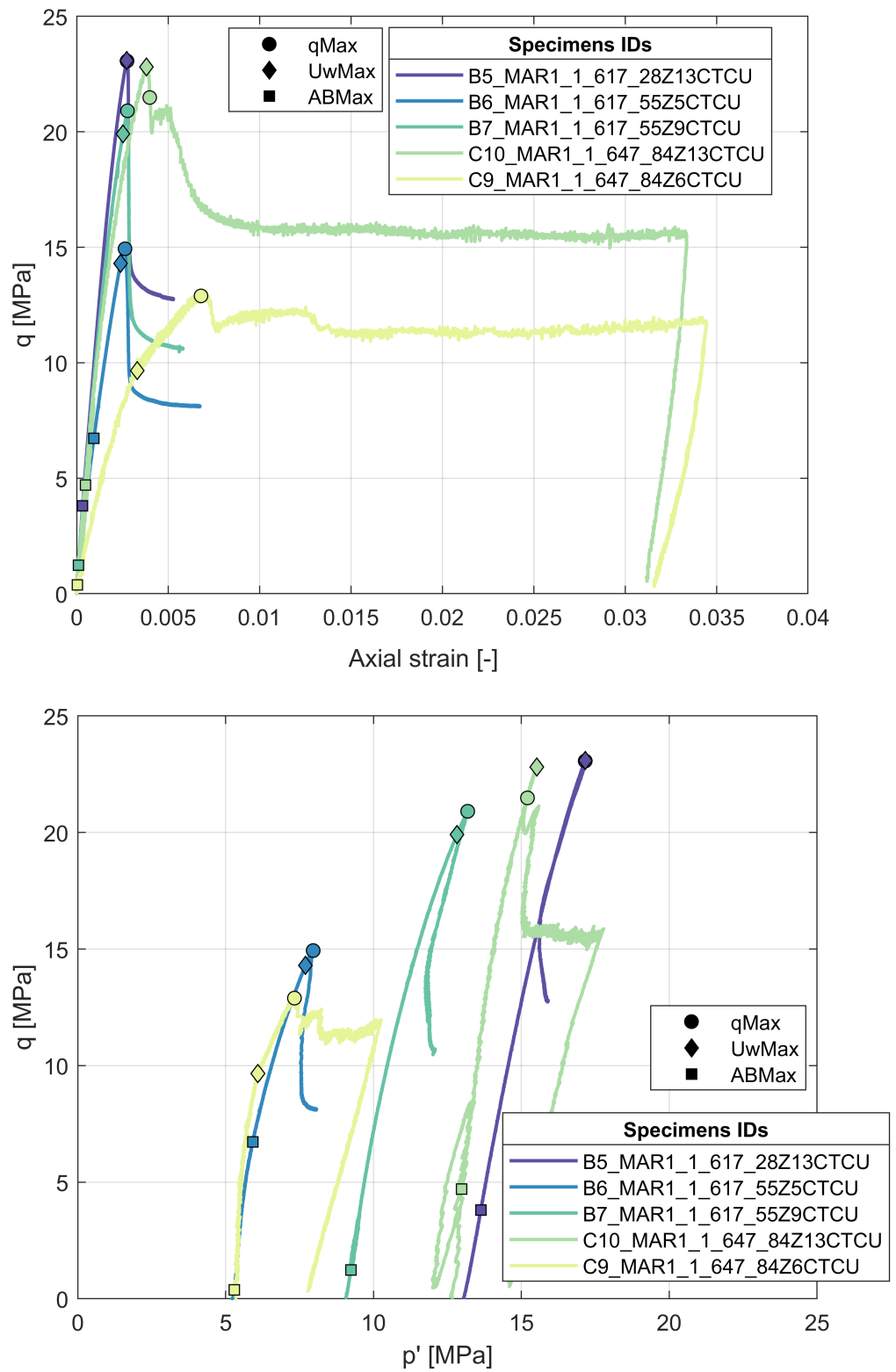


Fig. 6-16: Axial strain (top) and stress path (bottom) during the shearing phase of Z-geometry specimens

6.2.2.6 Analysis of the outlier results

In this section, tests that showed a response significantly different from the majority of the results (outliers) are analysed in more detail. Diagnostic plots are also available in Appendix C.

Test C8_MAR1_1_608_35P6CTCU

In the initial phase of the shearing, recurrent instability in the axial strain rate caused oscillations in deviatoric stress and pore pressure (Fig. 6-17). As a consequence, the stress path (Fig. 6-18) presents several jumps in the initial phase. After the unloading, the stability of the strain rate was restored, and the stress path evolved similarly to test B8_MAR1_1_617_68P5CTCU, which had similar testing conditions.

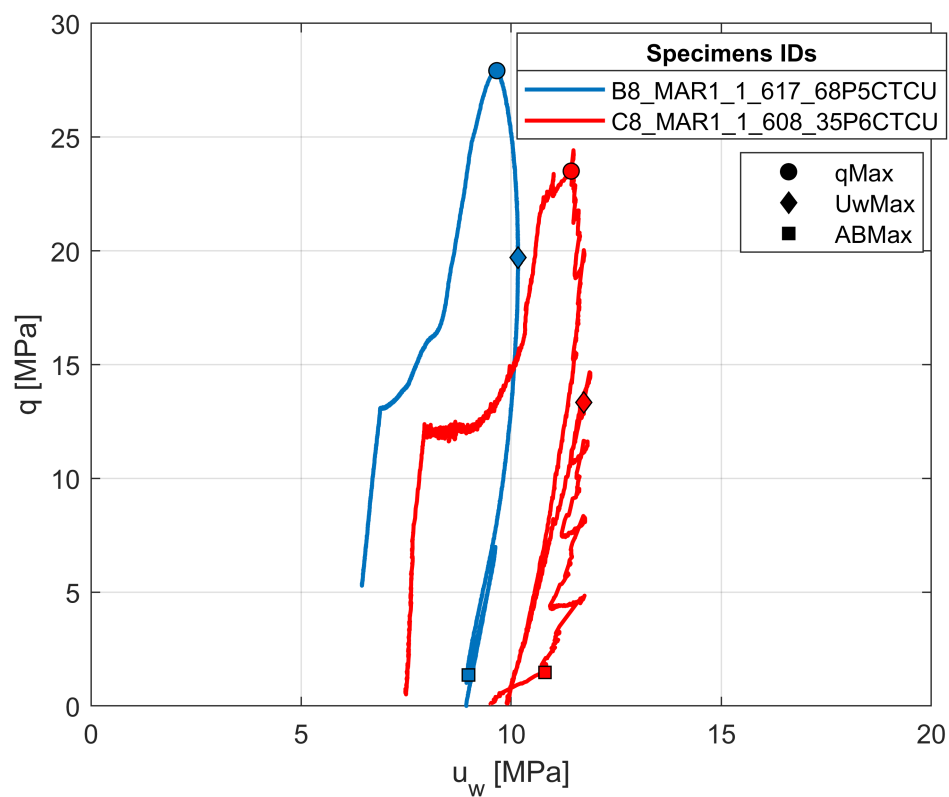


Fig. 6-17: Deviatoric stress versus pore pressure during shearing, Test C8_MAR1_1_608_35P6CTCU

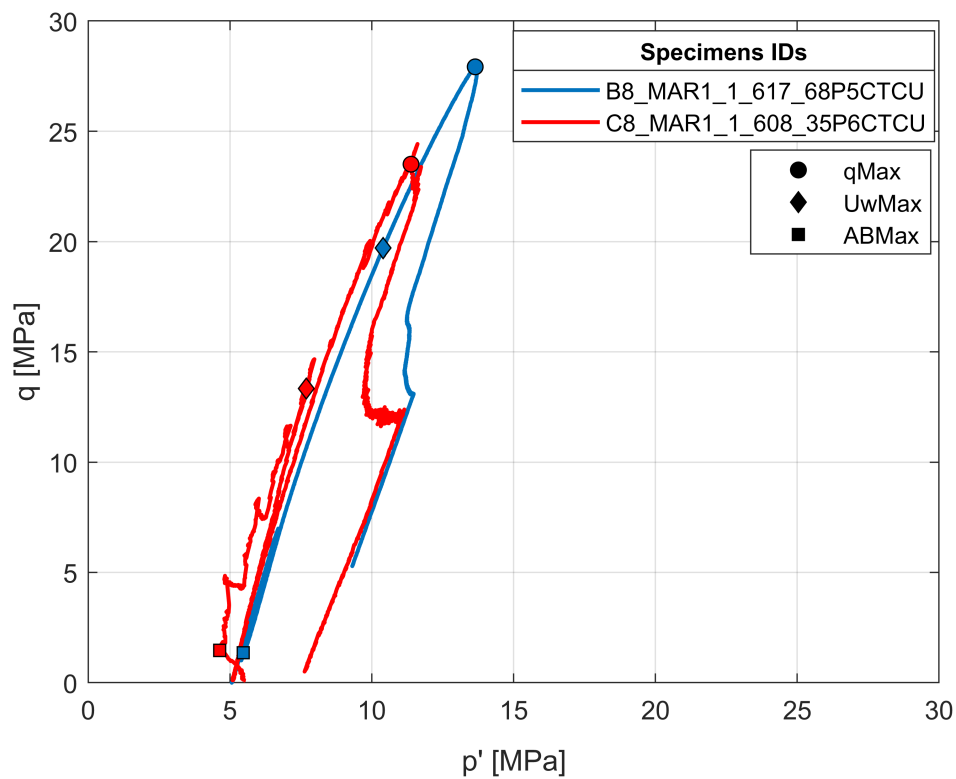


Fig. 6-18: Stress path of test C8_MAR1_1_608_35P6CTCU

Tests C7_MAR1_1_608_34S7CTCU and C3_MAR1_1_647_84S6CTCU

During shearing, test C7_MAR1_1_608_34S7CTCU showed a decrease in strength in steps: after the peak, the shear stress reduced rapidly with the axial strain, then stabilised for a long axial strain increase, then reduced again rapidly four times. The evolution of the deviatoric stress versus axial strain is reported in Fig. 6-19. A similar unexpected response was observed, although less marked in test C3_MAR1_1_647_84S6CTCU.

Figs. 6-20 and 6-21 show the evolution of the stress path and of the pore pressure during shearing for the two tests. Pore pressure evolution increased in both cases up to a value close to the radial confining stress, bringing the effective confinement close to zero.

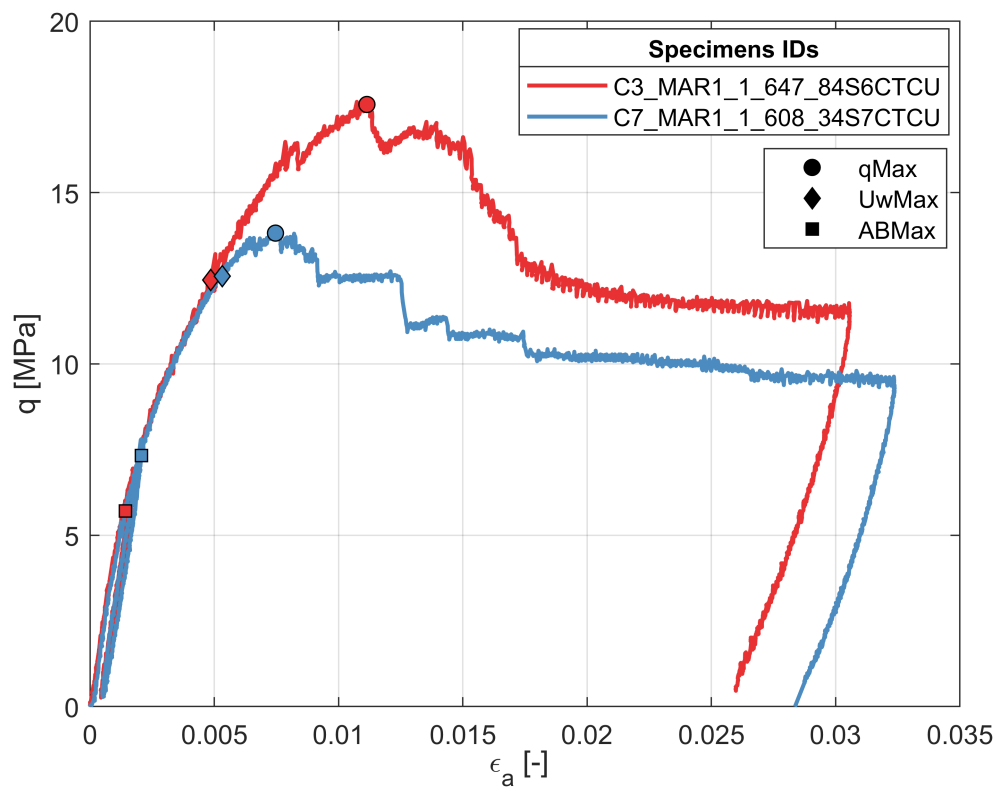


Fig. 6-19: Deviatoric stress versus axial strain during shearing

Test C7_MAR1_1_608_34S7CTCU and C3_MAR1_1_647_84S6CTCU.

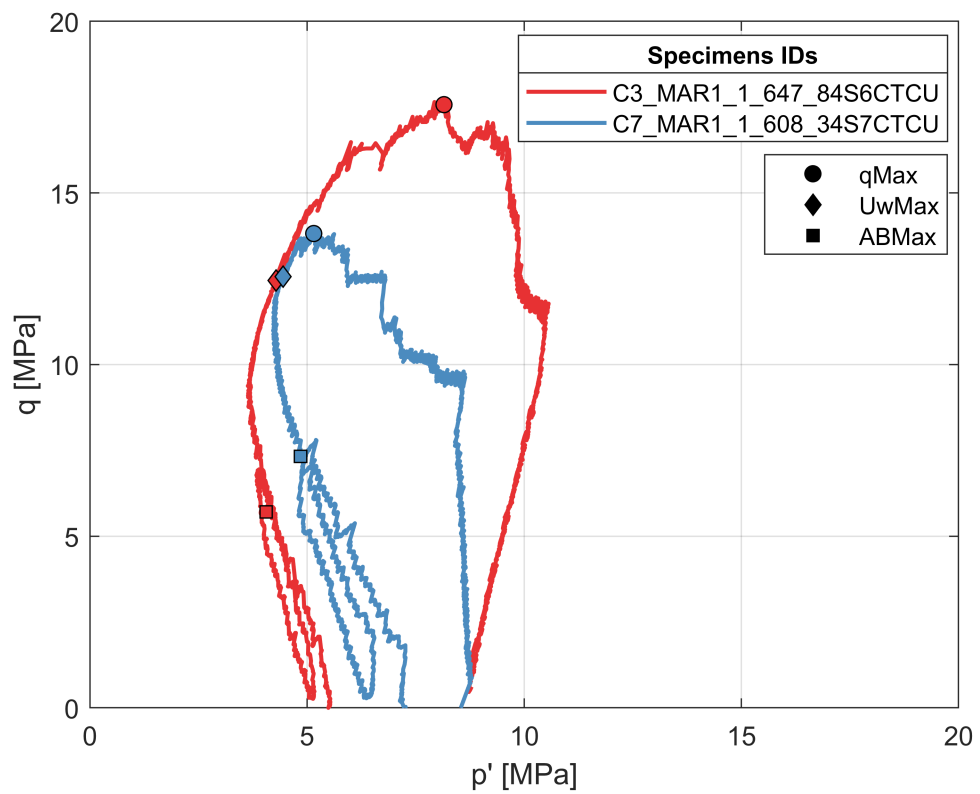


Fig. 6-20: Stress paths evolution

Test C7_MAR1_1_608_34S7CTCU and C3_MAR1_1_647_84S6CTCU.

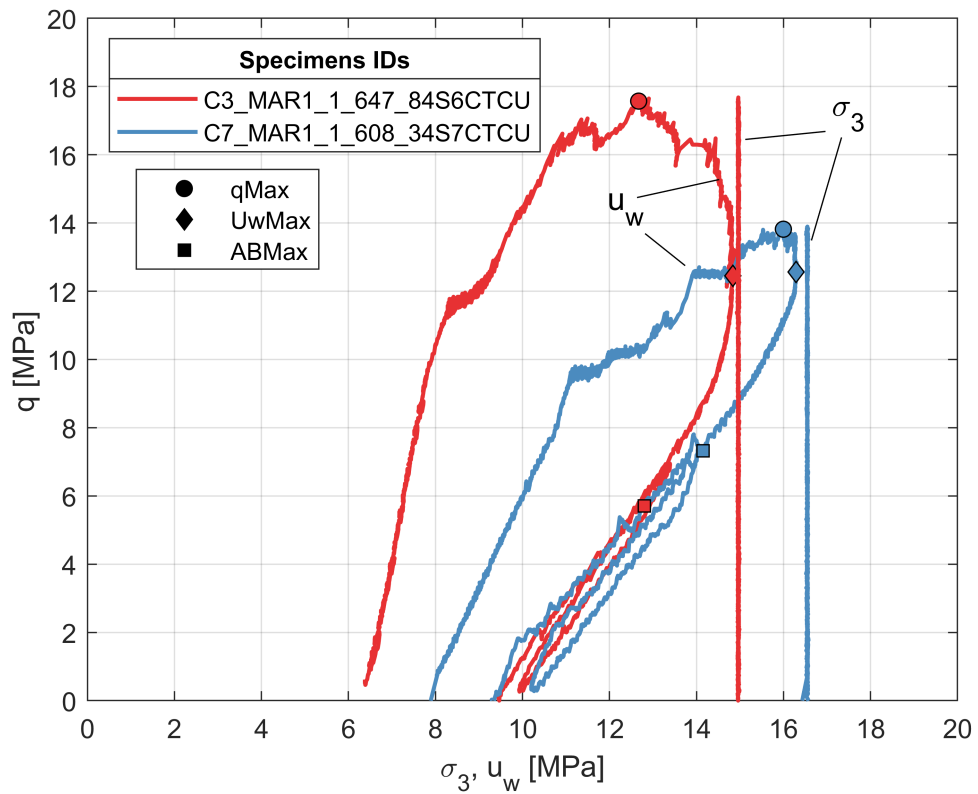


Fig. 6-21: Pore pressure and radial total stress evolution

Test C7_MAR1_1_608_34S7CTCU and C3_MAR1_1_647_84S6CTCU.

Test C4_MAR1_1_647_84P6CTCU

During shearing of test C4_MAR1_1_647_84P6CTCU (Figs. 6-22, 6-23), peak strength was achieved at a stress (16.7 MPa) level lower compared to the other tests with similar testing conditions (e.g. B3_MAR1_1_663_41P5CTCU). After the peak, the deviatoric stress decreased to 12.2 MPa, then tended to increase again as shearing continued. At axial strain $\sim 3\%$, the deviatoric stress had re-gained more than half of the stress decrement after the peak (shearing stopped at $q = 14.9$ MPa).

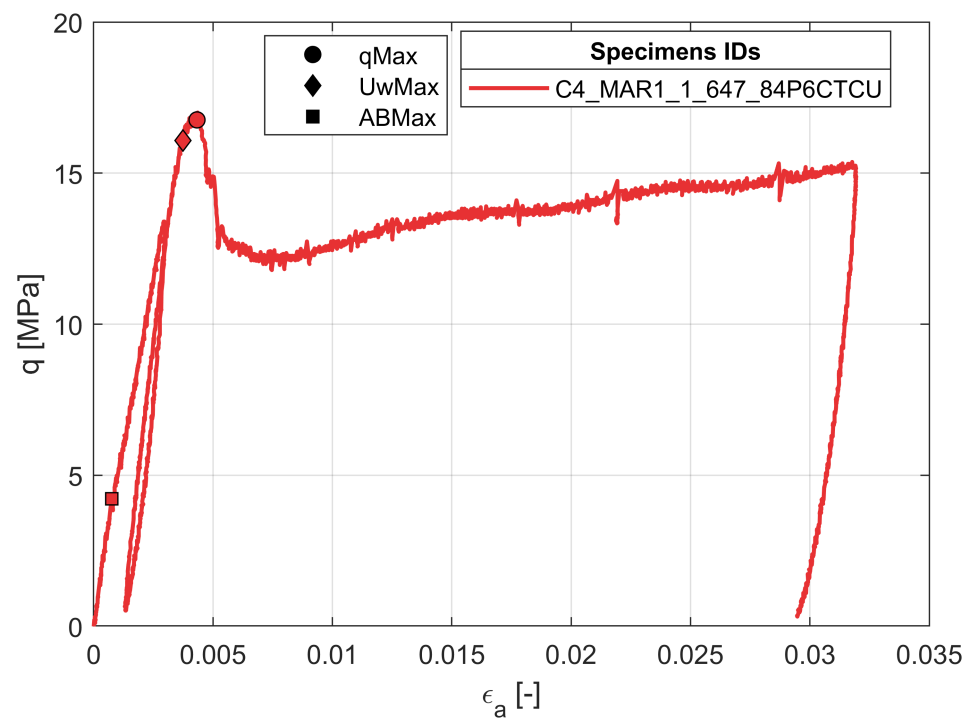


Fig. 6-22: Deviatoric stress versus axial strain during shearing, Test C4_MAR1_1_647_84P6CTCU

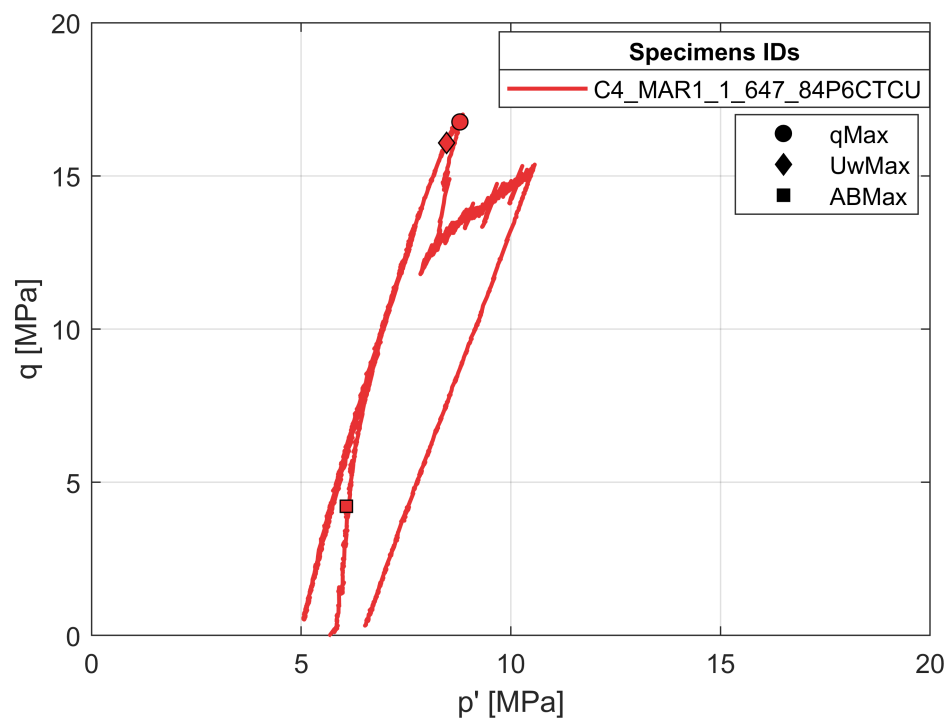


Fig. 6-23: Stress path of test C4_MAR1_1_647_84P6CTCU

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

This section includes the results obtained on:

- High-pressure oedometric tests (OED), including constant-head permeability tests (PERM)
- One-dimensional swelling test (OS), either allowing axial swelling in free stress conditions or at constant stress

The OED and OS tests were obtained in oedometric (lateral strain prevented) conditions. For 5 specimens, during oedometric tests, a constant-head permeability phase was performed.

6.3.1 Overview of the performed tests and initial specimen conditions

In the context of investigating the swelling behaviour of Opalinus Clay, the testing programme included some addition to the procedure described in Section 4.3.2. For 3 samples, one-dimensional swelling was obtained at constant total stress, and in 1 case the test was then continued as a high-pressure oedometric test. In 1 test, free swelling was allowed, then the test continued as OED (high-pressure). An overview of the performed tests is provided in Tab. 6-3.

The tests in oedometric conditions were labelled by indicating the letter of the laboratory that performed the test (e.g. Lab A), a sequential number (01) and the type of test (OED = oedometric, OS = one-dimensional swelling).

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

For specimen geometry see Fig. 4-1. ^a Total stress reported in brackets for swelling under constant stress. ^b High-pressure oedometric test.

ID	OS ^a	OED ^b	Per m	Depth [m]	Labora- tory	Specimen geometry	Formation
A03OED_MAR1_1			×	506.94	A	P	Variansmergel Fm.
A03bOED_MAR1_1		×		506.94	A	P	Variansmergel Fm.
D04OED_MAR1_1		×	×	535.08	D	P	«Parkinsoni-W.-Sch.»
D06OED_MAR1_1		×	×	551.96	D	P	«Humphriesoolith Fm.»
D07OED_MAR1_1		×		629.77	D	S	Opalinus Clay
D08OED_MAR1_1		×		629.77	D	P	Opalinus Clay
A01OED_MAR1_1		×	×	677.16	A	P	Opalinus Clay
A02OED_MAR1_1		×		677.26	A	S	Opalinus Clay
D01OED_MAR1_1	×	×		702.09	D	S	Opalinus Clay
D02OED_MAR1_1		×		702.09	D	S	Opalinus Clay
D03OED_MAR1_1	(0.1)	×		702.00	D	S	Opalinus Clay
D05OED_MAR1_1		×	×	702.05	D	P	Opalinus Clay
A03_OS_MAR1_1	×			507.00	A	P	Variansmergel Fm.
D04_OS_MAR1_1	×			629.80	D	S	Opalinus Clay
D05_OS_MAR1_1	×			629.84	D	P	Opalinus Clay
A01_OS_MAR1_1	×			677.22	A	P	Opalinus Clay
A02_OS_MAR1_1	×			677.30	A	S	Opalinus Clay
D01_OS_MAR1_1	(1.0)			702.05	D	S	Opalinus Clay
D02_OS_MAR1_1	(0.02)			702.05	D	S	Opalinus Clay
D03_OS_MAR1_1	×			702.00	D	P	Opalinus Clay

6.3.2 Oedometric test results

The settlement over time curve for each loading increment is analysed adopting an extension to the 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 in 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 are reported in Fig. 6-23, 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.

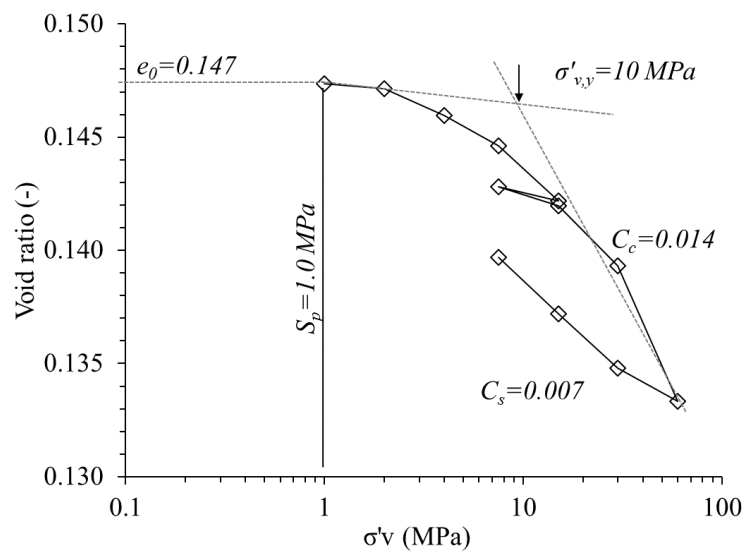


Fig. 6-23: Oedometric curve in void ratio versus vertical effective stress plane
Results from A03bOED_MAR1_1.

Tab. 6-4: Oedometric test results

For specimen geometry see Fig. 4-1. ^a Void ratio at the end of the swelling under constant stress.

Specimen ID	Specimen geometry	Specimen depth	e_0	C_c	S_p	$\sigma'_{v,y}$	C_s	PERM test at σ'_v
		[m]	[-]	[-]	[MPa]	[MPa]	[-]	[MPa]
A03OED_MAR1_1	P	506.94		-	-	-	-	8.0
A03bOED_MAR1_1	P	506.94	0.147	0.014	1.0	10	0.007	-
D04OED_MAR1_1	P	535.08	0.134	0.009	5.6	10	0.004	7.0
D06OED_MAR1_1	P	551.96	0.134	0.008	1.7	10	0.003	7.0
D07OED_MAR1_1	S	629.77	0.174	0.025	5.8	17	0.014	-
D08OED_MAR1_1	P	629.77	0.147	0.013	7.0	20	0.005	-
A01OED_MAR1_1	P	677.16	0.129	0.016	1.0	13	0.008	8.0
A02OED_MAR1_1	S	677.26	0.151	0.023	4.0	11	0.016	-
D01OED_MAR1_1	S	702.09	0.171 (0.238 ^a)	0.032	-	1.5	0.014	-
D02OED_MAR1_1	S	702.09	0.192	0.035	4.5	15.5	0.020	-
D03OED_MAR1_1	S	702.00	0.166 (0.192 ^a)	0.025	-	4.1	0.016	-
D05OED_MAR1_1	P	702.05	0.176	0.023	8.7	-	0.005	7.0

A wide range of swelling pressures was measured in oedometric conditions, ranging from about 1 MPa to 9 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 Fig. 6-24, 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. Data of hydraulic conductivity computed from the consolidation and those from constant-head tests are reported in Tab. 6-5. It is noted that the hydraulic conductivity in the direction perpendicular to bedding (S) is generally lower than the direction parallel to bedding (P), in agreement with literature data. An exception is represented by the results from tests D01OED_MAR1_1 and D03OED_MAR1_1, where the sample is allowed to swell at low- or none-vertical stress, causing a considerable increase in porosity, which is reflected in an increment in hydraulic conductivity for the considered stress range of 5 – 20 MPa (swelled samples marked in Fig. 6-24).

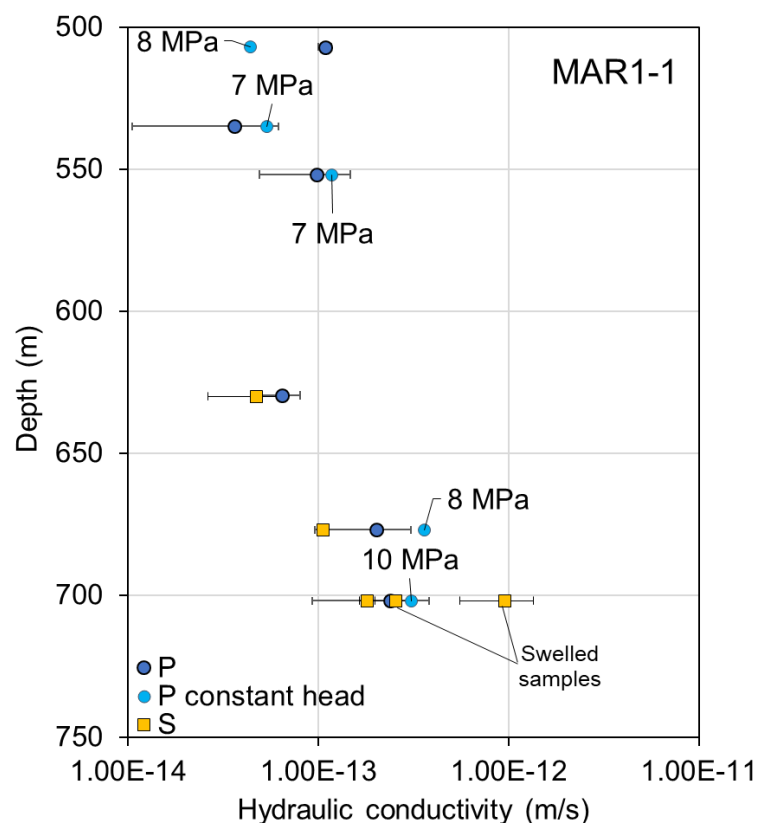


Fig. 6-24: Hydraulic conductivity versus depth

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

Tab. 6-5: Hydraulic conductivity test results: from consolidation and constant-head tests

Specimen ID	Specimen geometry	Depth	Consolidation stages		Constant-head tests	
			Average k (4 – 20 MPa)	Standard deviation	k	σ'_v
		[m]	[m/s]	[m/s]	[m/s]	[MPa]
A03OED_MAR1_1	P	506.94	-	-	4.37×10^{-14}	8.0
A03bOED_MAR1_1	P	506.94	1.1×10^{-13}	8.9×10^{-15}		
D04OED_MAR1_1	P	535.08	3.6×10^{-14}	2.6×10^{-14}	5.38×10^{-14}	7.0
D06OED_MAR1_1	P	551.96	9.8×10^{-14}	4.9×10^{-14}	1.18×10^{-13}	7.0
D07OED_MAR1_1	S	629.77	4.7×10^{-14}	2.1×10^{-14}		
D08OED_MAR1_1	P	629.77	6.4×10^{-14}	1.6×10^{-14}		
A01OED_MAR1_1	P	677.16	2.0×10^{-13}	1.1×10^{-13}	3.62×10^{-13}	8.0
A02OED_MAR1_1	S	677.26	1.1×10^{-13}	7.8×10^{-15}		
D01OED_MAR1_1	S	702.09	9.5×10^{-13}	4.0×10^{-13}		
D02OED_MAR1_1	S	702.09	1.8×10^{-13}	1.7×10^{-14}		
D03OED_MAR1_1	S	702.00	2.5×10^{-13}	6.9×10^{-14}		
D05OED_MAR1_1	P	702.05	2.4×10^{-13}	1.4×10^{-13}	3.08×10^{-13}	10.0

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. 6-25. The results are summarised in Tab. 6-6 (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 four tests, injecting pressure at 50 kPa and applying constant stress to counterbalance the fluid pressure. Lab A injected water at ~ 7 kPa, counterbalanced by the weight of the top piston.

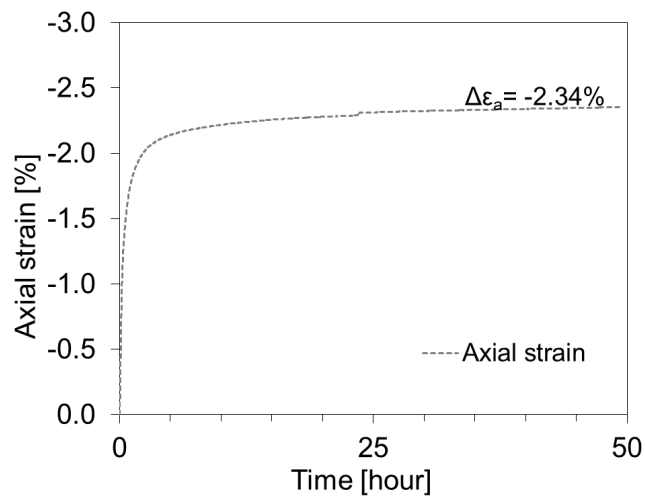


Fig. 6-25: One-dimensional swelling test, evolution of the axial strain in time
Results obtained on specimen A02OS_MAR1_1.

Tab. 6-6: One-dimensional swelling test results: unconstrained swelling and swelling under constant load

For specimen geometry see Fig. 4-1.

Specimen ID	Core depth	Specimen geometry	Formation	ϵ_{swell} (absolute value)	Vertical effective stress	u_w
	[m]			[%]	[kPa]	[kPa]
A03_OS_MAR1_1	507.00	P	Variansmergel Fm.	0.57		~7
D04_OS_MAR1_1	629.80	S	Opalinus Clay	2.75		50
D05_OS_MAR1_1	629.84	P	Opalinus Clay	3.69		50
A01_OS_MAR1_1	677.22	P	Opalinus Clay	0.51		~7
A02_OS_MAR1_1	677.30	S	Opalinus Clay	2.34		~7
D01_OS_MAR1_1	702.05	S	Opalinus Clay	1.09	1.0	50
D02_OS_MAR1_1	702.05	S	Opalinus Clay	3.64	0.02	50
D03_OS_MAR1_1	702.00	P	Opalinus Clay	9.60		50

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.

One possible way to address this is to compare wave velocities derived from the sonic scanner *in situ* (Fig. 7-1 blue line) and ultrasonic velocities constrained in triaxial tests on cores in the laboratory (*cf.* Tabs. 5-3 and E-1, orange squares 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 115 m thick interval of Opalinus Clay, and each measurement reflects a mean value over a 25 cm core interval.

The water content of geomechanically tested specimens was measured on trimming material from the specimen preparation and on the specimens themselves (before and after testing). For a same specimen, these values were averaged, and the error bars constructed to include the variability in the measurements. Those results are reported versus the specimens sourcing depth in Fig. 7-2. Based on the water content, the comparison with the larger database reveals that the spectrum of water contents 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's 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 was 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 specimens from the same vertical interval of the core. The clay-mineral content (%) is reported in Fig. 7-3 against depth, together with the large database obtained from University of Bern (*cf.* Dossier VIII).

The comparisons of Figs. 7-2 and 7-3 demonstrates that tested specimens in the geomechanical testing programme capture the material variability and can thus be considered representative of the entire formation.

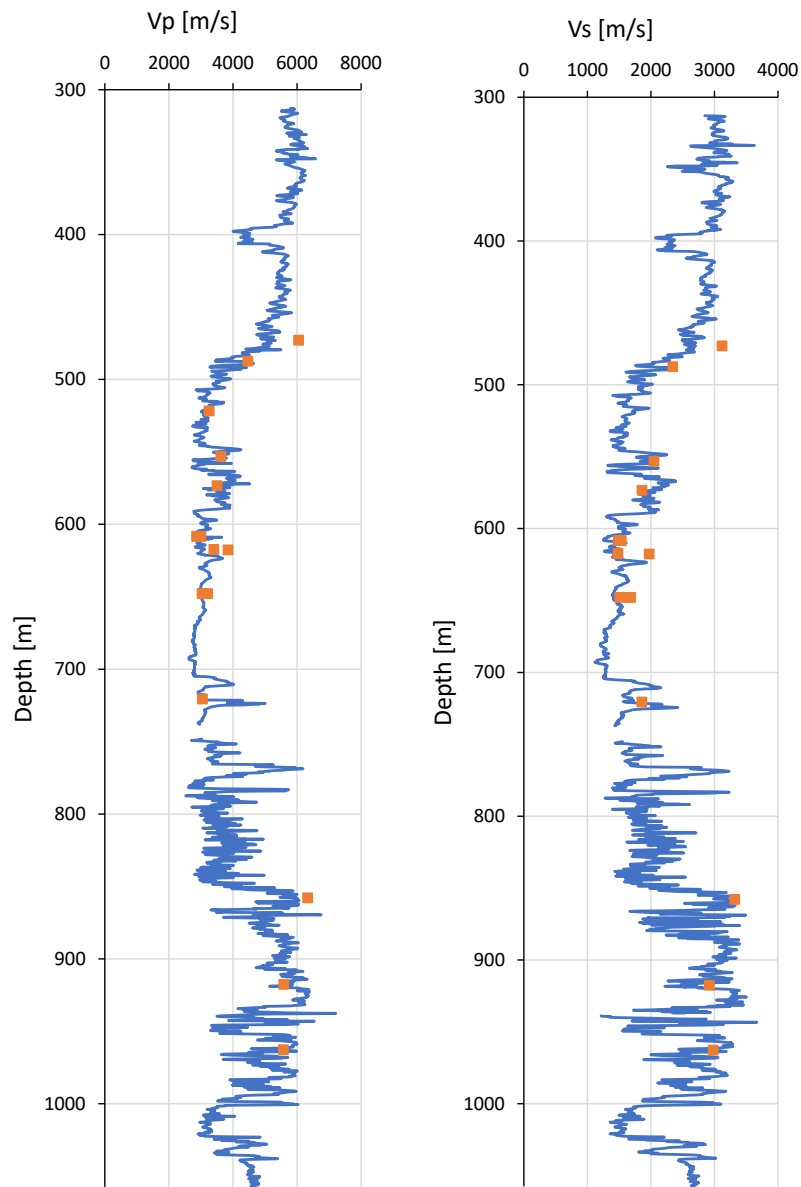


Fig. 7-1: Compressional (a) and shear-wave velocities (b) from borehole and lab testing

Blue line: continuous measurements from borehole sonic scanner (kHz frequency). The gap in the continuous velocity measurements at 740 m was due to constraints of the logging tool string in the selected casing scheme. Orange squares: lab-measurements (MHz frequency).

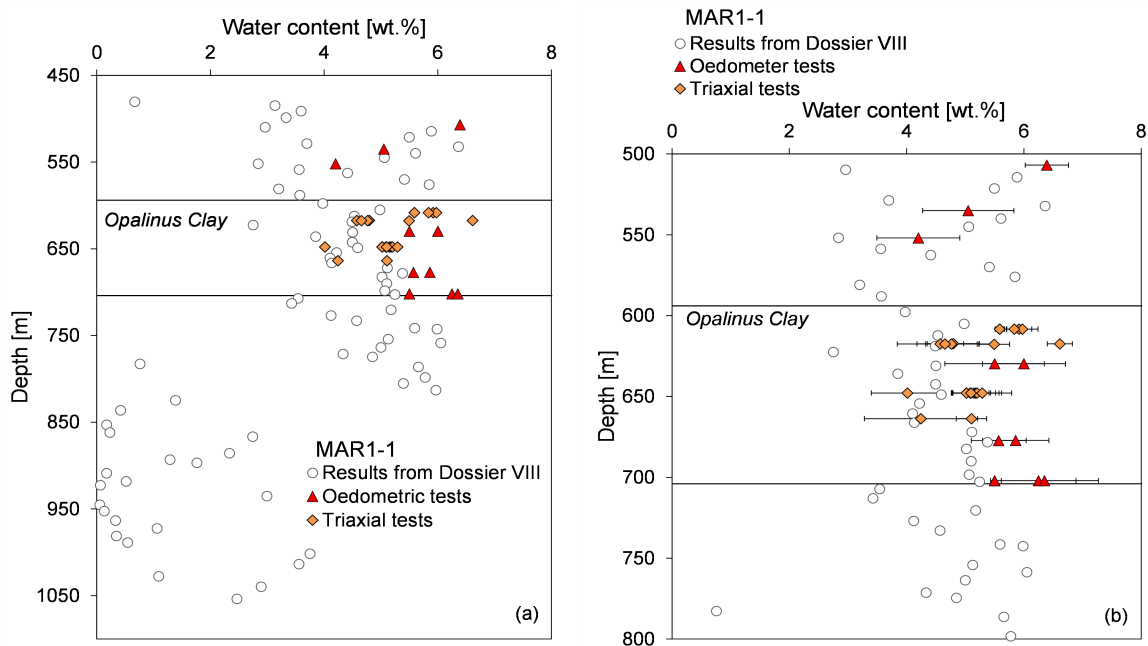


Fig. 7-2: Water content vs. depth from the large database and from specimens tested in this report

Results include those from Dossier VIII for the MAR1-1 borehole (a) and zoom into the Opalinus Clay and confining units' section (b).

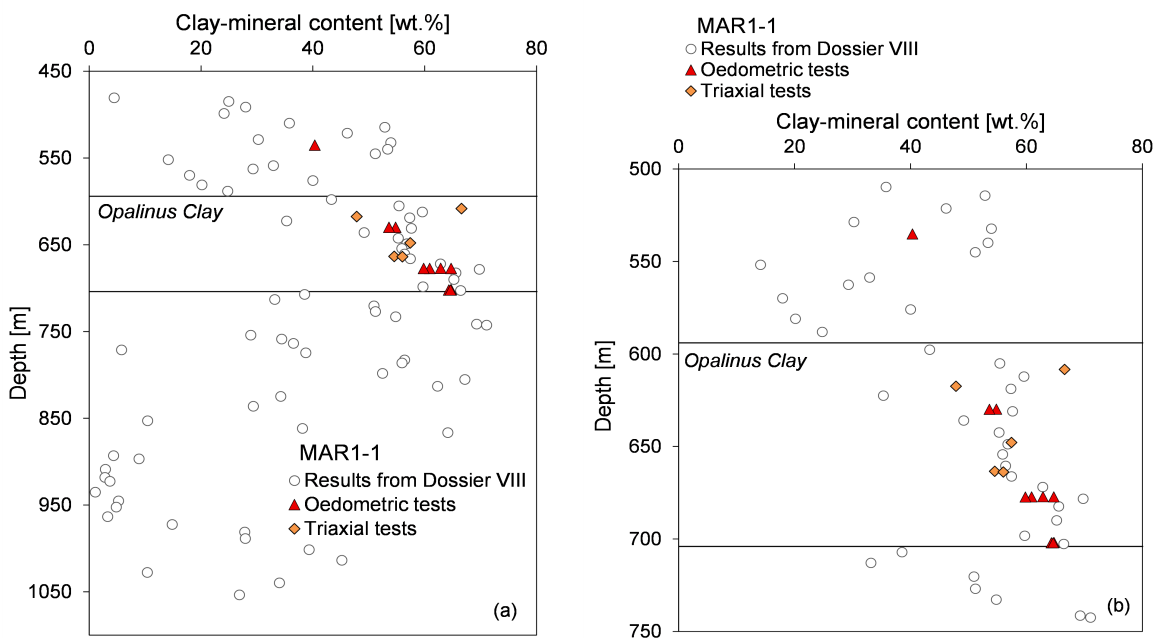


Fig. 7-3: Clay-mineral content vs. depth from the large database and from specimens tested in this report

Results include those from Dossier VIII for the MAR1-1 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

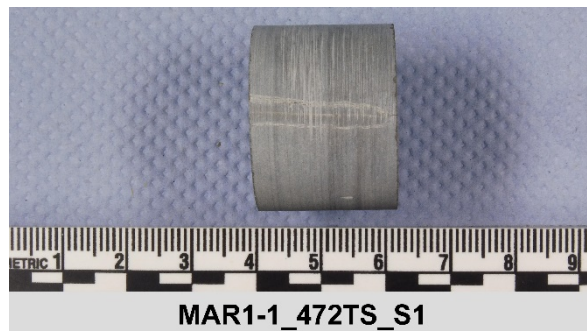
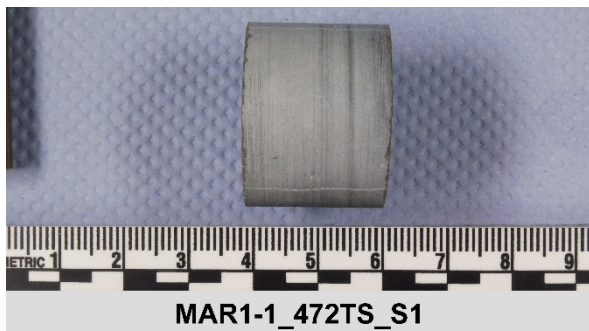
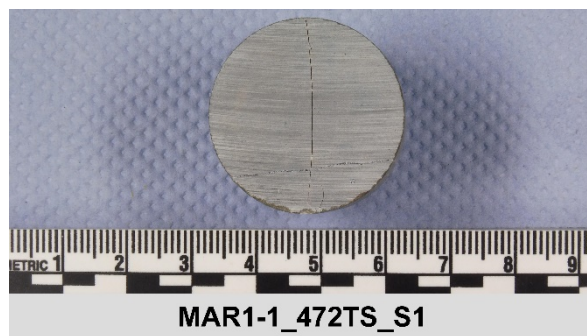
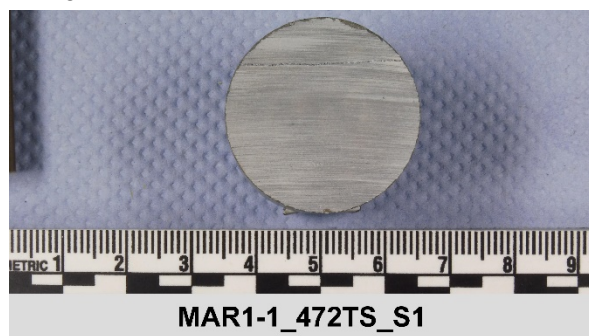
TS (average depth of sample)

TRU1-1

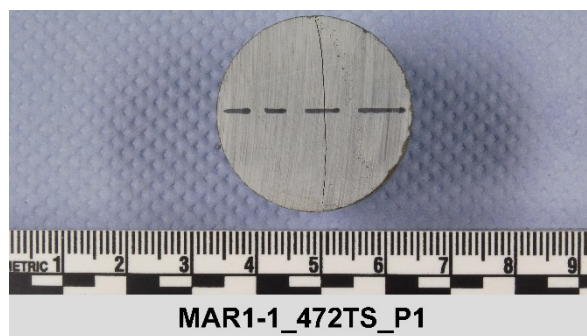
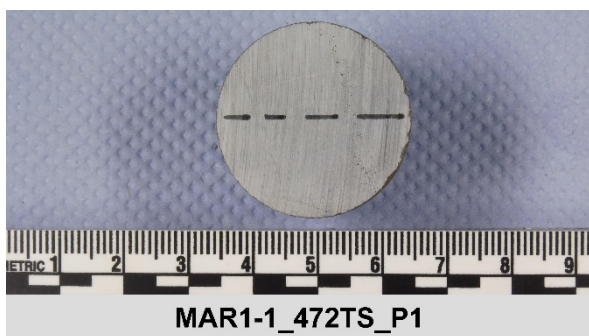
before testing

after testing

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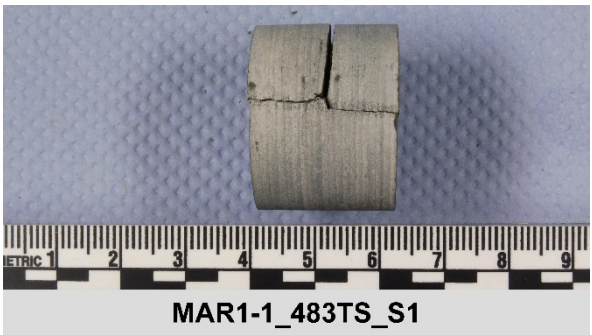
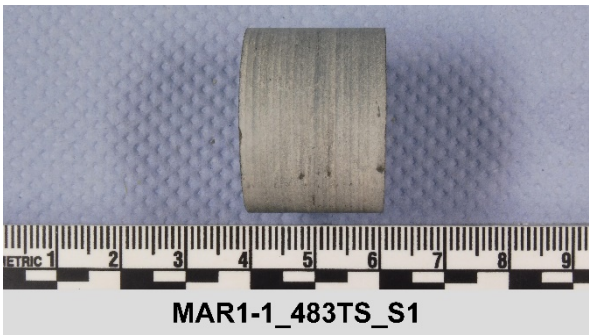
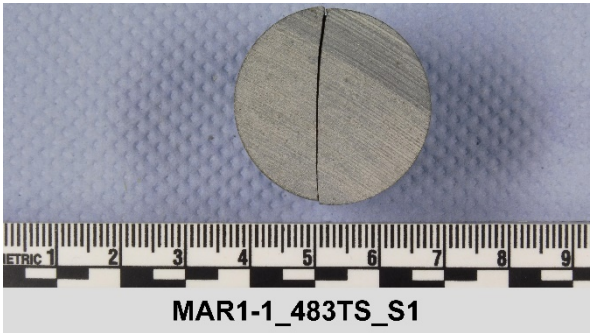
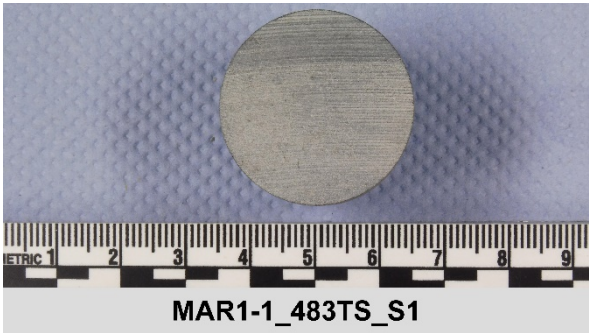
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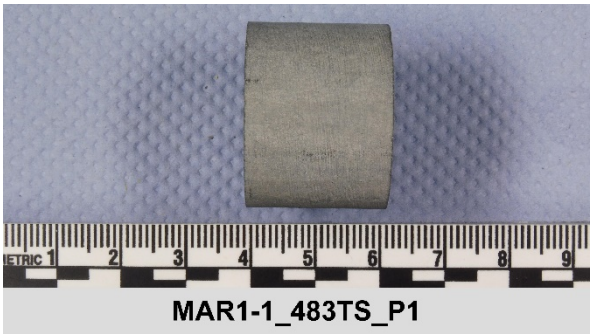
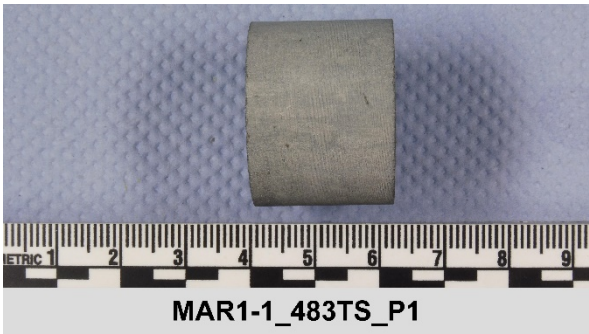
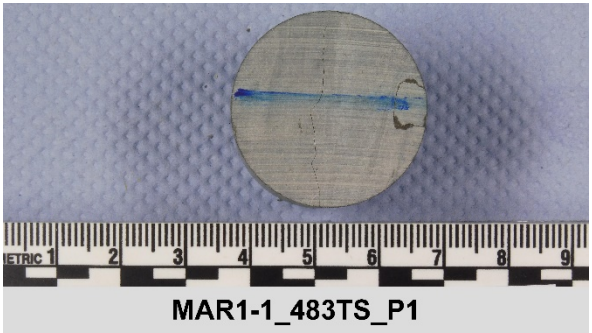
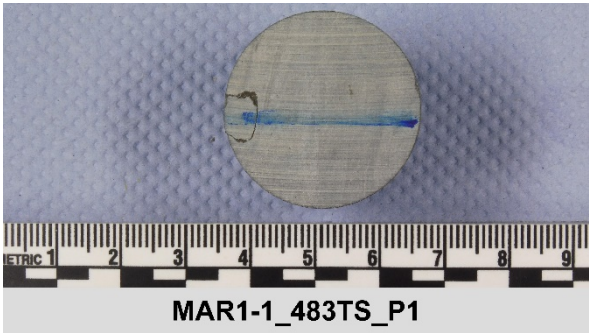
before testing

after testing

484.24 m



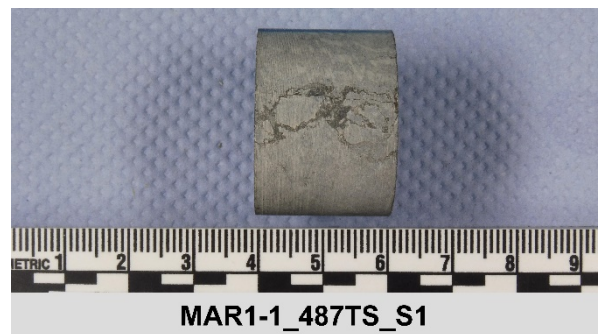
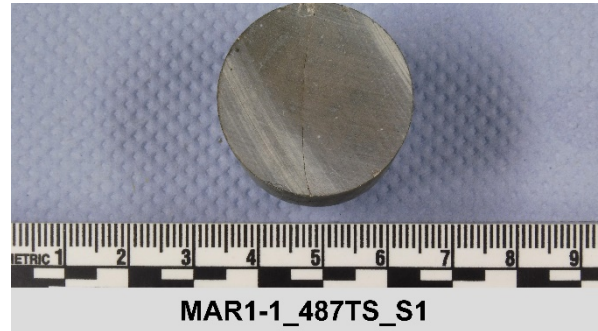
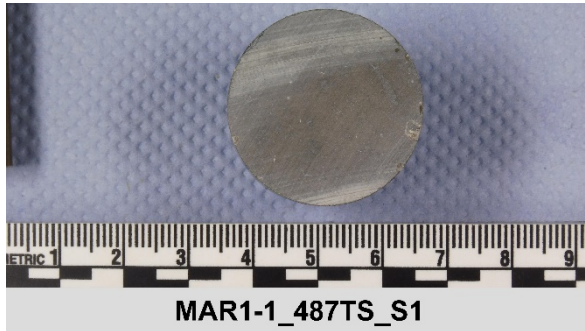
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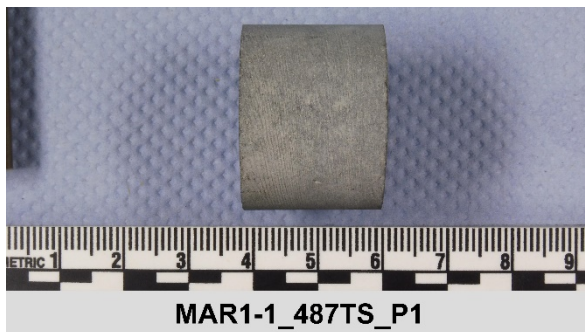
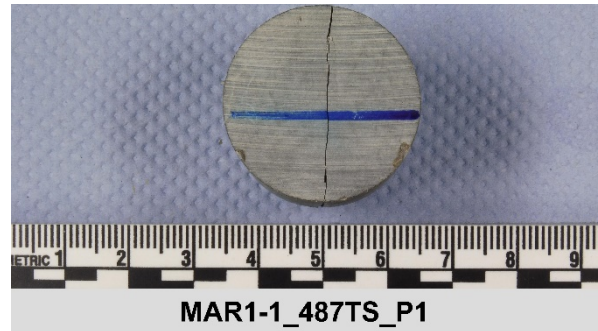
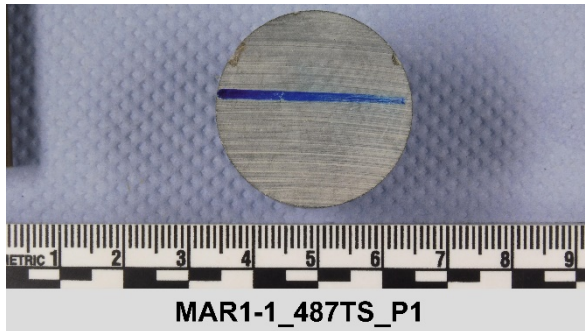
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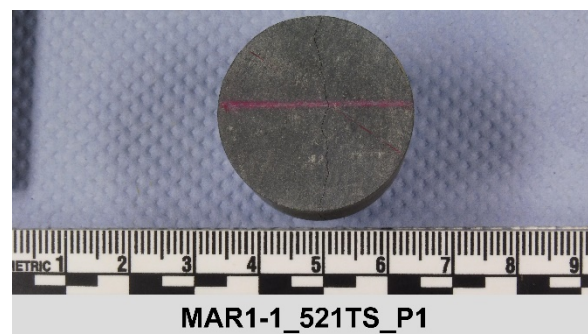
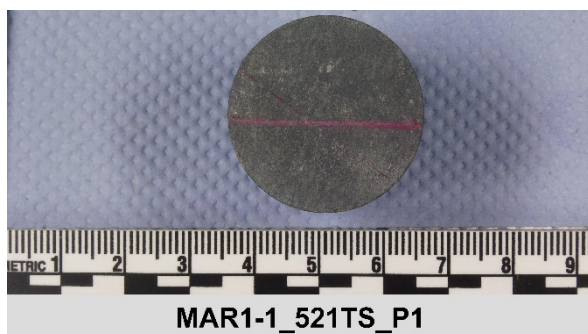


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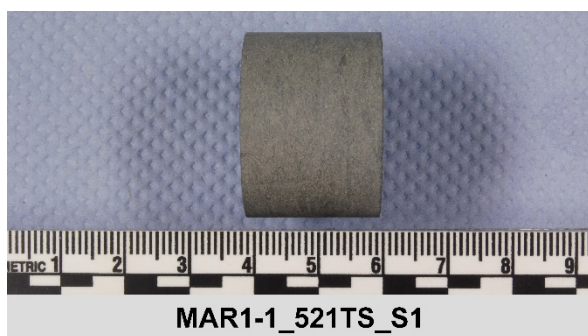
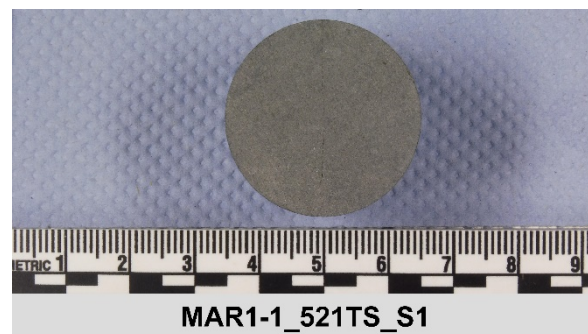
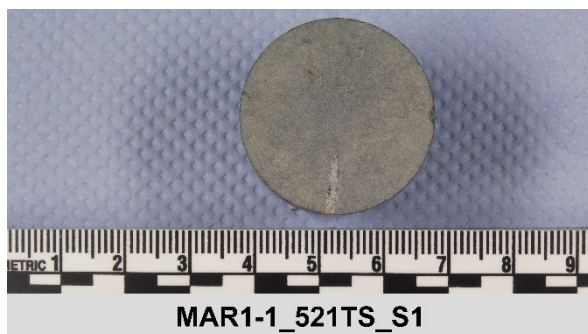


before testing**after testing**

521.80 m



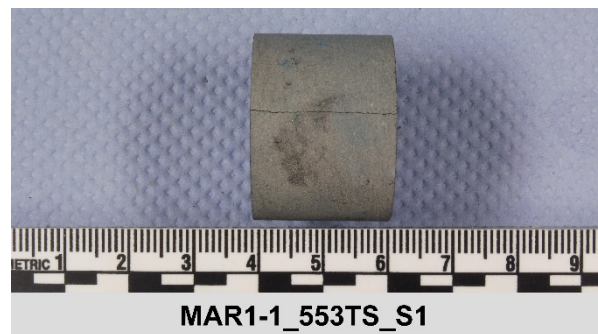
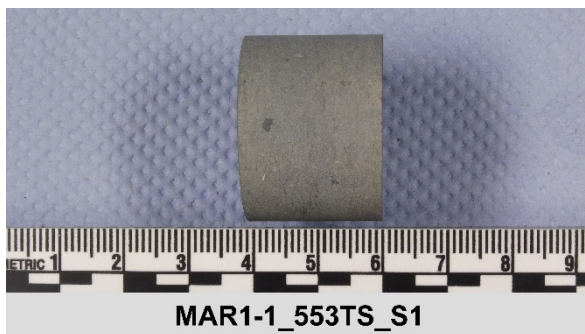
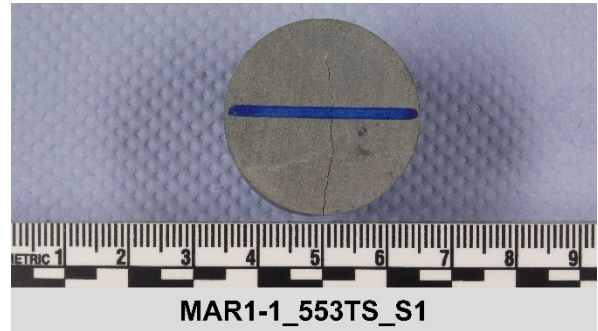
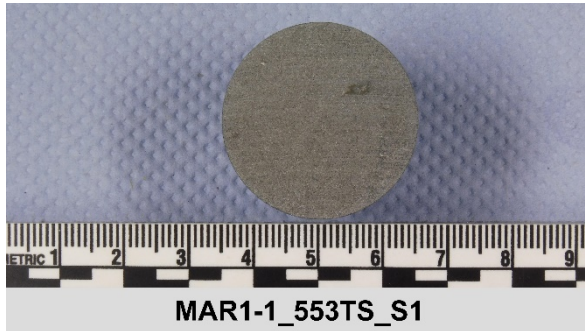
521.83 m



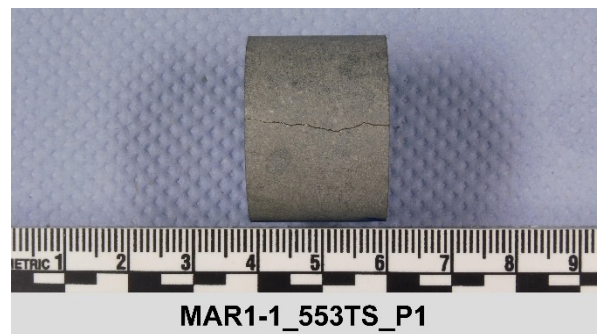
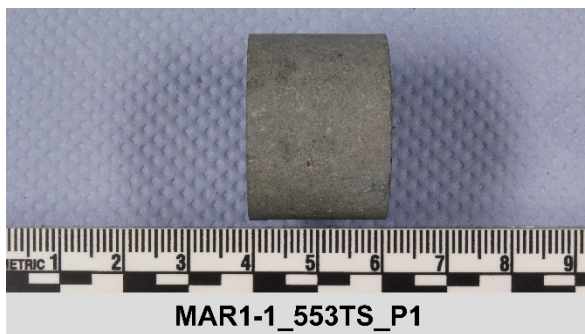
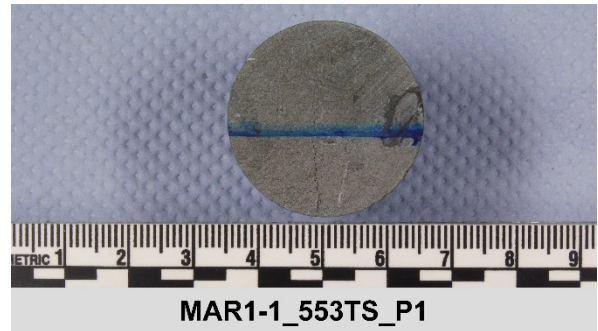
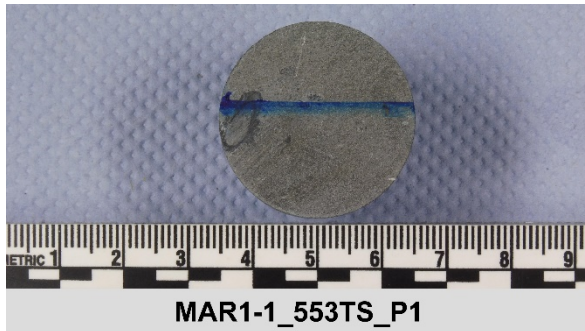
before testing

after testing

553.21 m

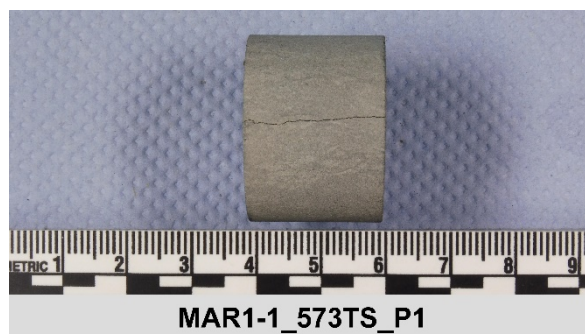
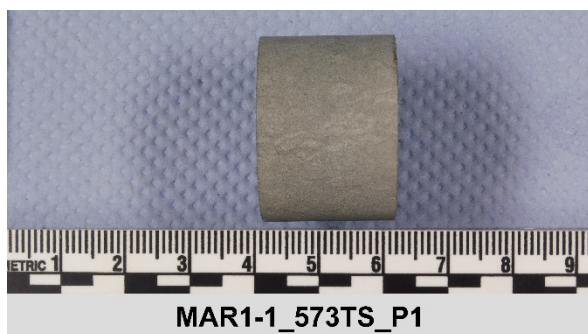
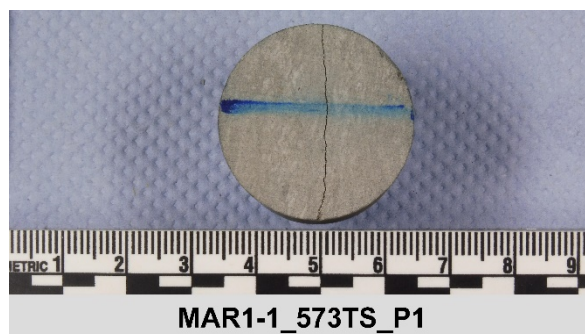
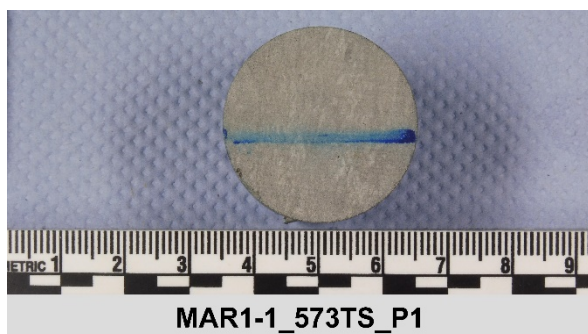


553.26 m

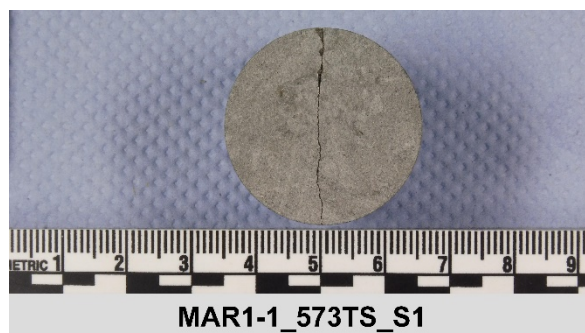


before testing**after testing**

573.31 m



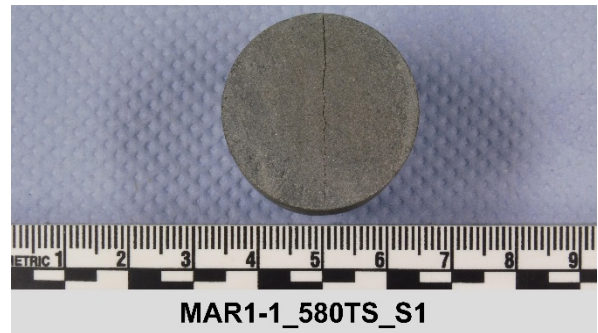
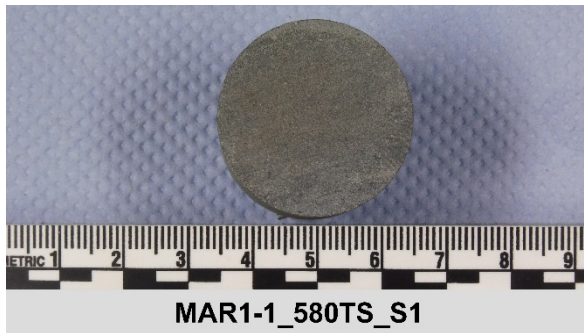
573.35 m



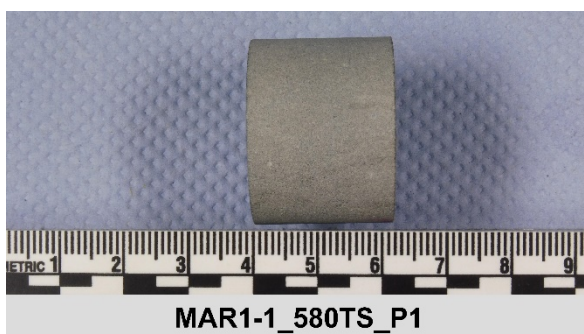
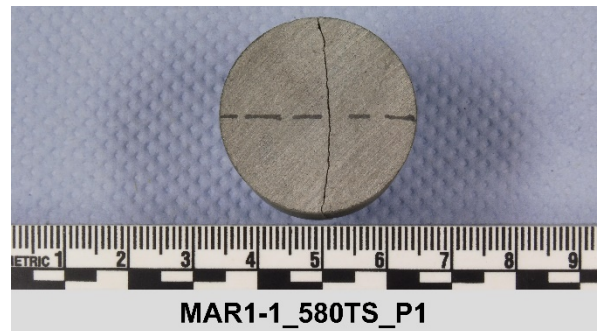
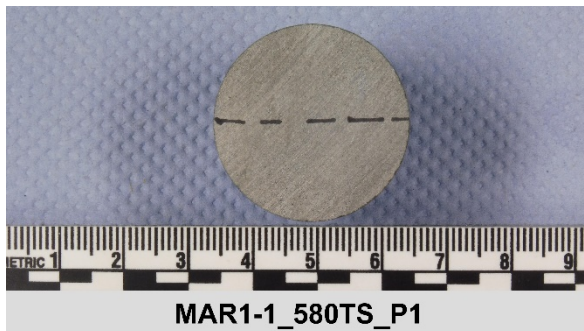
before testing

after testing

580.33 m



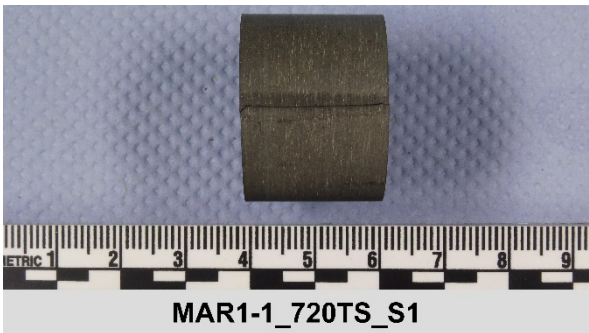
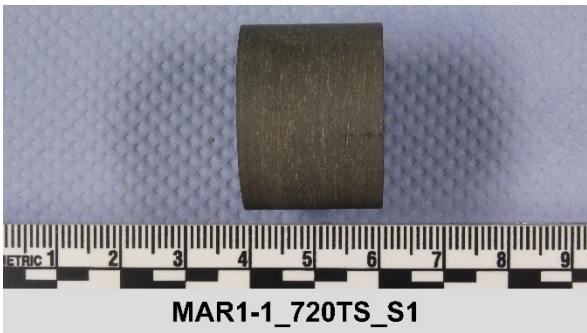
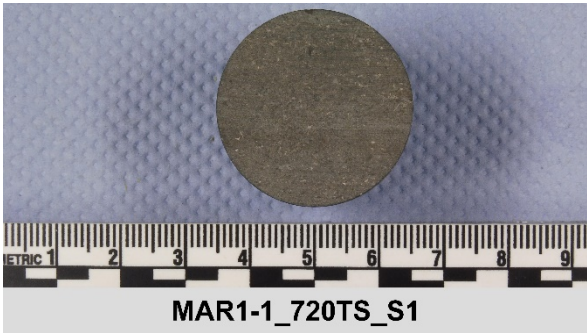
580.36 m



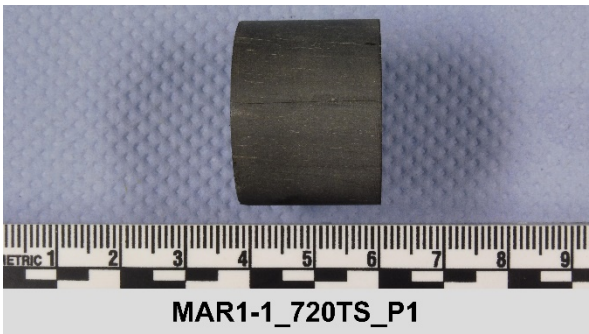
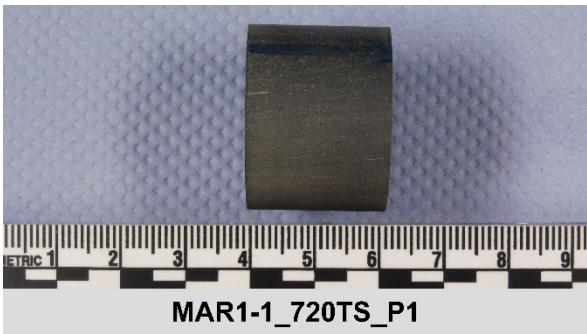
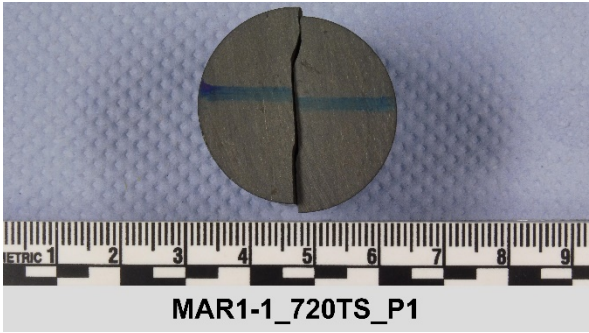
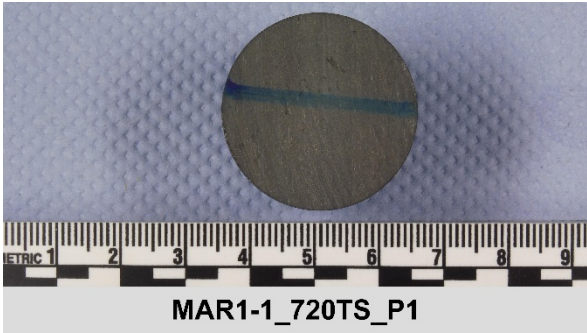
before testing

after testing

720.56 m



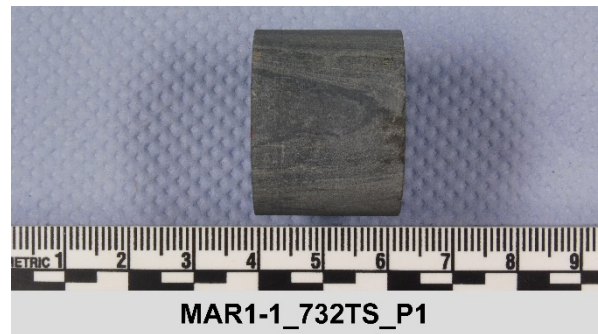
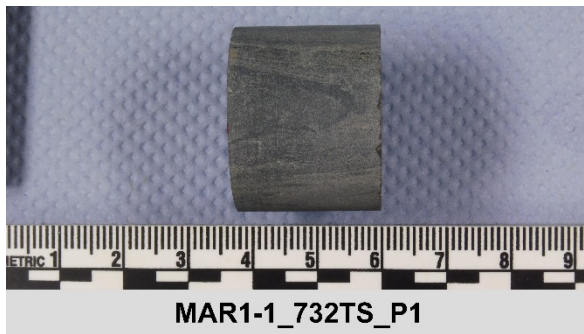
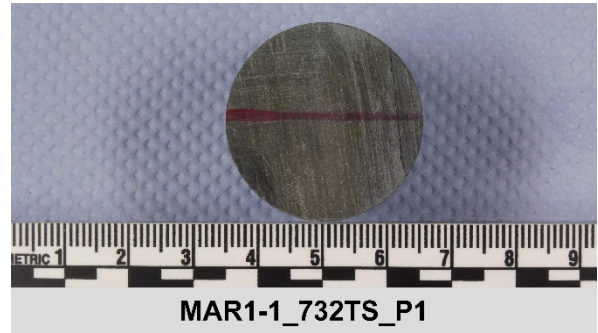
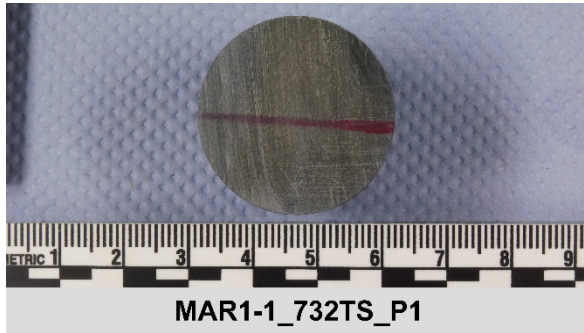
720.60 m



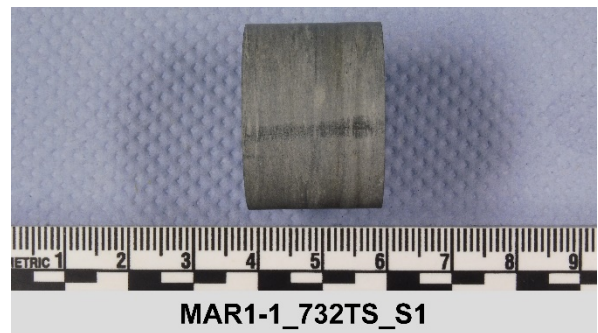
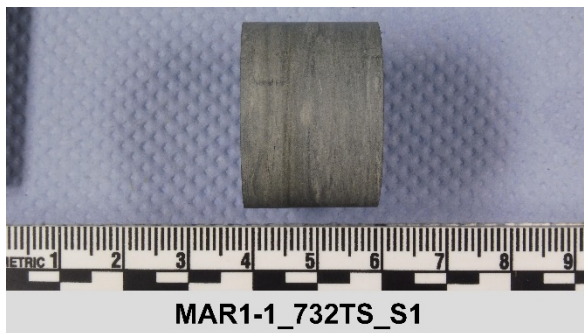
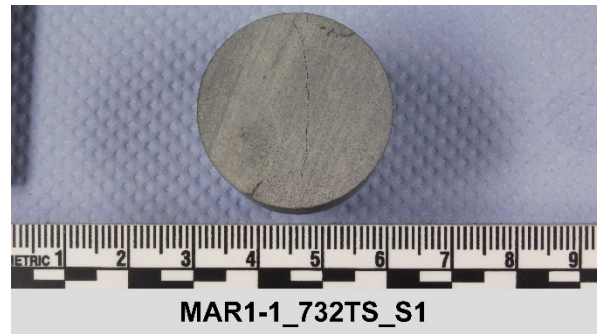
before testing

after testing

732.46 m

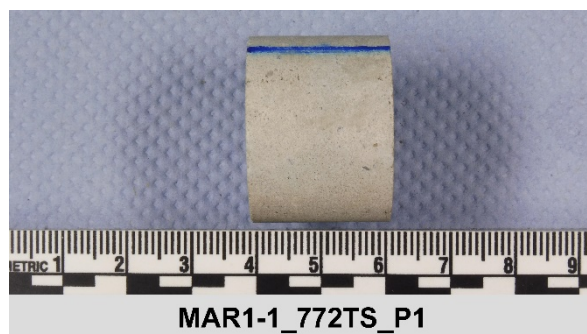
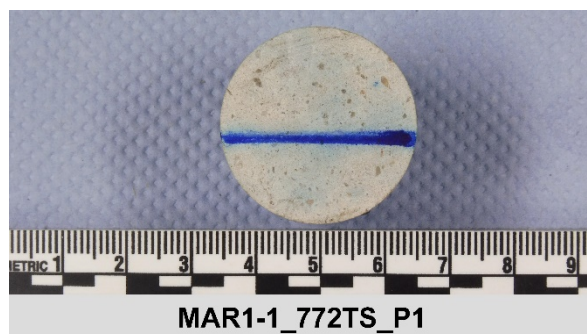
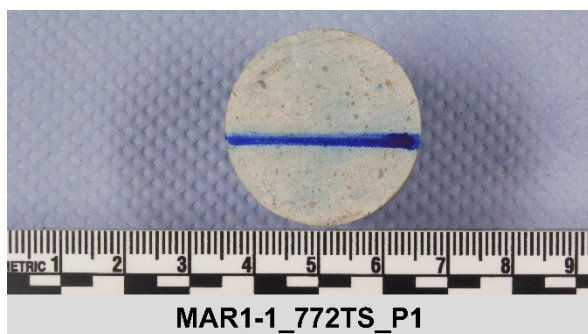


732.50 m



before testing**after testing**

772.40 m



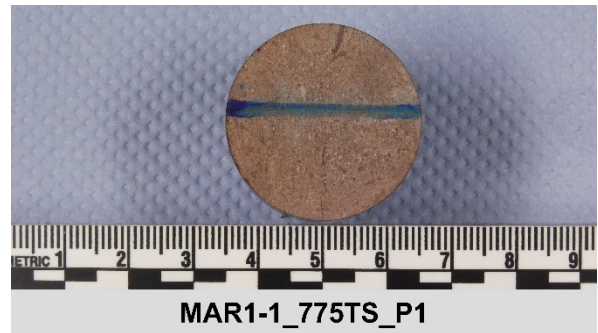
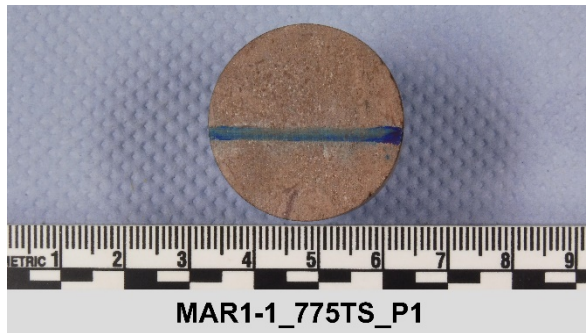
772.45 m



before testing

after testing

775.74 m



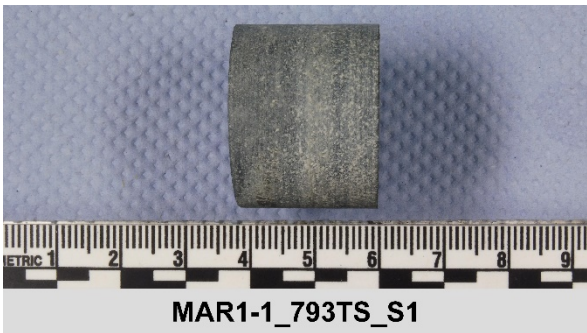
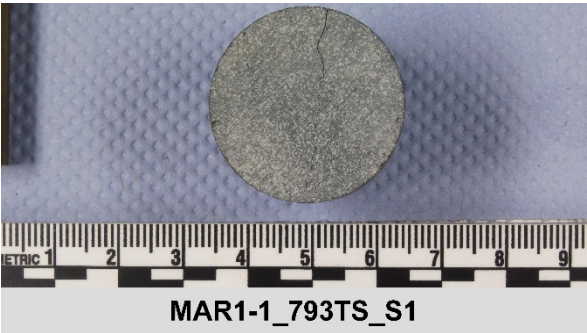
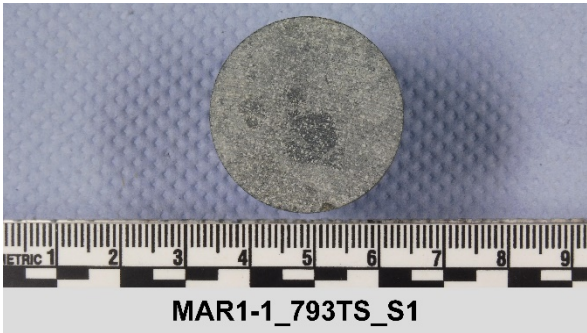
775.78 m



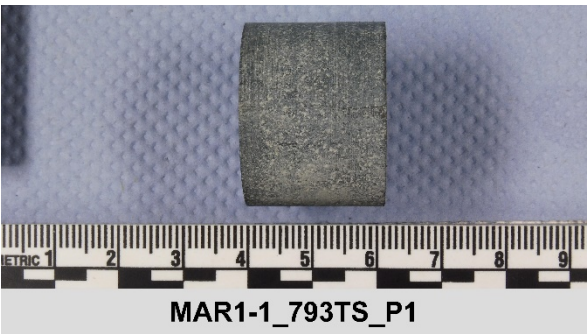
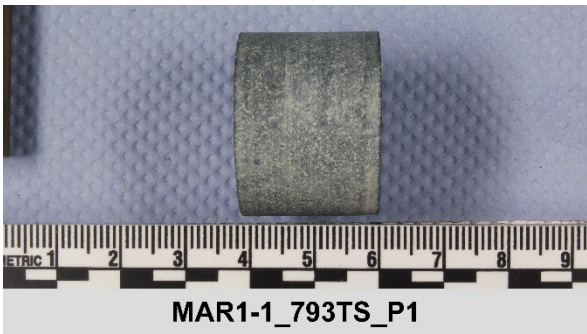
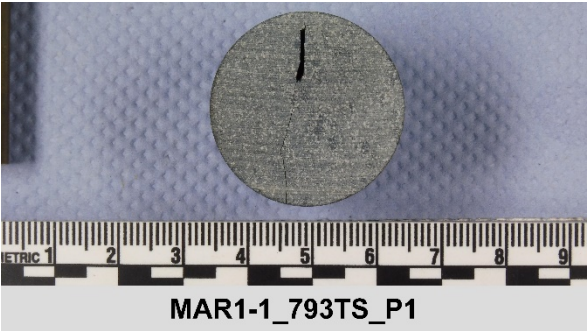
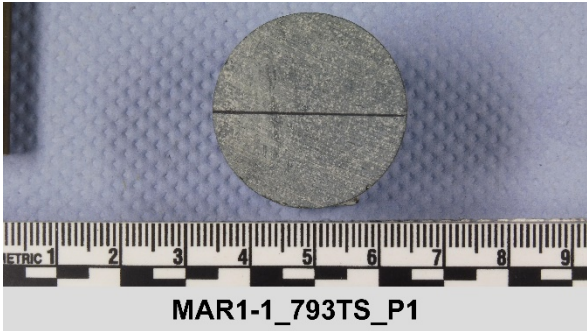
before testing

after testing

793.77 m



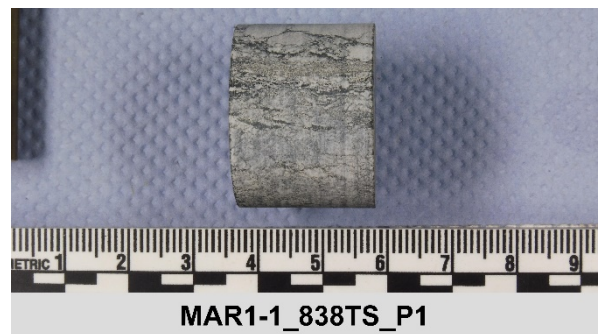
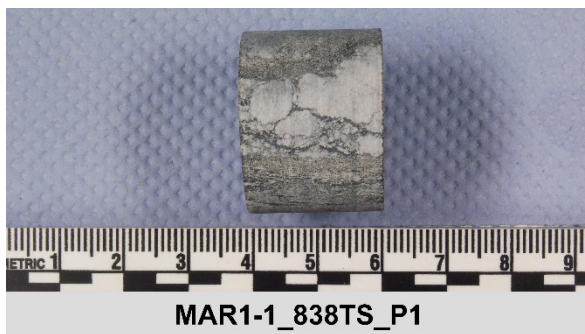
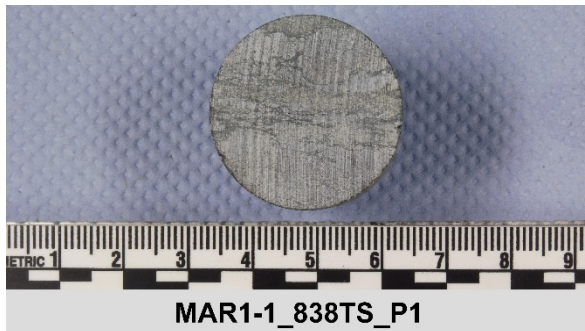
793.81 m



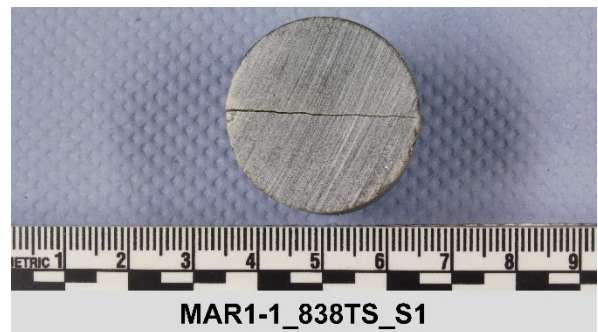
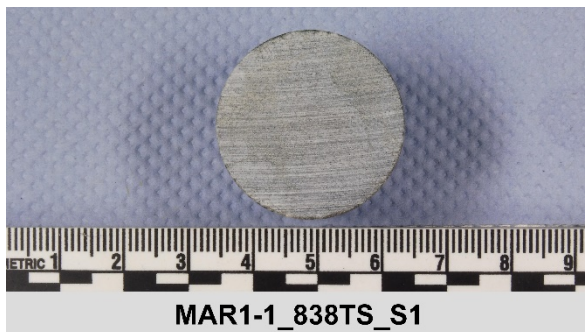
before testing

after testing

839.12 m



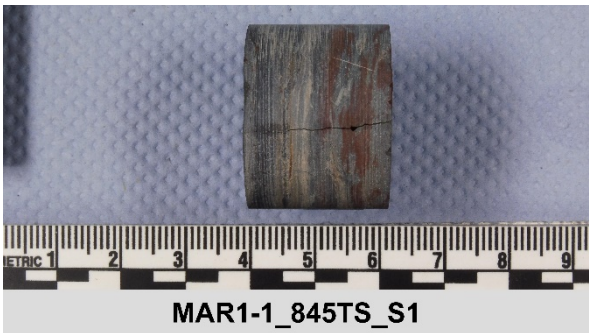
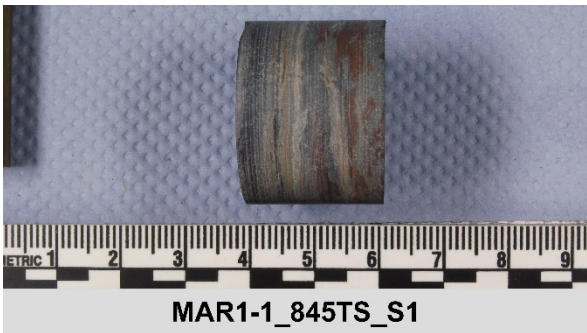
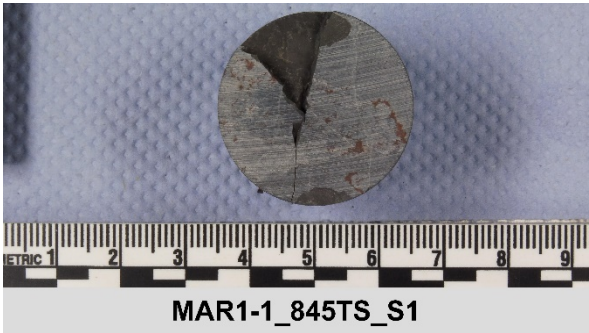
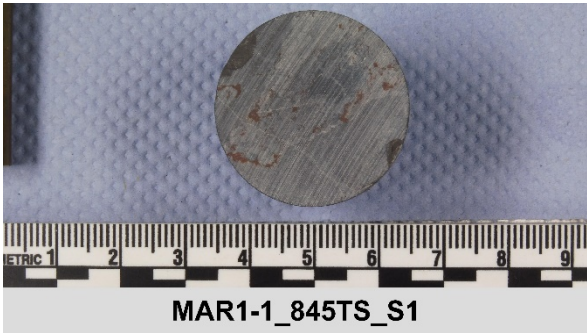
839.16 m



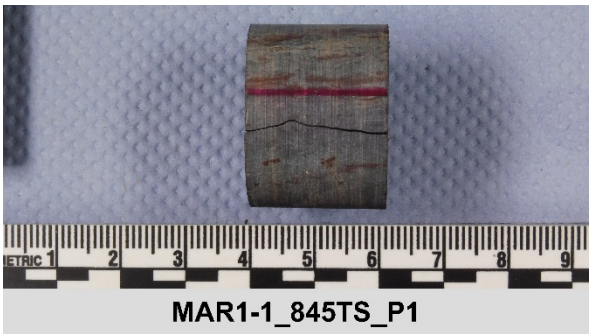
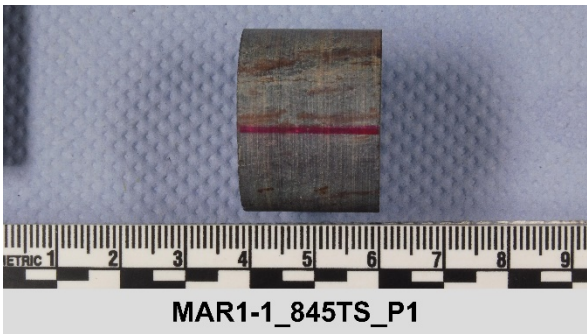
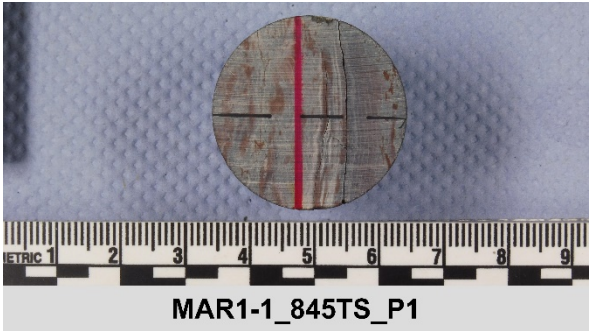
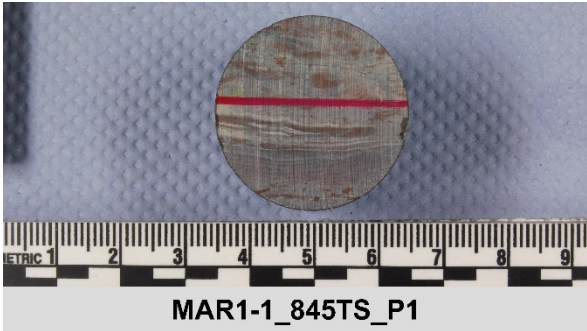
before testing

after testing

845.48 m



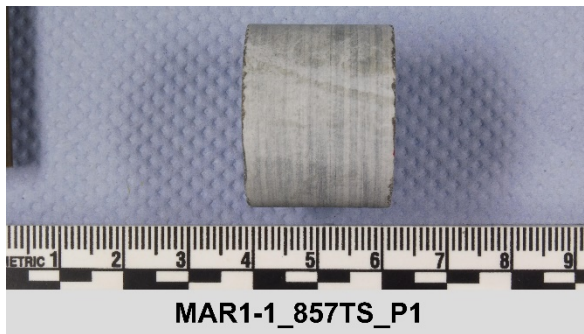
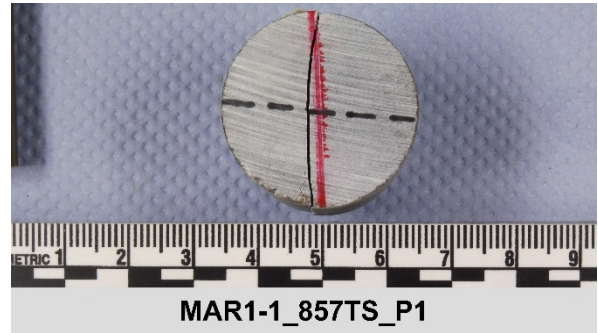
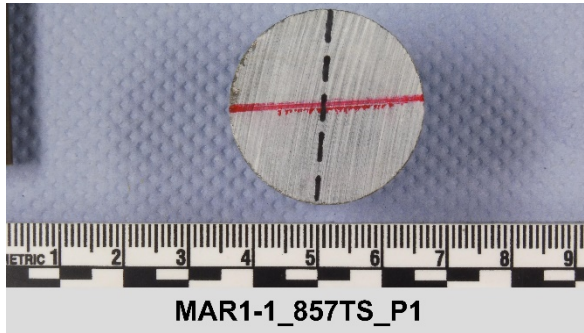
845.52 m



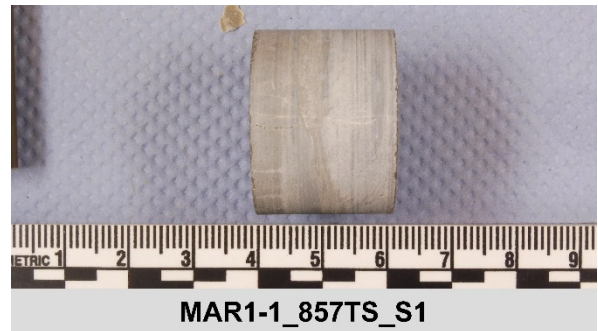
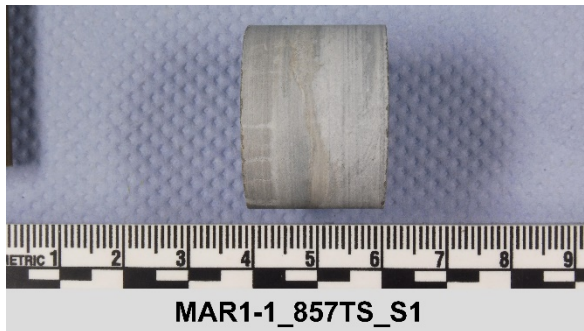
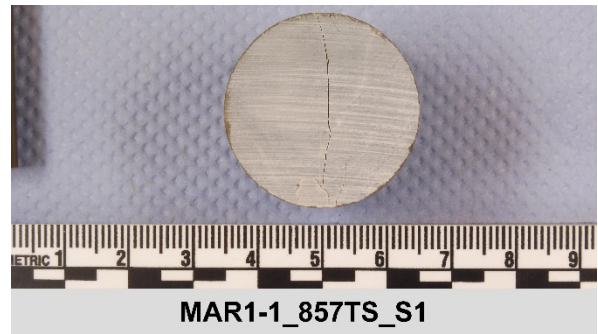
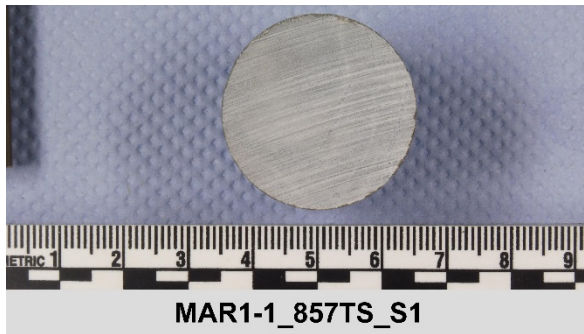
before testing

after testing

857.91 m



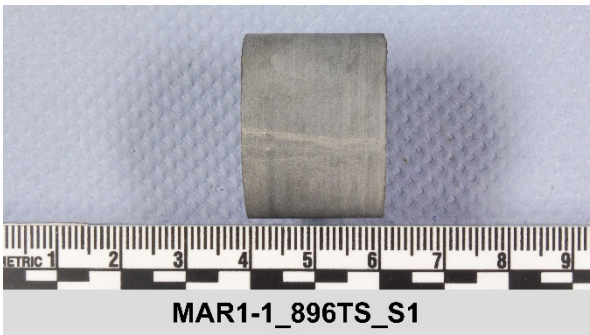
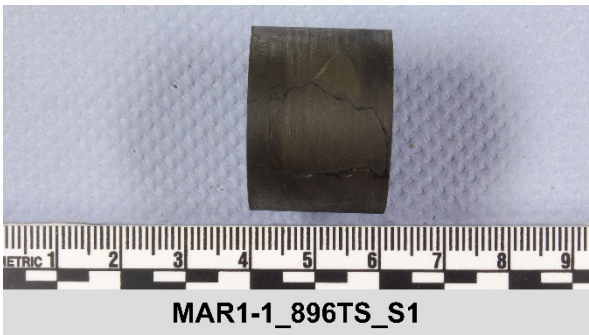
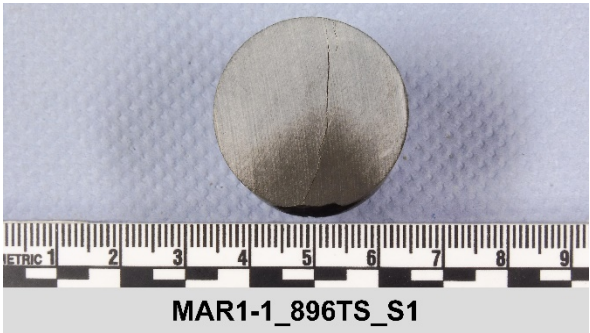
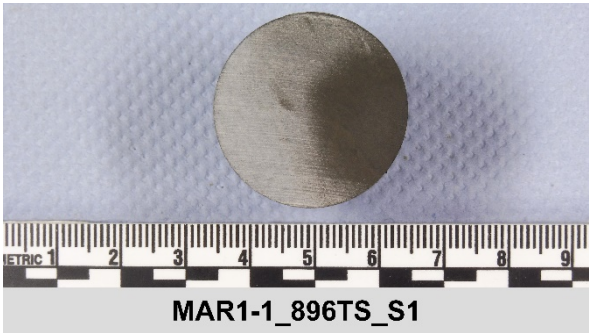
857.95 m



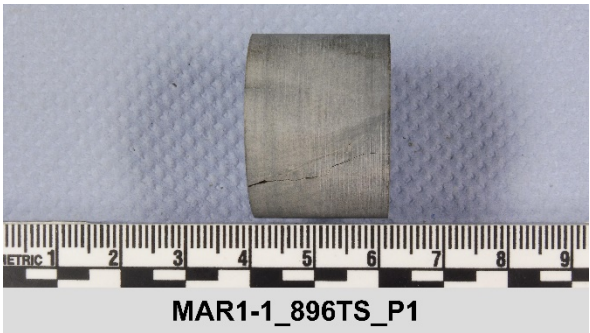
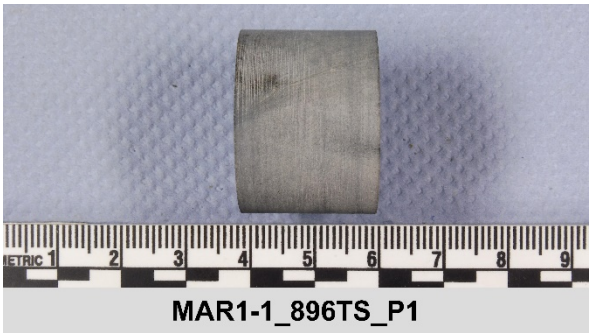
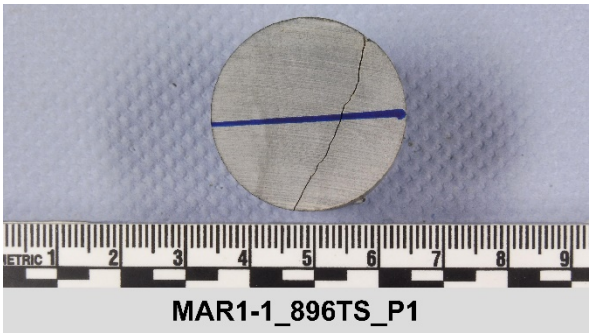
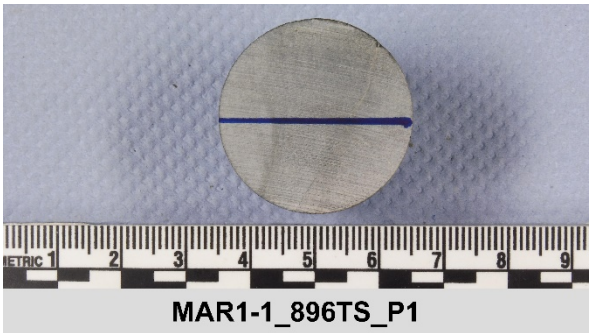
before testing

after testing

897.04 m



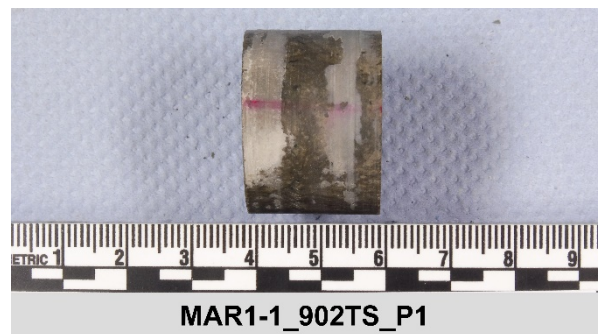
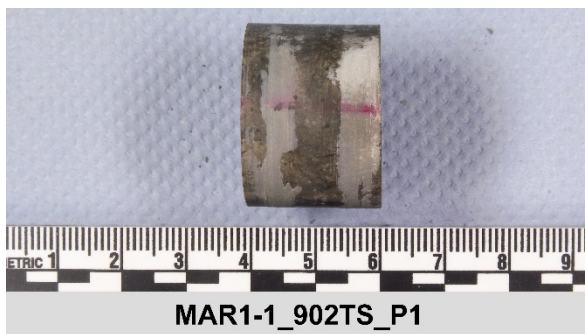
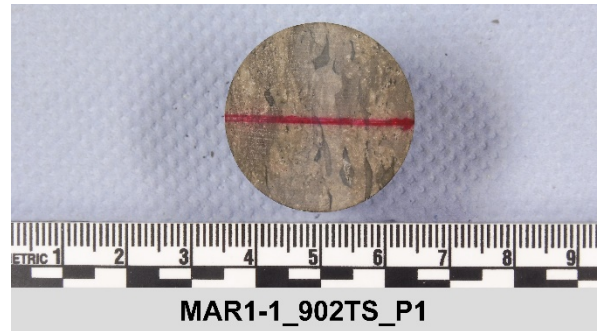
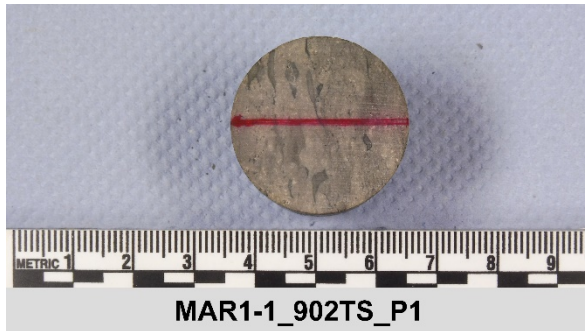
897.08 m



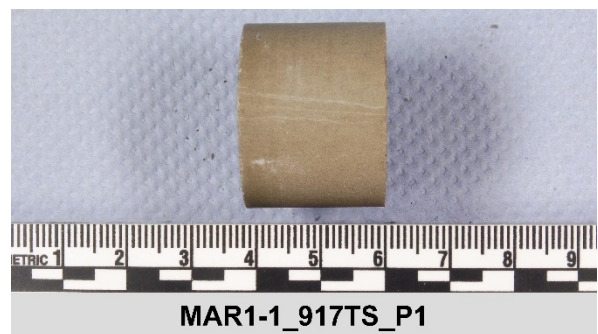
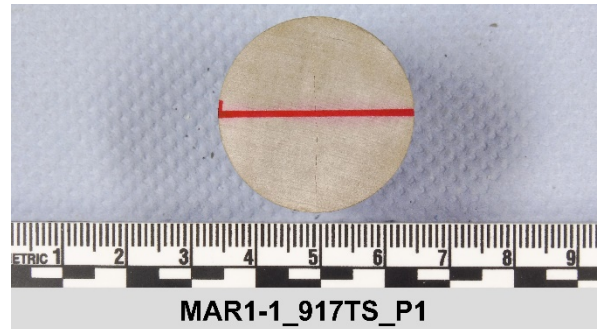
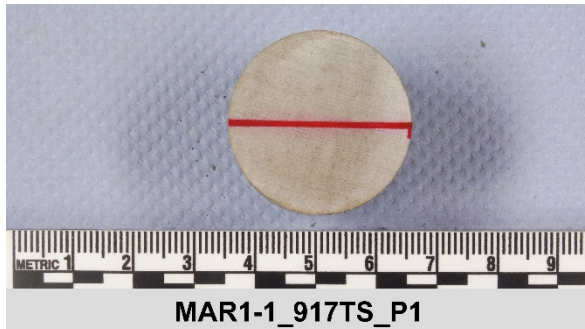
before testing

after testing

902.86 m

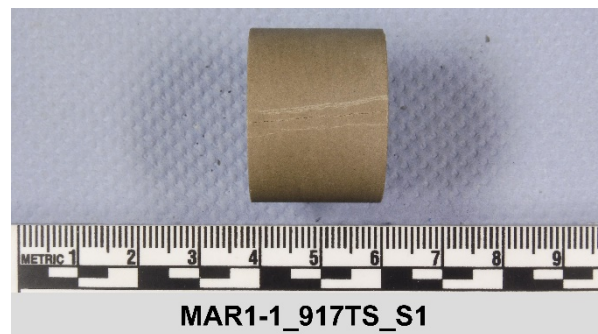
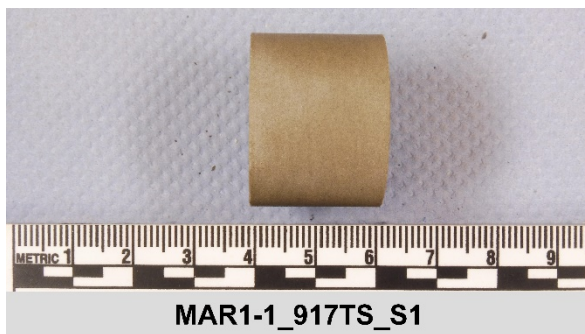
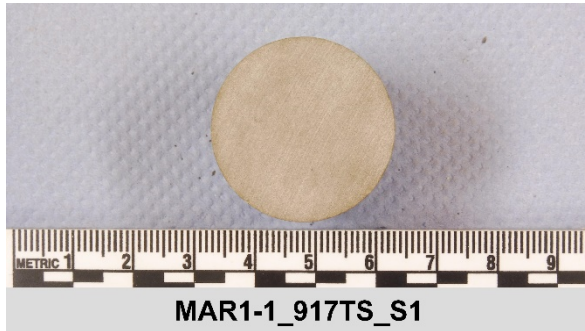


1093.27



before testing**after testing**

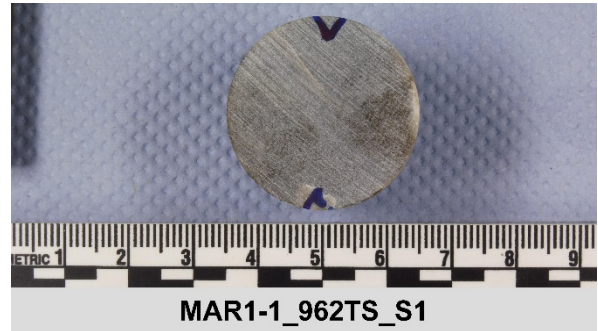
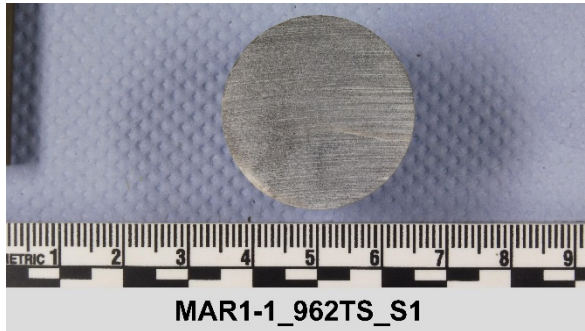
917.72 m



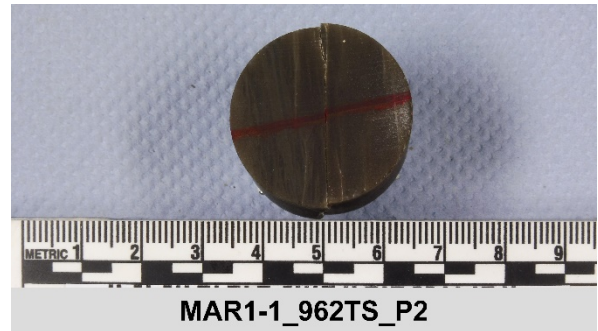
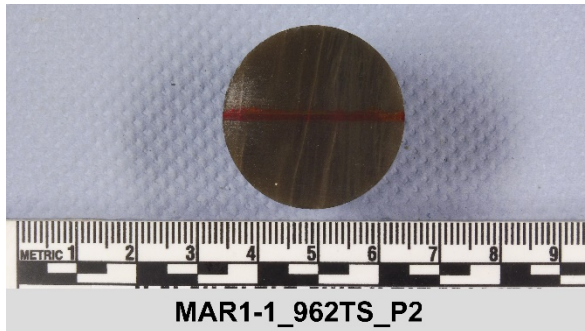
before testing

after testing

962.74 m



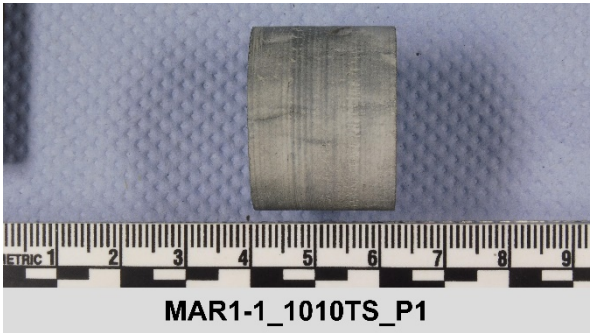
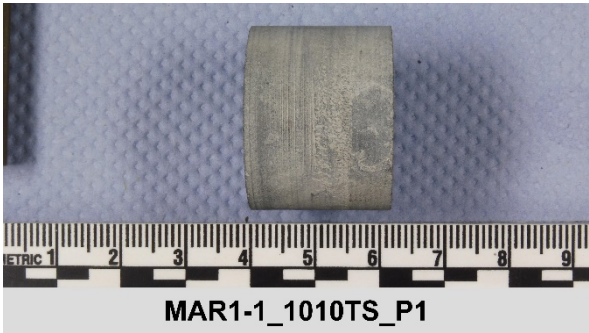
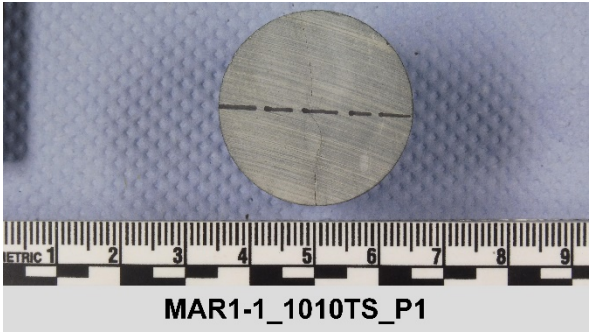
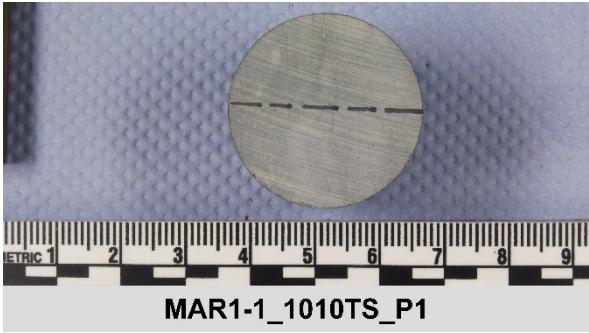
962.77 m



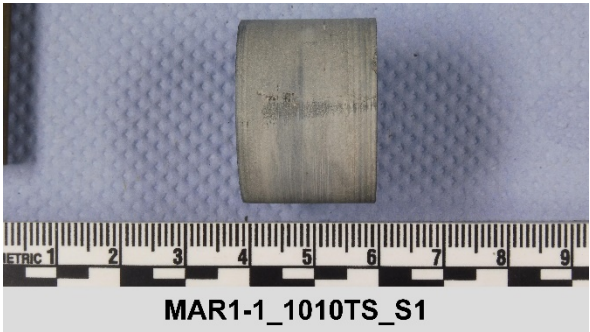
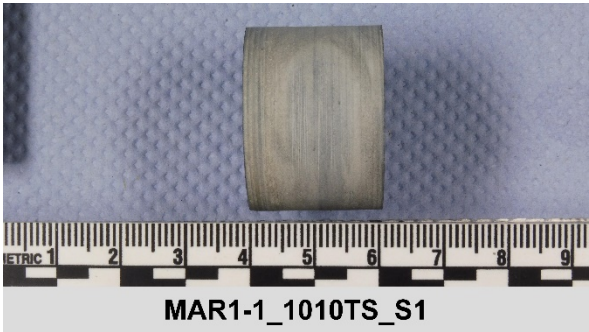
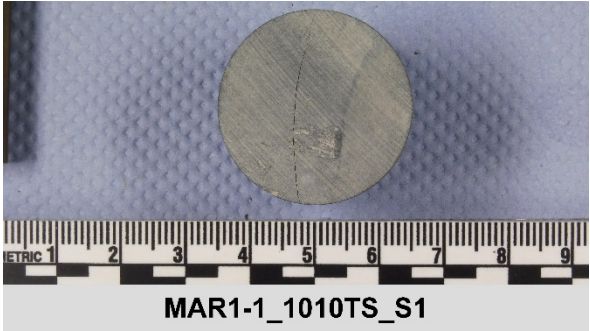
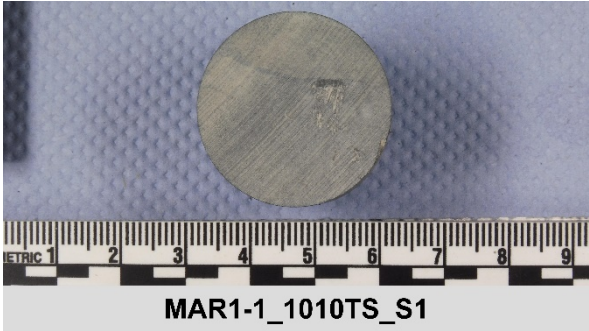
before testing

after testing

1010.68 m



1010.72 m



UCS (depth of sample top)

MAR1-1

before testing

after testing

472.96 m



473.00 m



484.16 m



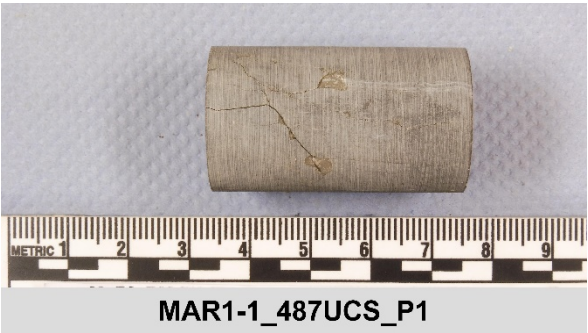
484.15 m



before testing

after testing

487.23 m



487.44 m



521.68 m



521.74 m



before testing

after testing

553.13 m



553.17 m



553.43 m



573.39 m



before testing

after testing

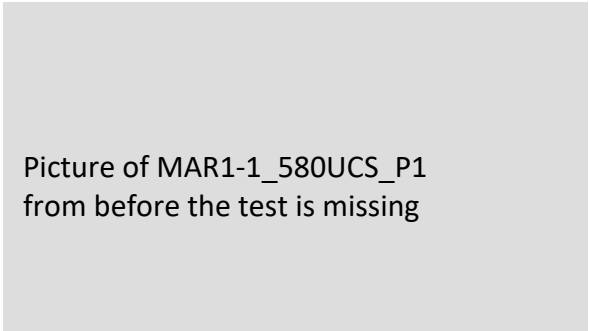
573.44 m



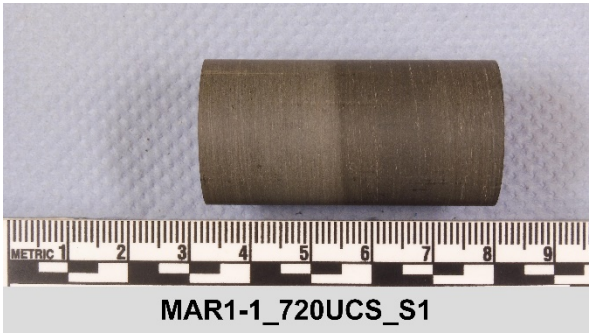
580.27 m



580.27 m



644.66 m



before testing

after testing

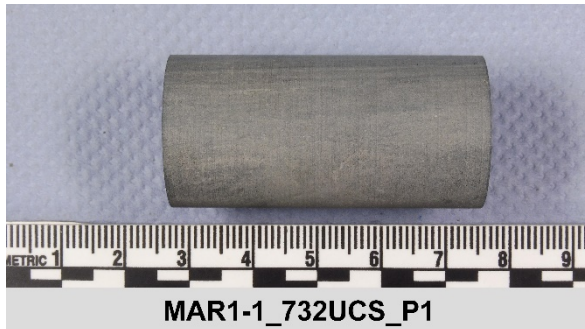
720.51 m



732.28 m



732.55 m

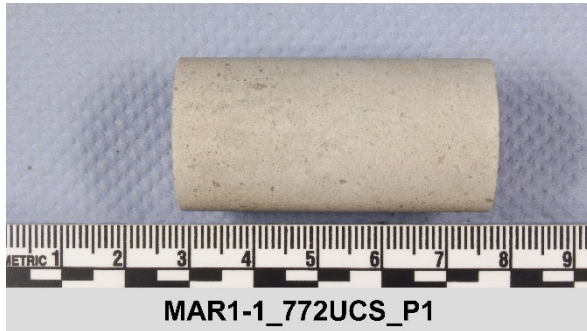


772.50 m



before testing**after testing**

772.50 m



775.82 m



775.88 m



793.89 m



before testing

after testing

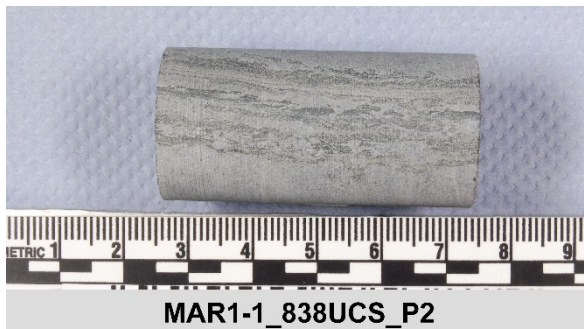
839.02 m



839.22 m

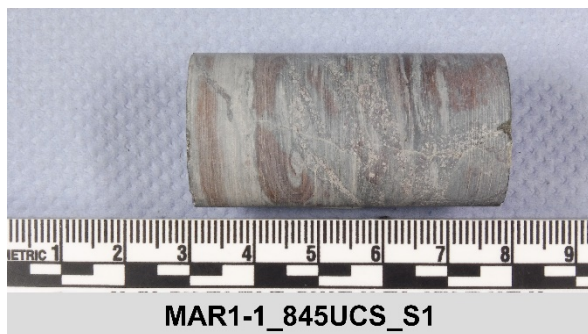


839.22 m



before testing**after testing**

945.39 m



857.81 m



857.81 m



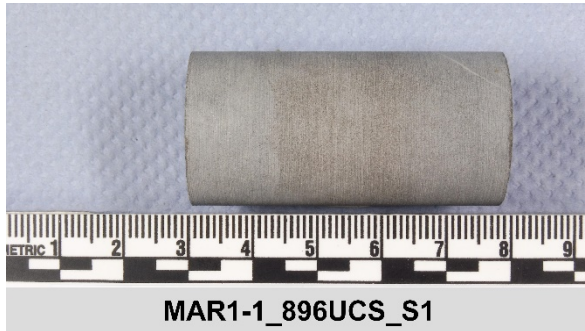
857.81 m



before testing

after testing

896.96 m



902.78 m



917.60 m



917.64 m

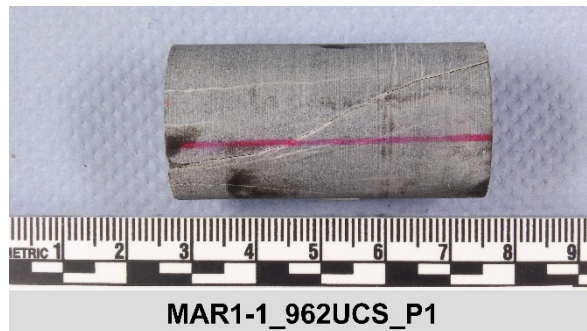


before testing**after testing**

962.62 m



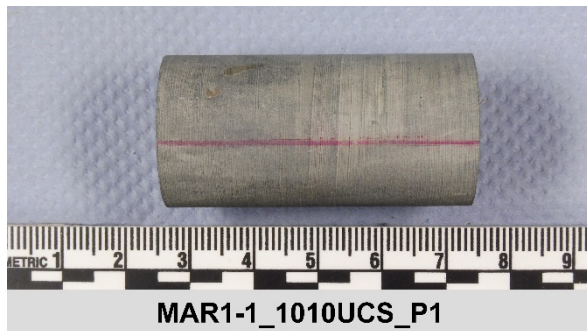
962.67 m



1010.45 m

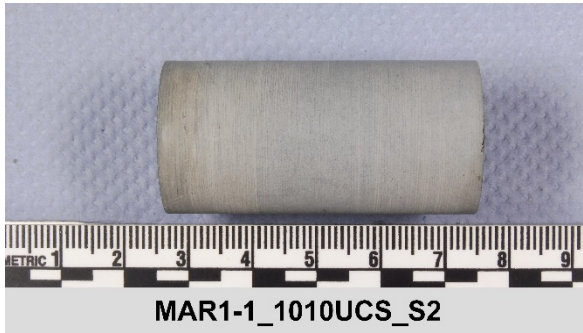


1010.49 m



before testing

1010.77 m



after testing

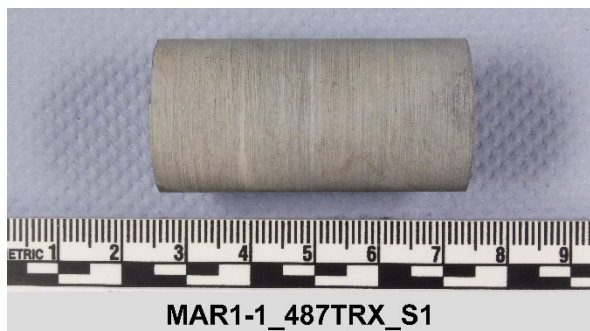


TRX (depth of sample top)**MAR1-1****before testing****after testing**

472.96 m



487.44 m



521.73 m



553.13 m



before testing**after testing**

573.44 m



720.66 m

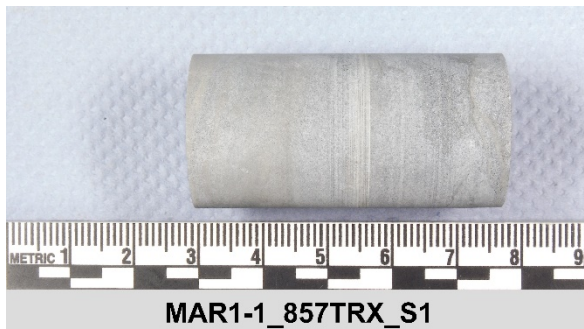


775.88 m

Picture of MAR1-1_755TRX_S1
from before the test is missing



857.87 m

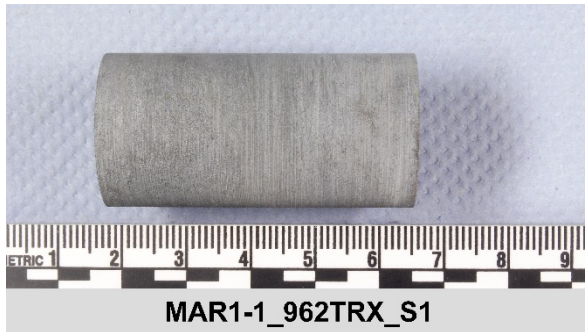


before testing**after testing**

917.60 m

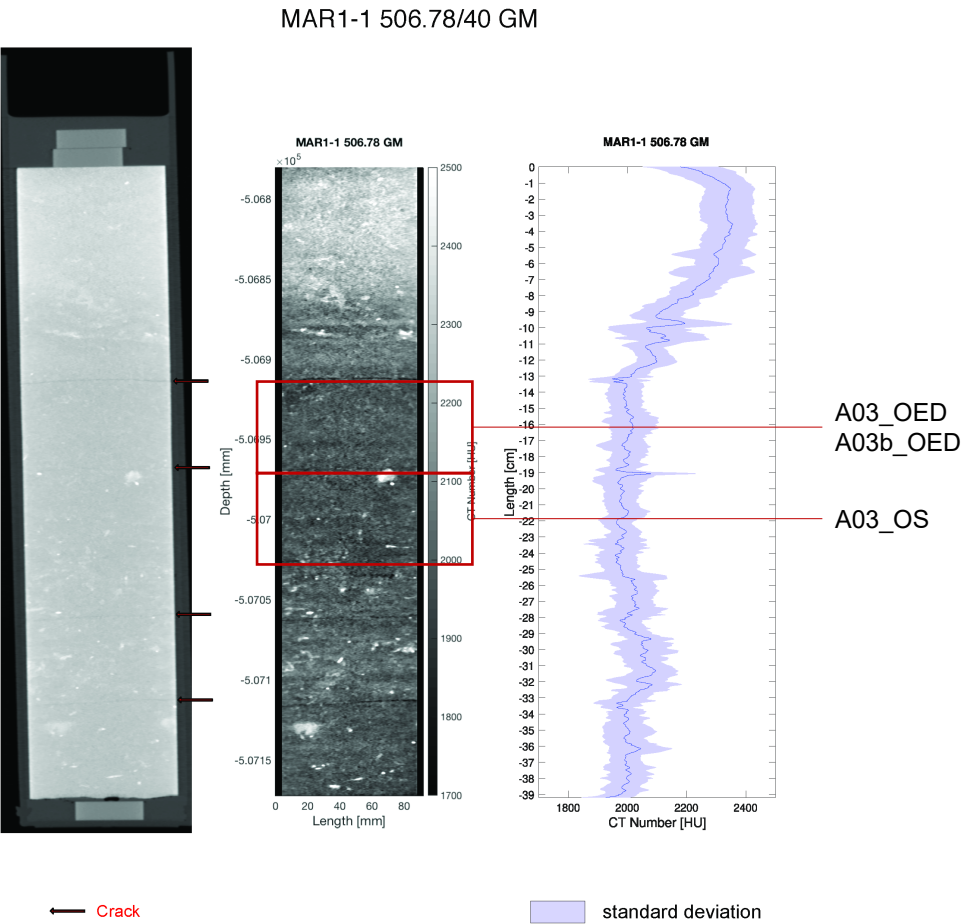


962.62 m

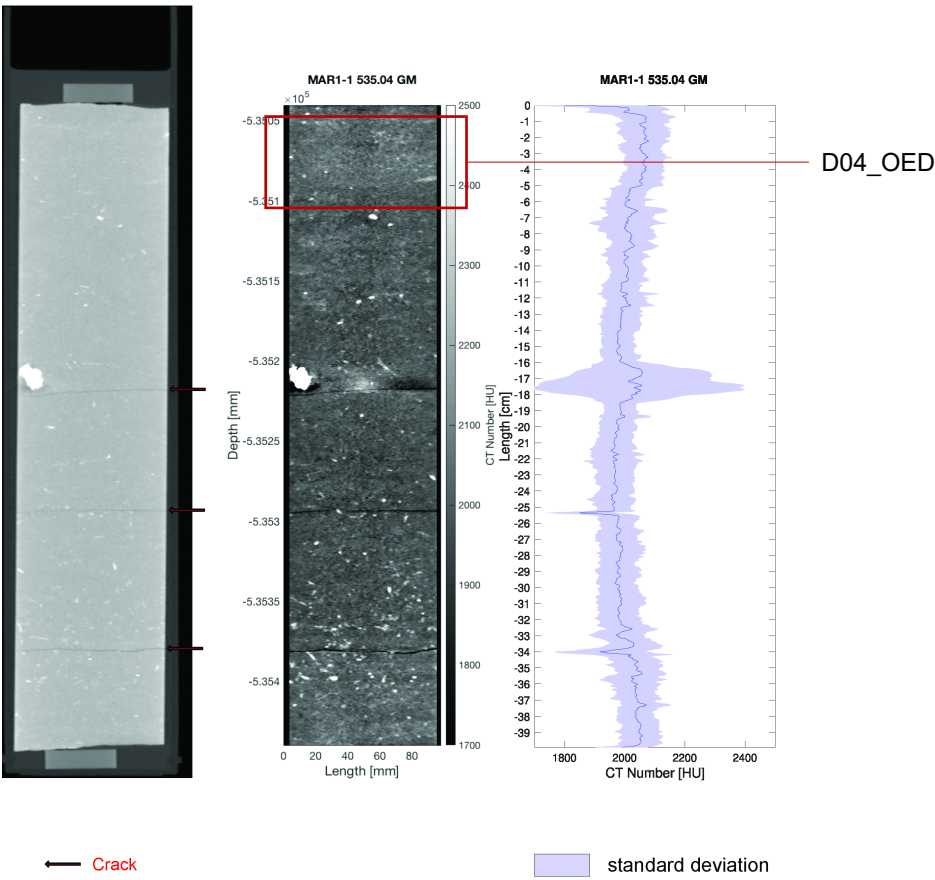


Appendix B: XCT cross-sections with selection of test specimens

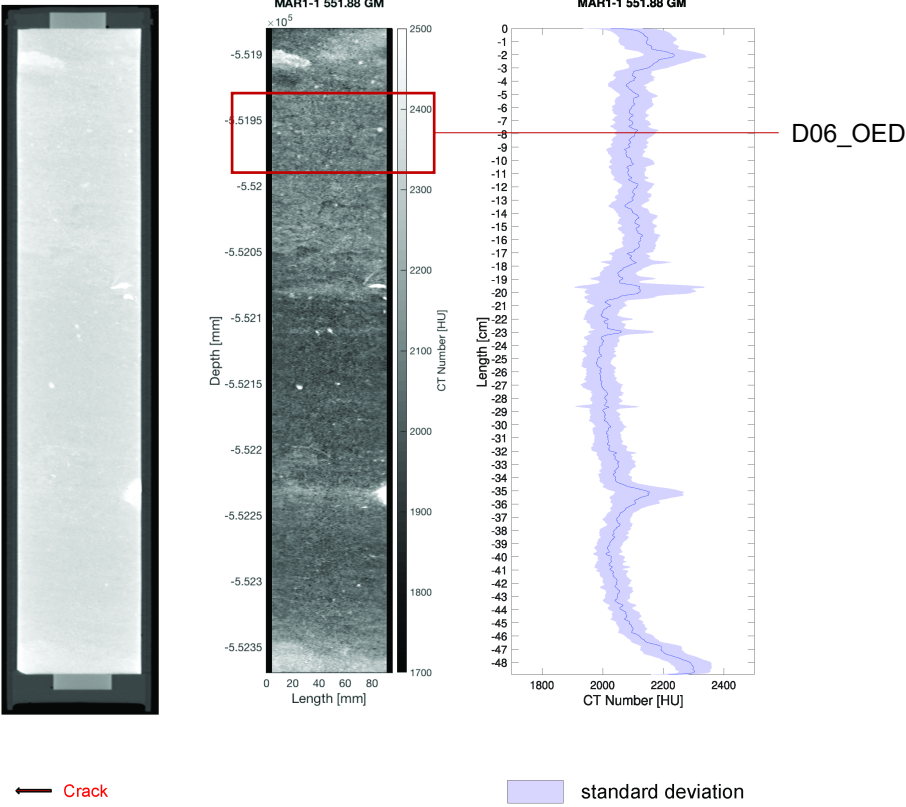
The computed tomography scans of the tested cores are here reported, and the sections used in the testing programme are indicated.



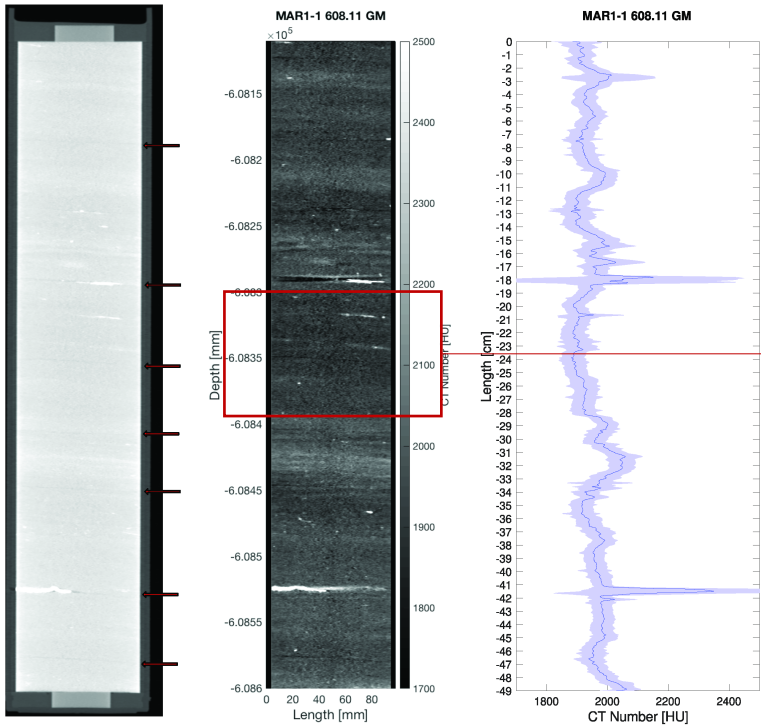
MAR1-1 535.04/40 GM



MAR1-1 551.88/50 GM



MAR1-1 608.11/50 GM

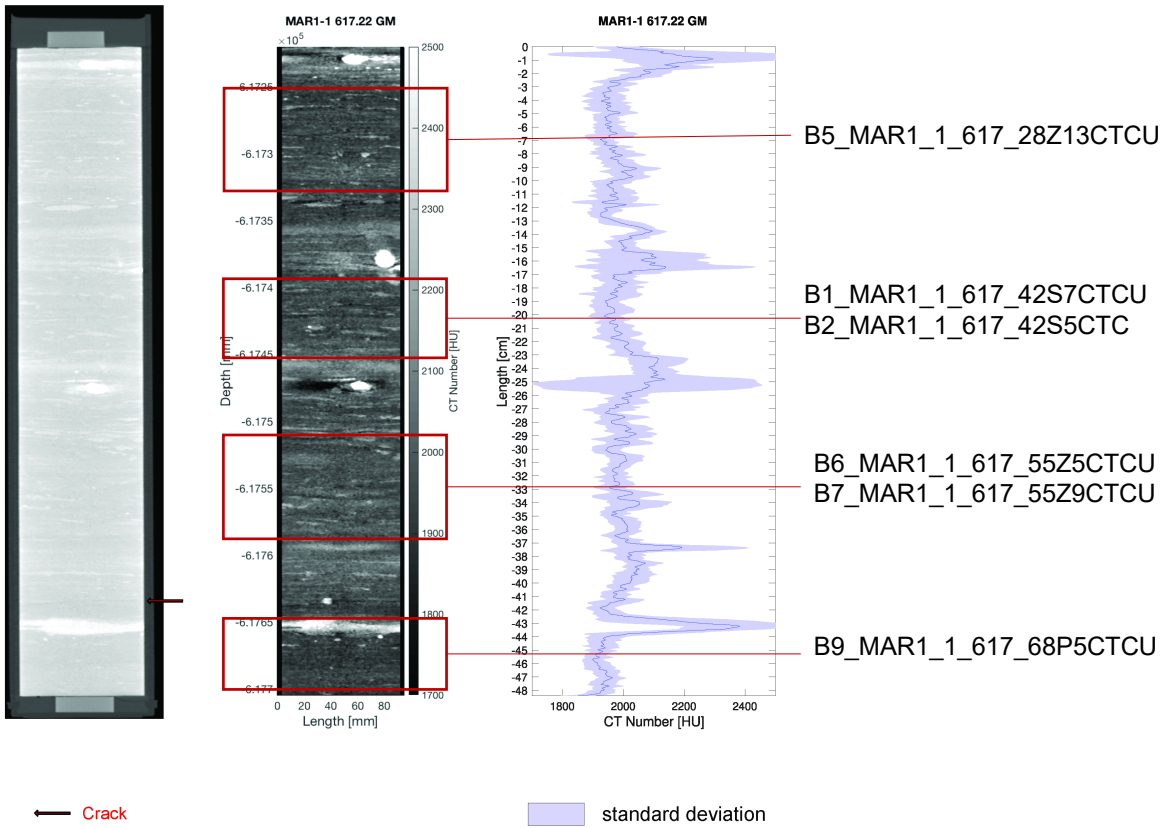


C8_MAR1_1_608_35P6CTCU
C6_MAR1_1_608_34P13CTCU
C5_MAR1_1_608_34S13CTCU
C7_MAR1_1_608_34S7CTCU

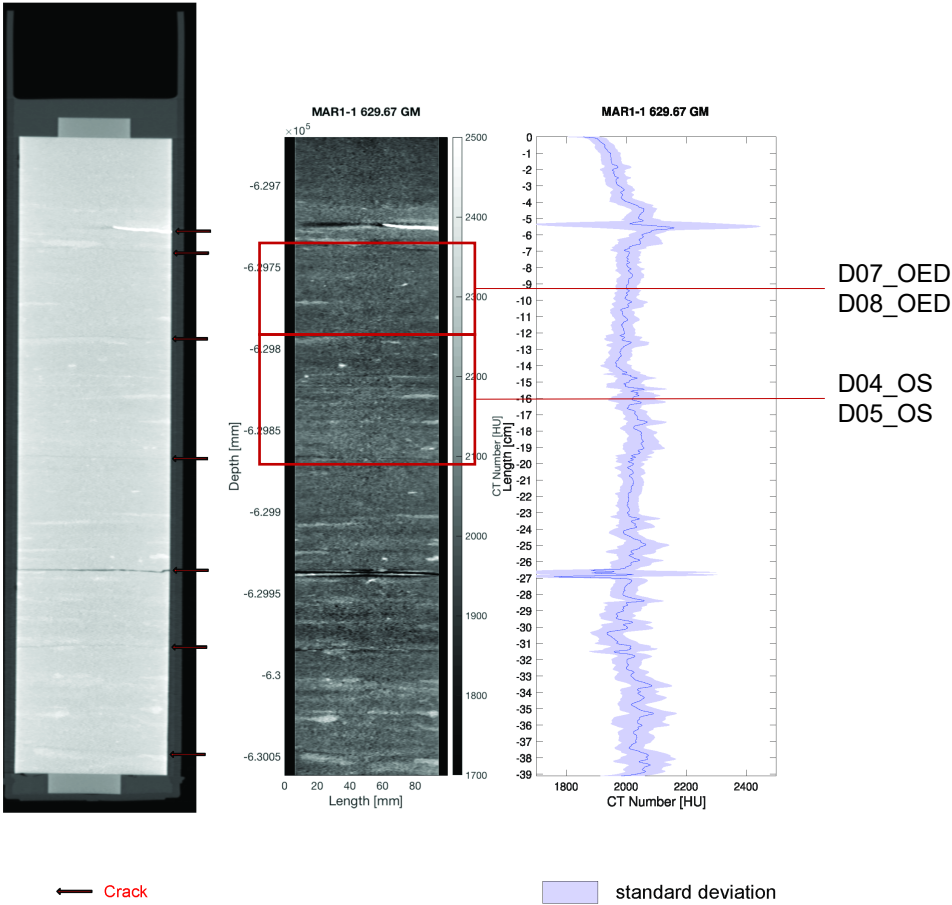
← Crack

standard deviation

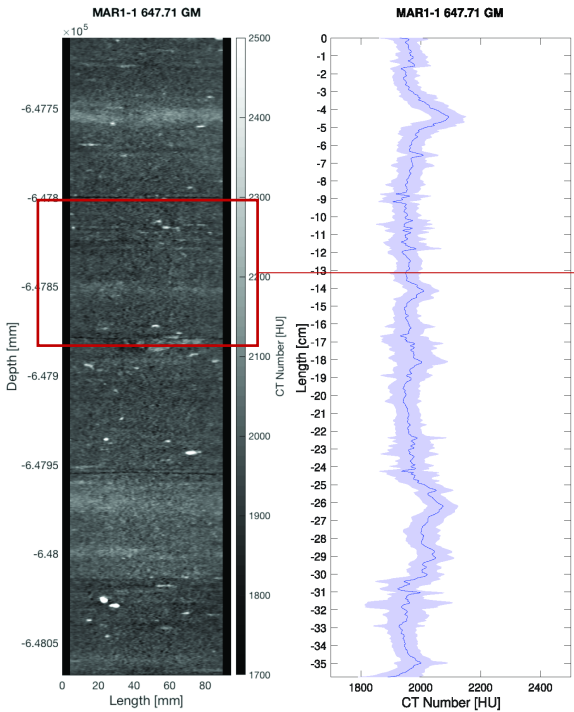
MAR1-1 617.22/50 GM



MAR1-1 629.67/40 GM



MAR1-1 647.71/36 GM

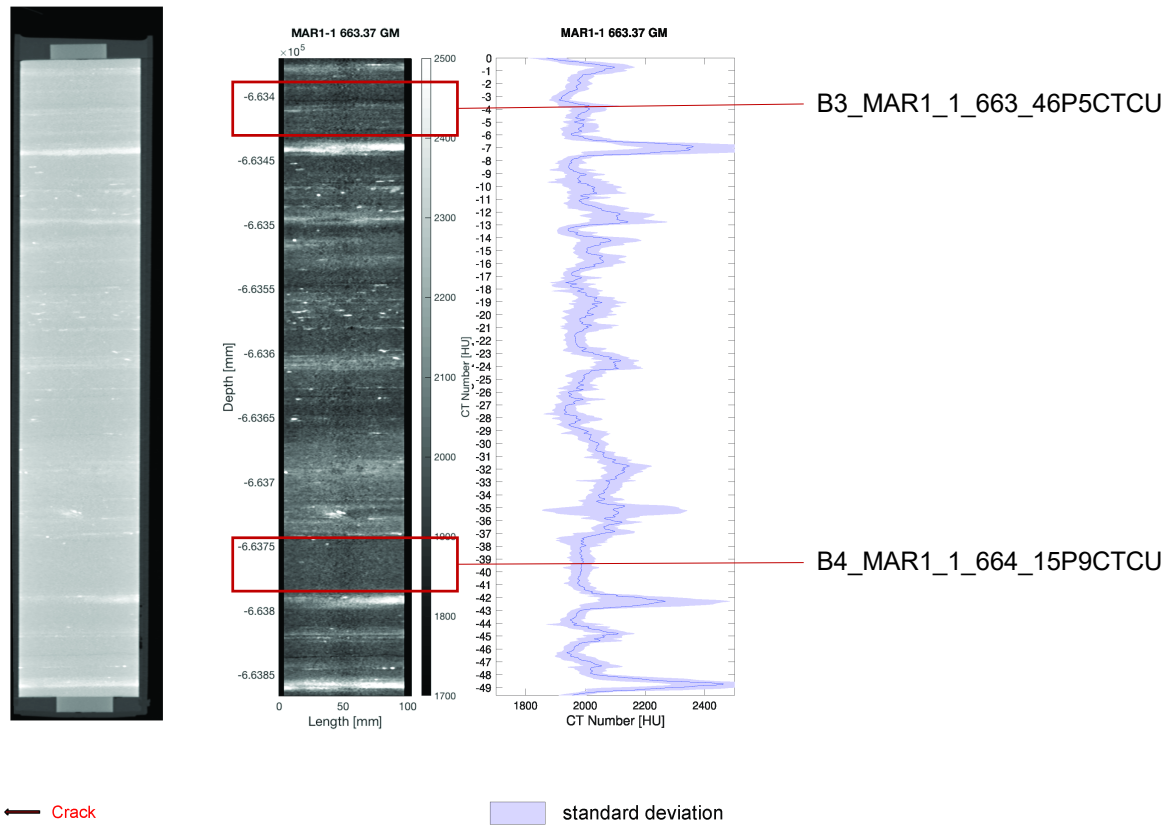


- C1_MAR1_1647_84S13CTCU
- C2_MAR1_1_647_84P13CTCU
- C3_MAR1_1_647_84S6CTCU
- C4_MAR1_1_647_84P6CTCU
- C9_MAR1_1_647_84Z6CTCU
- C10_MAR1_1_647_84Z13CTCU
- C11_MAR1_1_647_84S5CTC
- C12_MAR1_1_647_84P5CTC

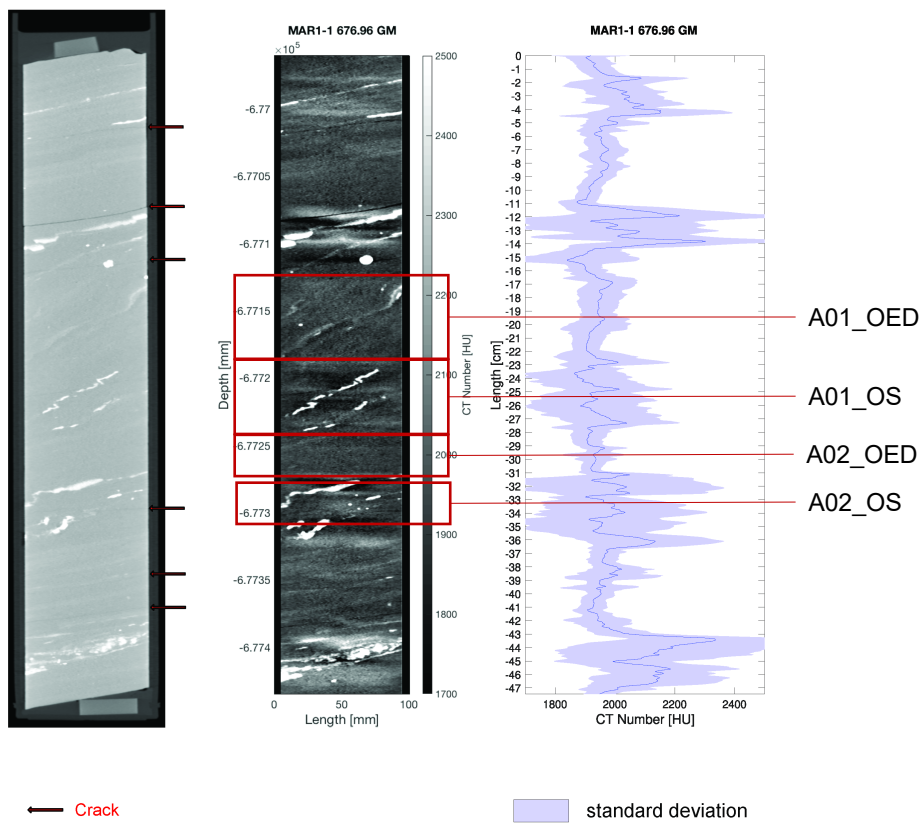
— Crack

standard deviation

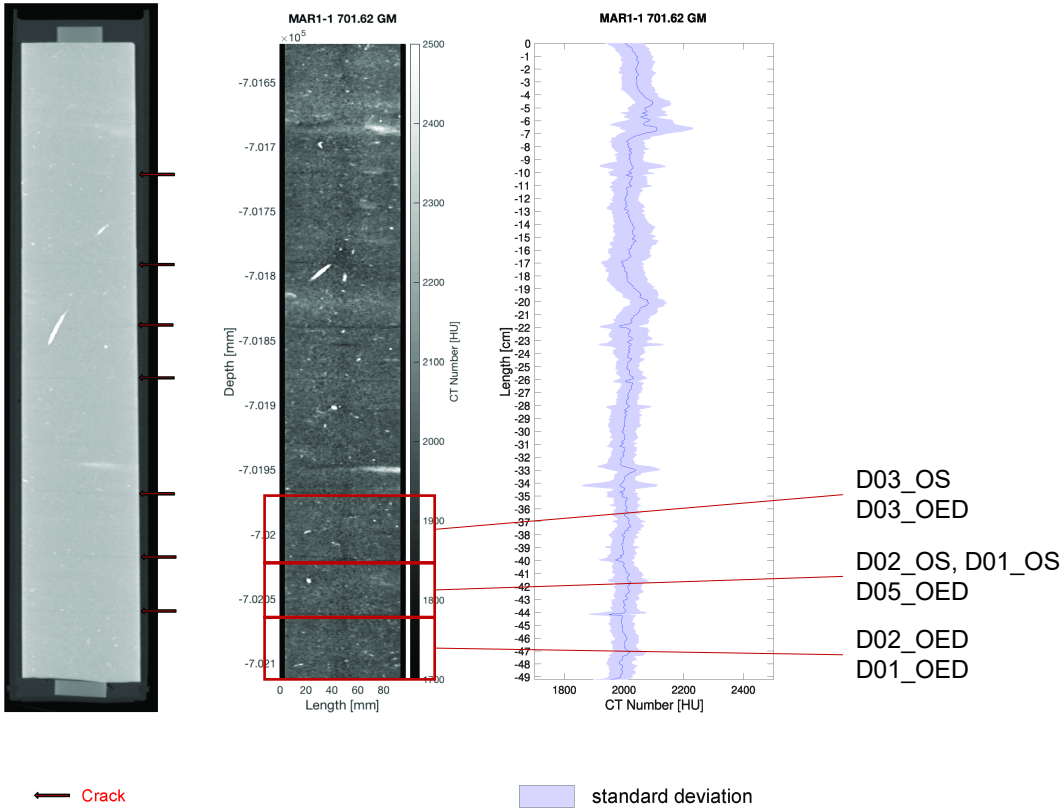
MAR1-1 663.37/50 GM



MAR1-1 676.96/49 GM



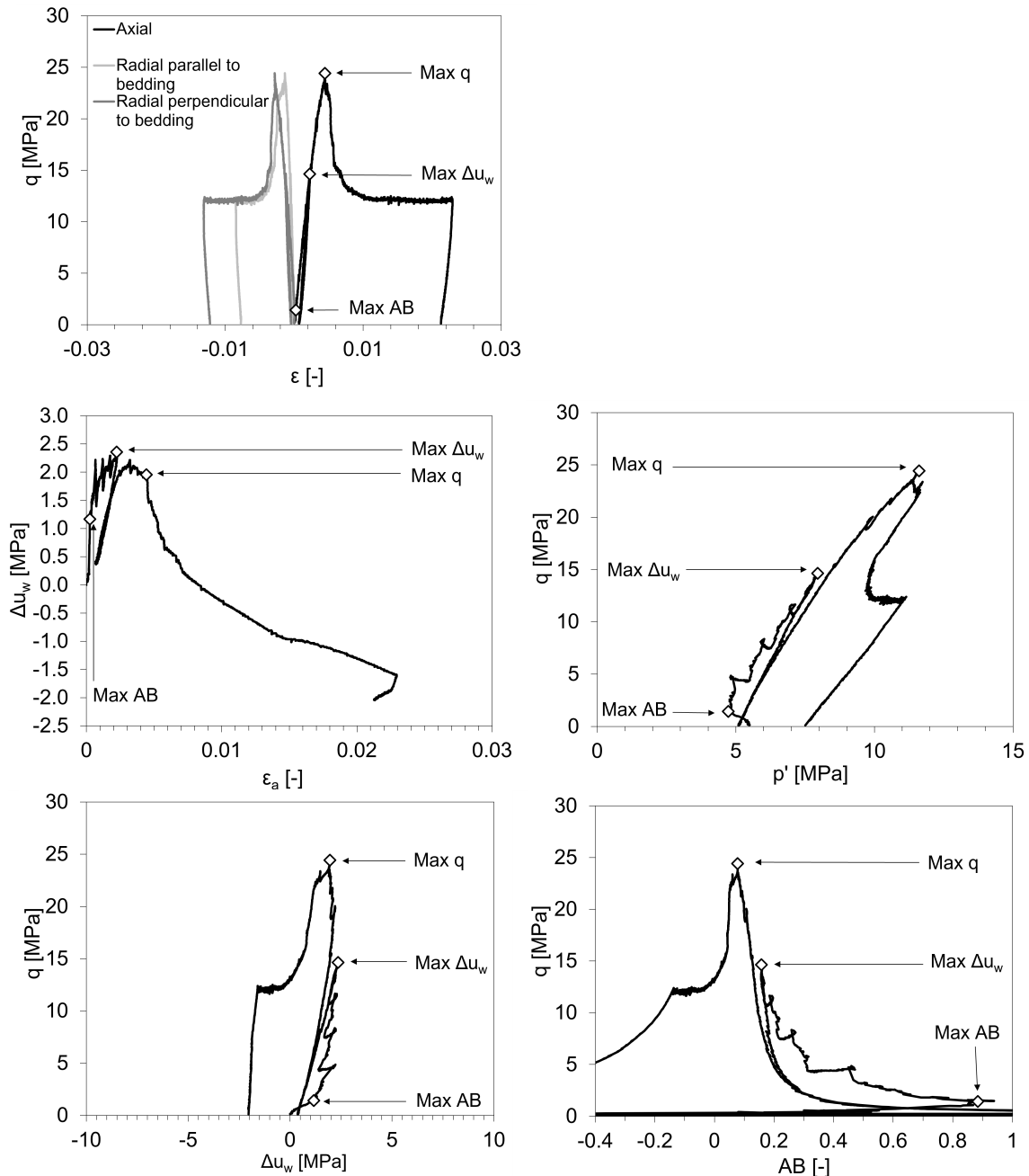
MAR1-1 701.62/51 GM



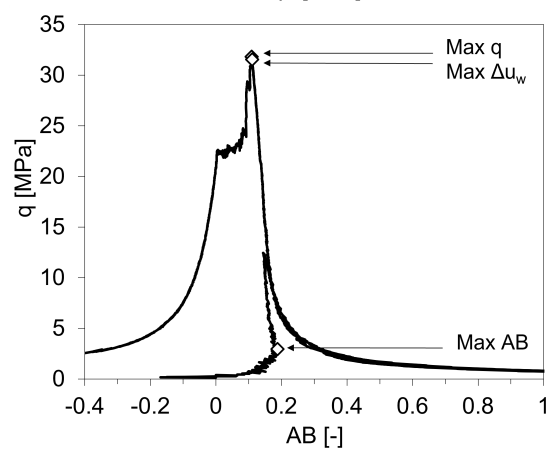
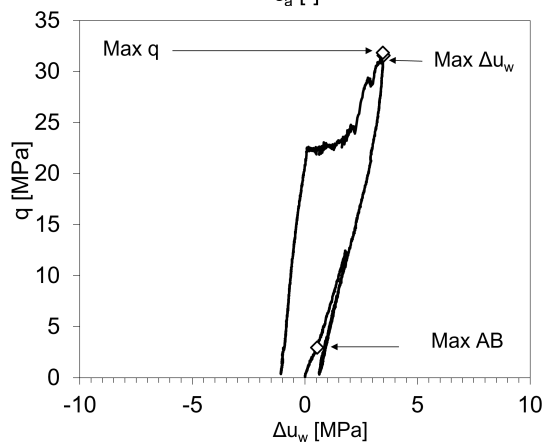
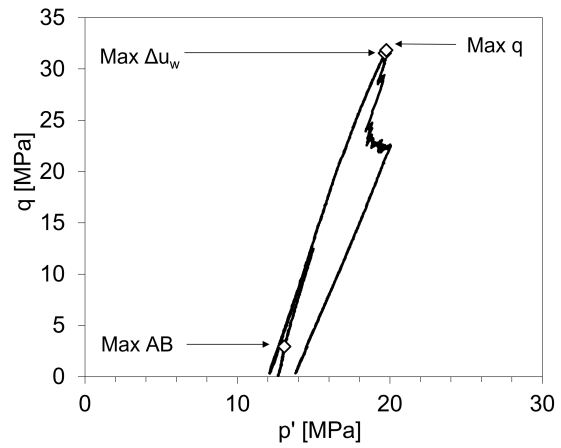
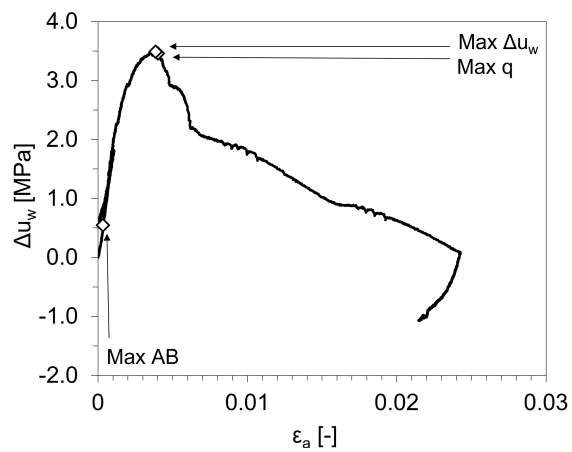
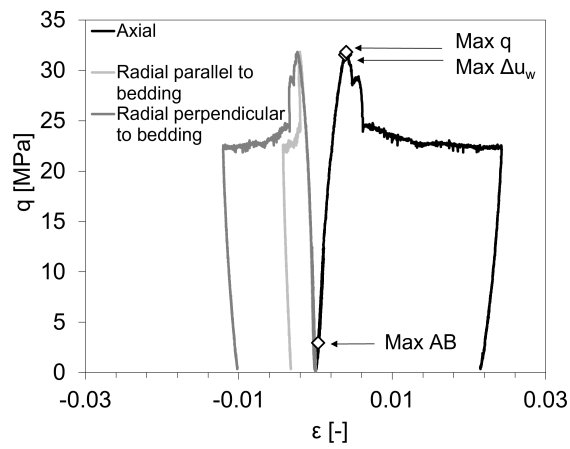
Appendix C Diagnostic plots of triaxial tests (geomechanical testing programme)

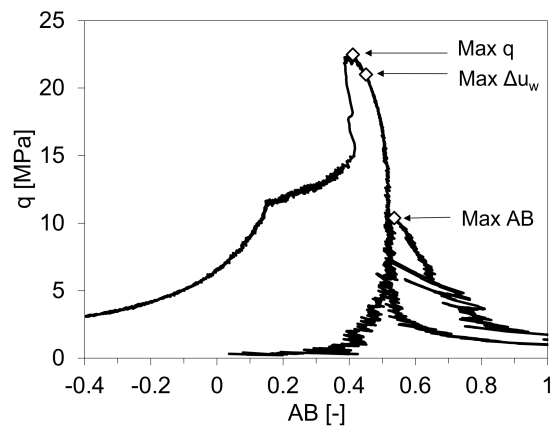
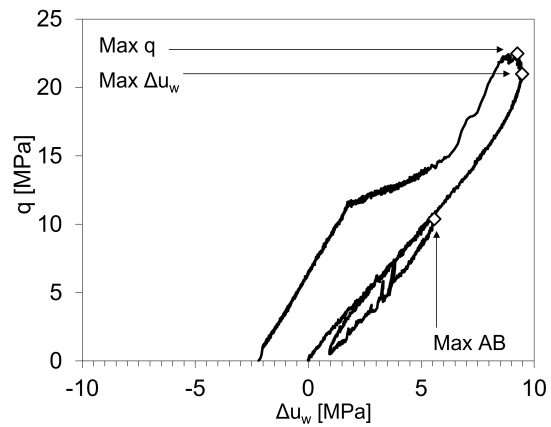
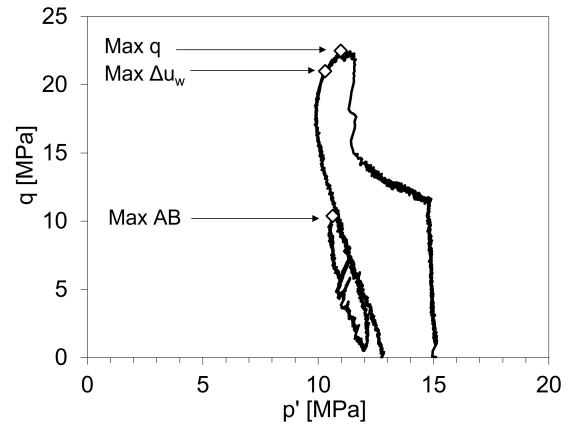
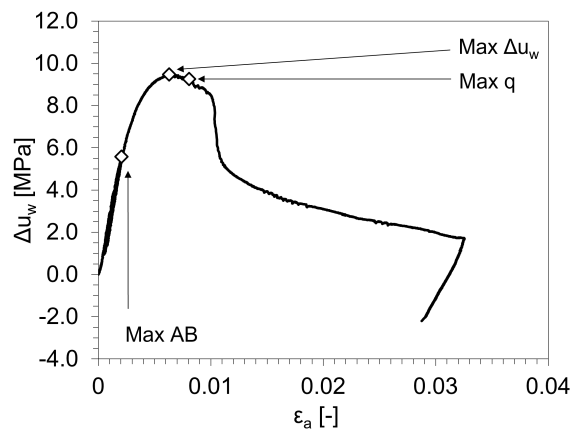
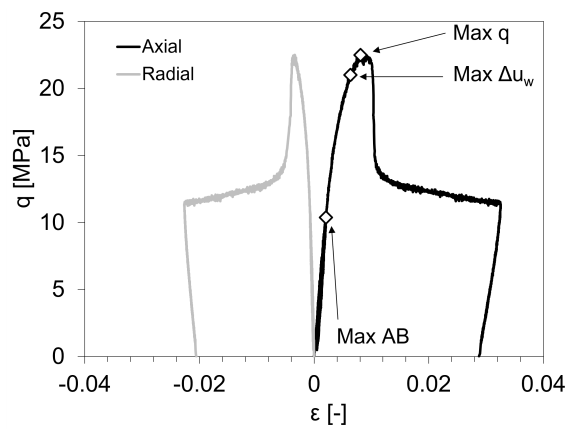
For each triaxial test, the diagnostic plots are here reported (order of tests according to Tab. E-2). The plots include: the evolution of axial and radial strains with deviatoric stress q , the pore pressure change Δu_w during shearing versus the axial strain ε_a , the stress path in the $p' - q$ plane, 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 6.2.2.4. Testing conditions are reported in Tab. 6-2.

C8_MAR1_1_608_35P6CTCU

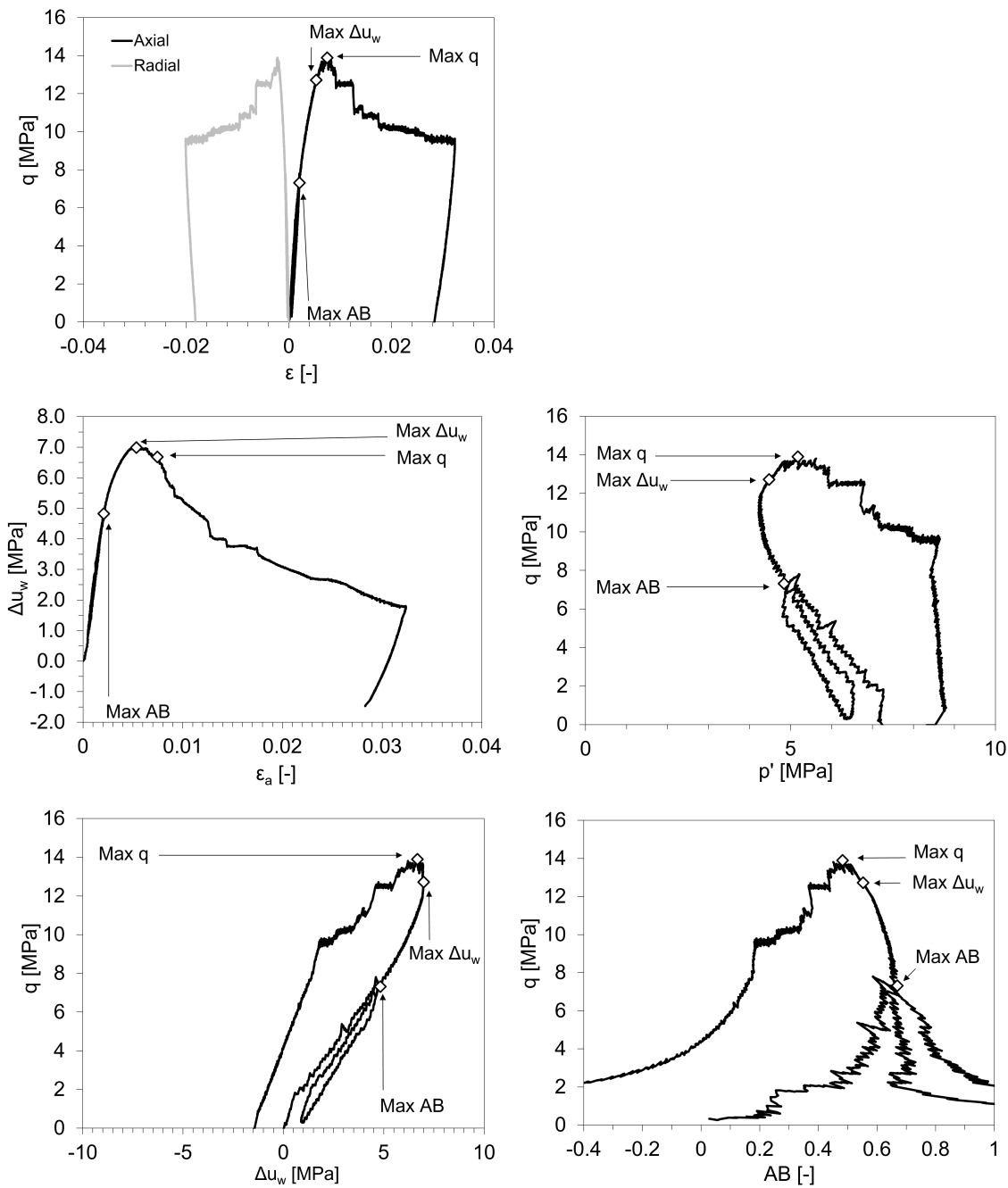


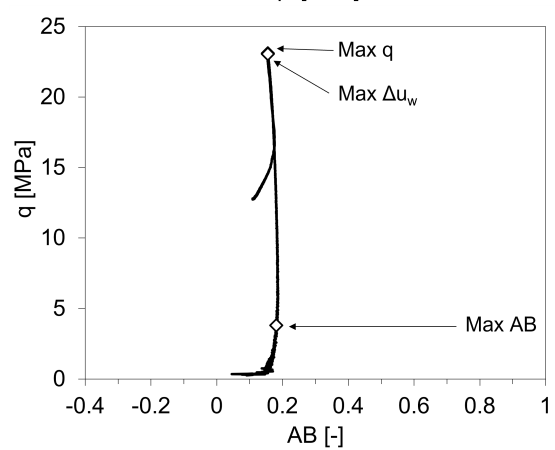
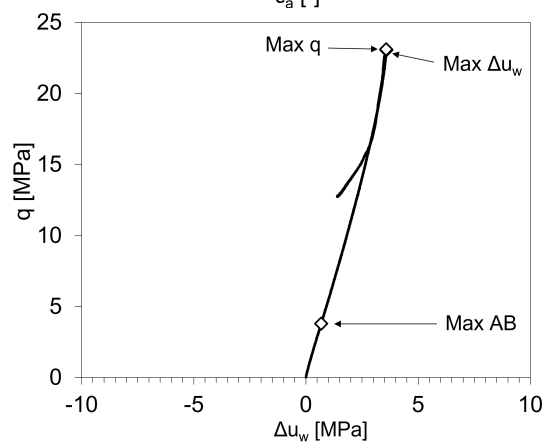
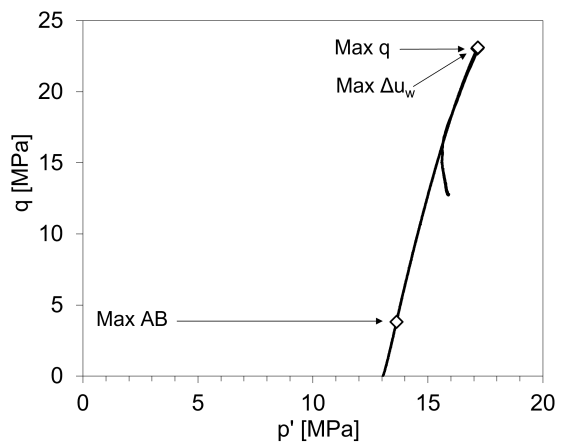
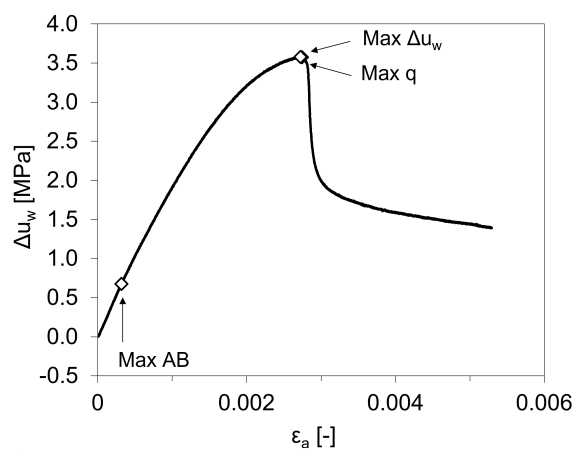
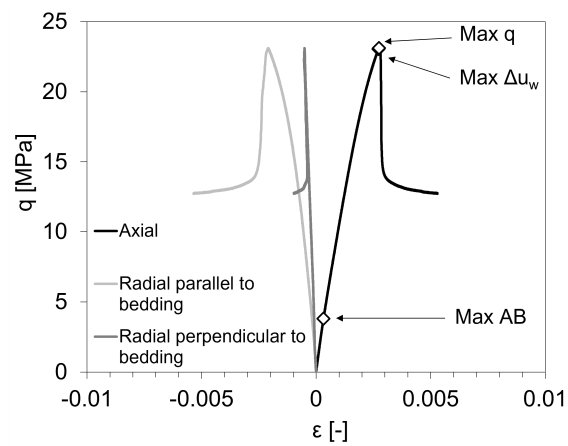
C6_MAR1_1_608_34P13CTCU



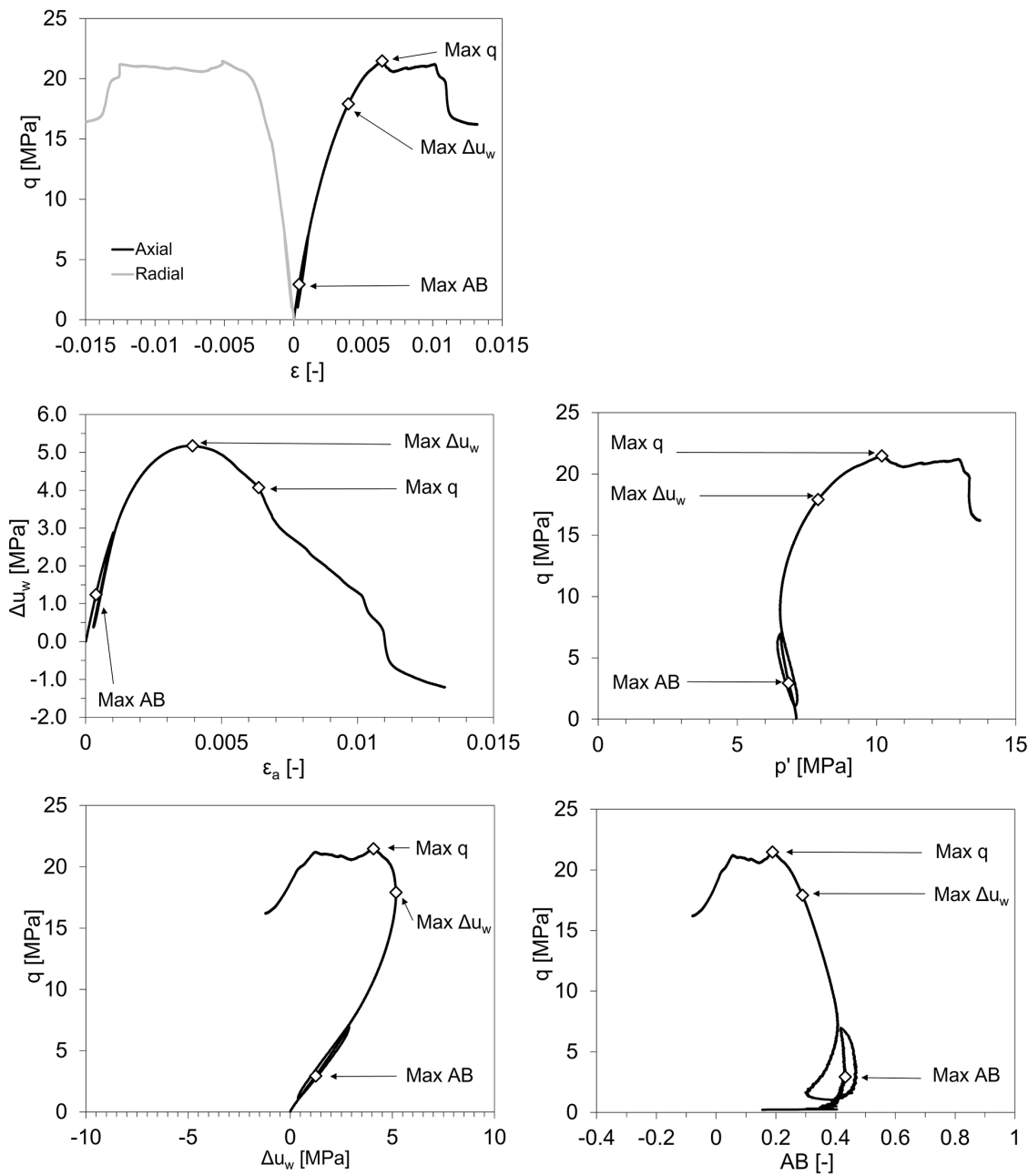
C5_MAR1_1_608_34S13CTCU

C7_MAR1_1_608_34S7CTCU

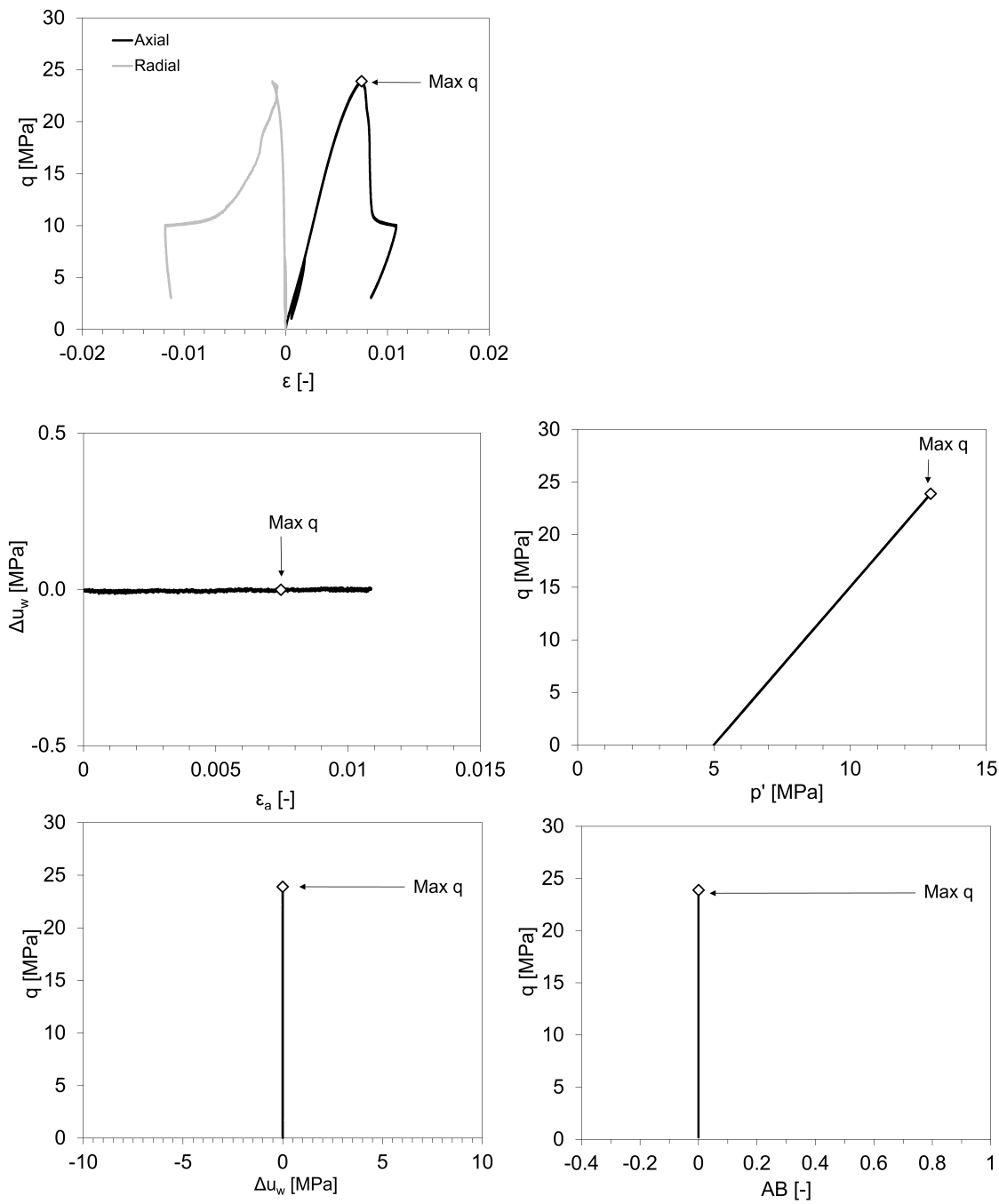


B5_MAR1_1_617_28Z13CTCU

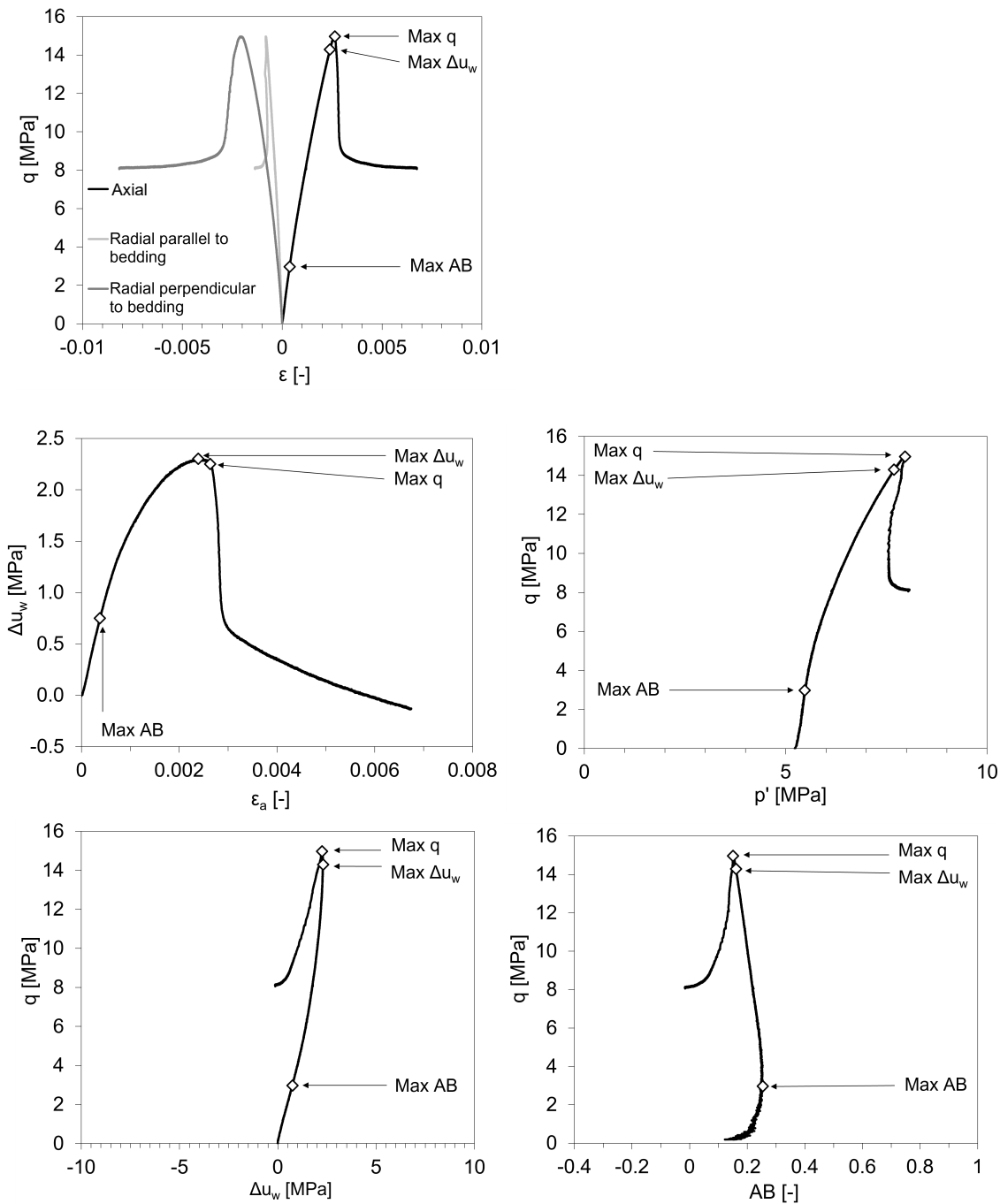
B1_MAR1_1_617_42S7CTCU



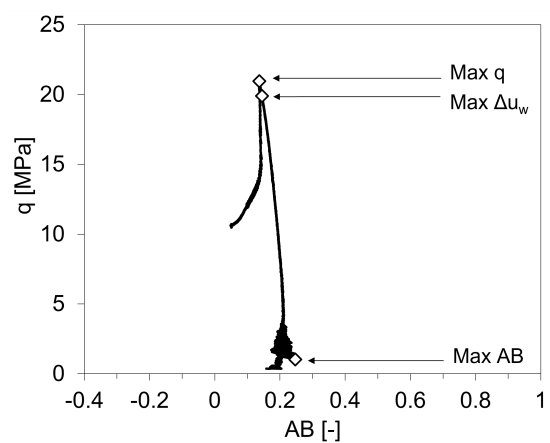
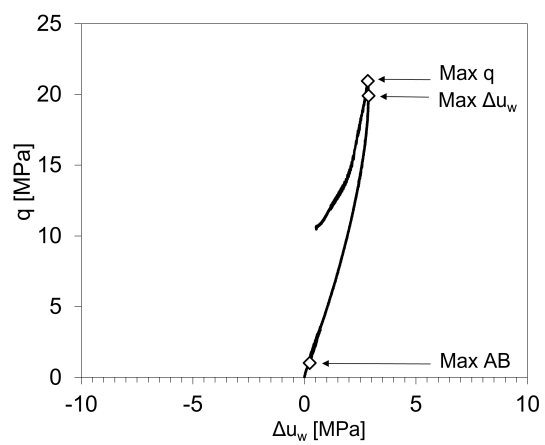
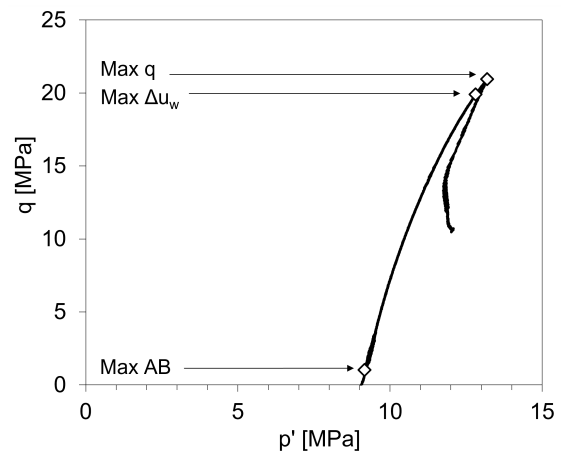
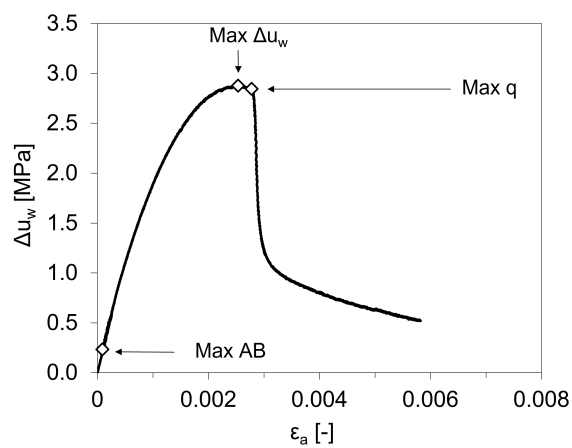
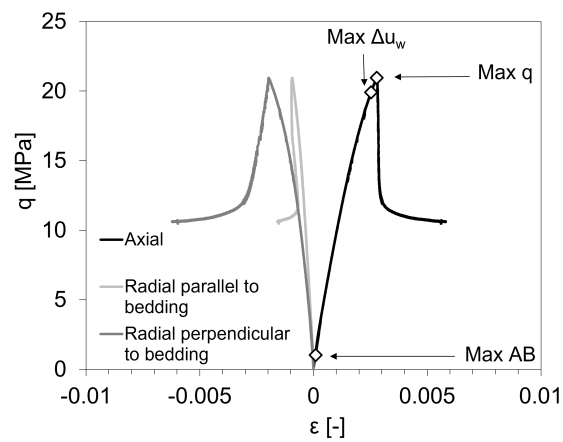
B2_MAR1_1_617_42S5CTC



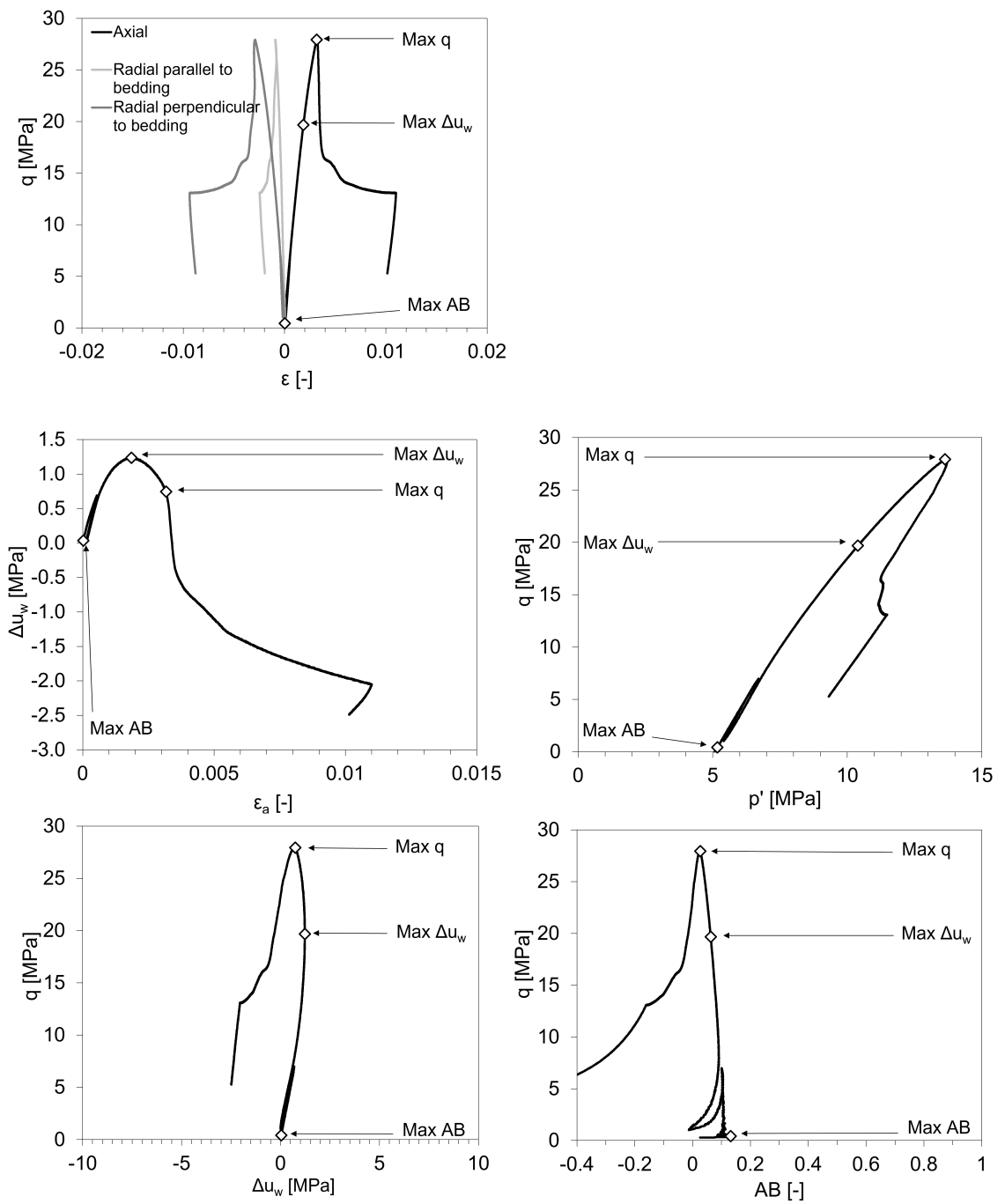
B6_MAR1_1_617_55Z5CTCU



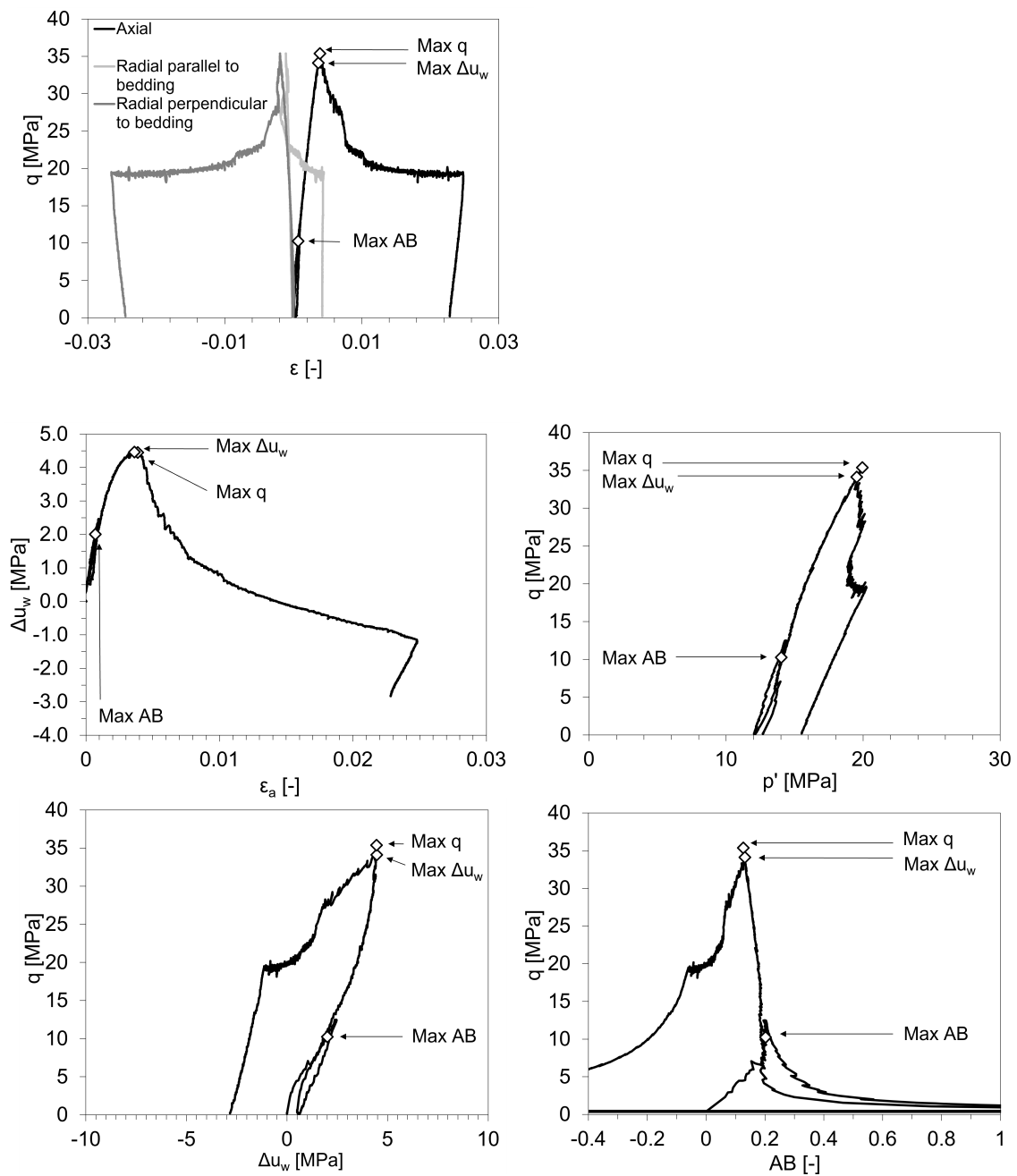
B7_MAR1_1_617_55Z9CTCU



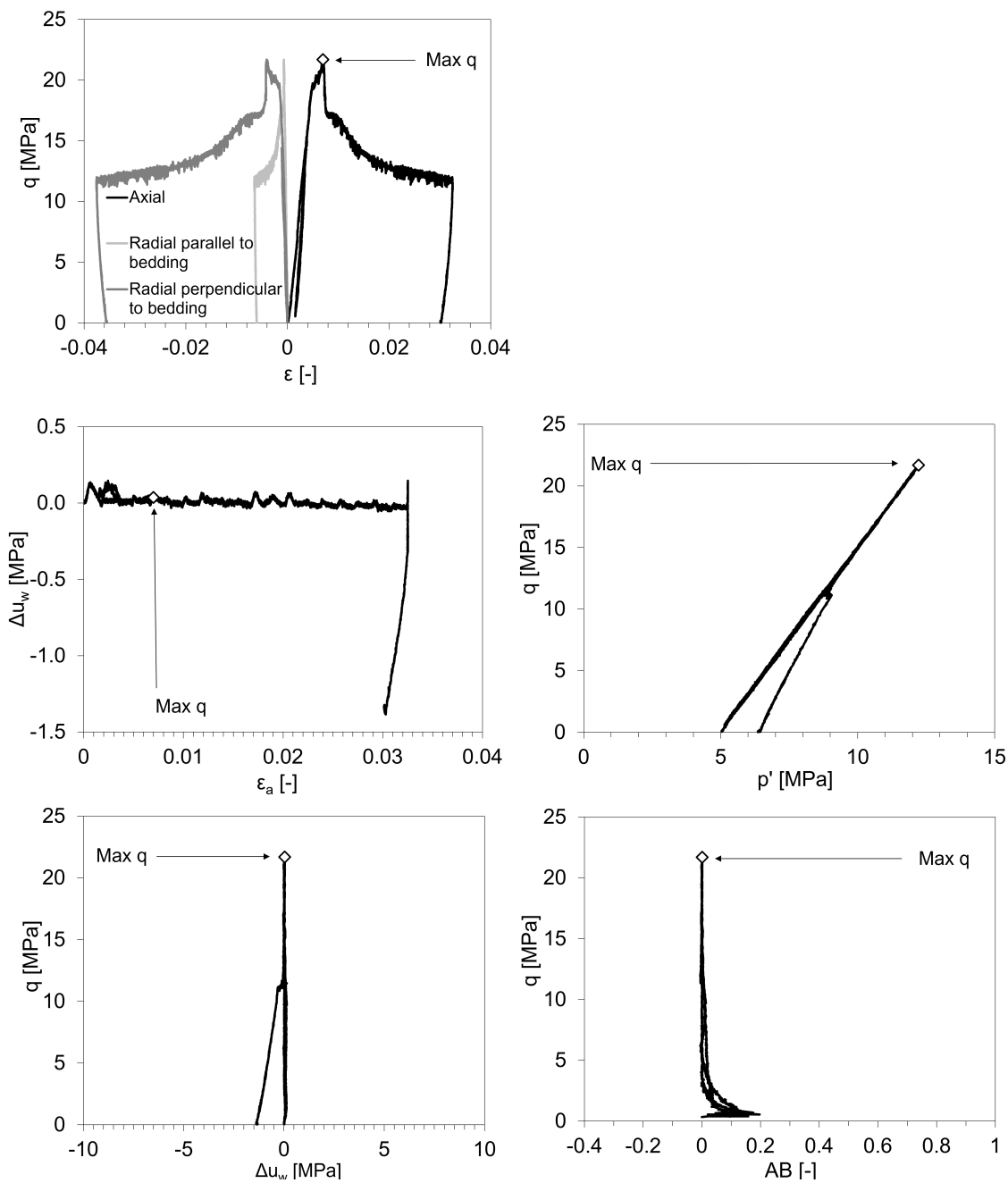
B8_MAR1_1_617_68P5CTCU



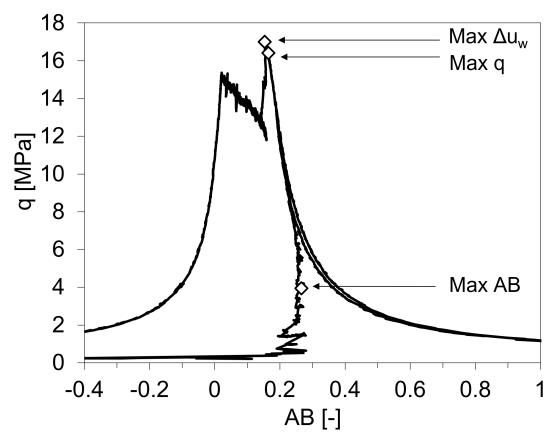
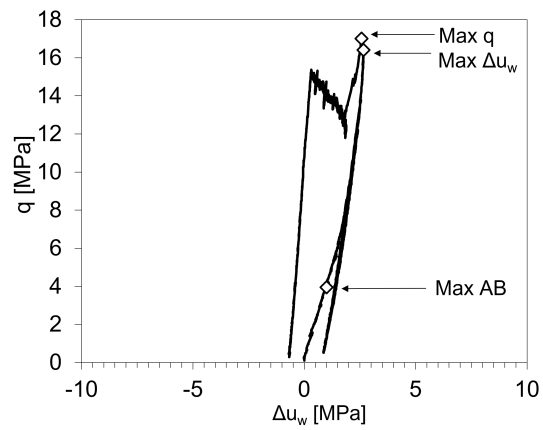
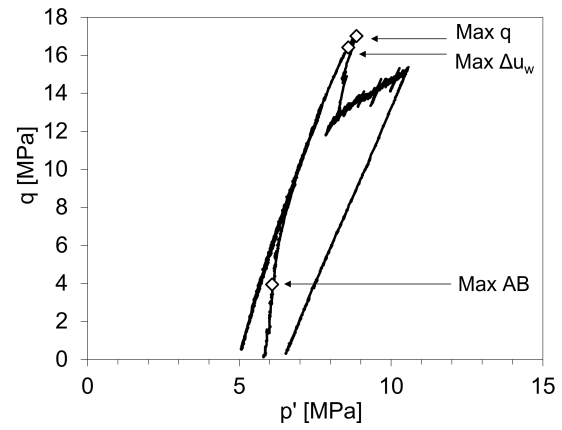
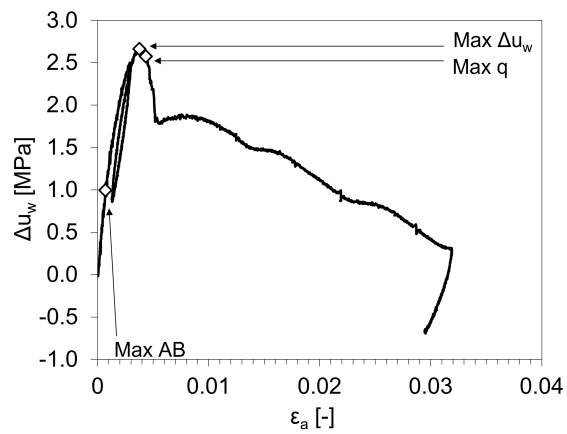
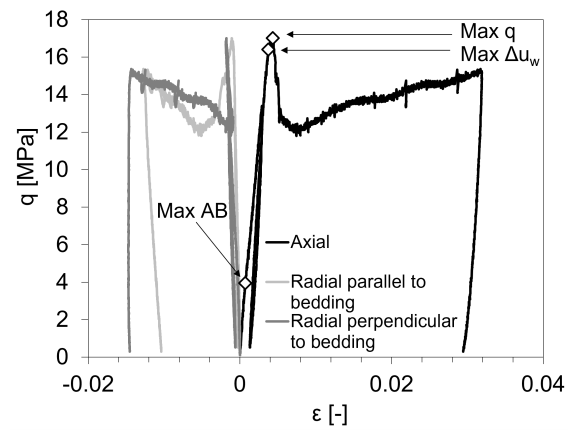
C2_MAR1_1_647_84P13CTCU



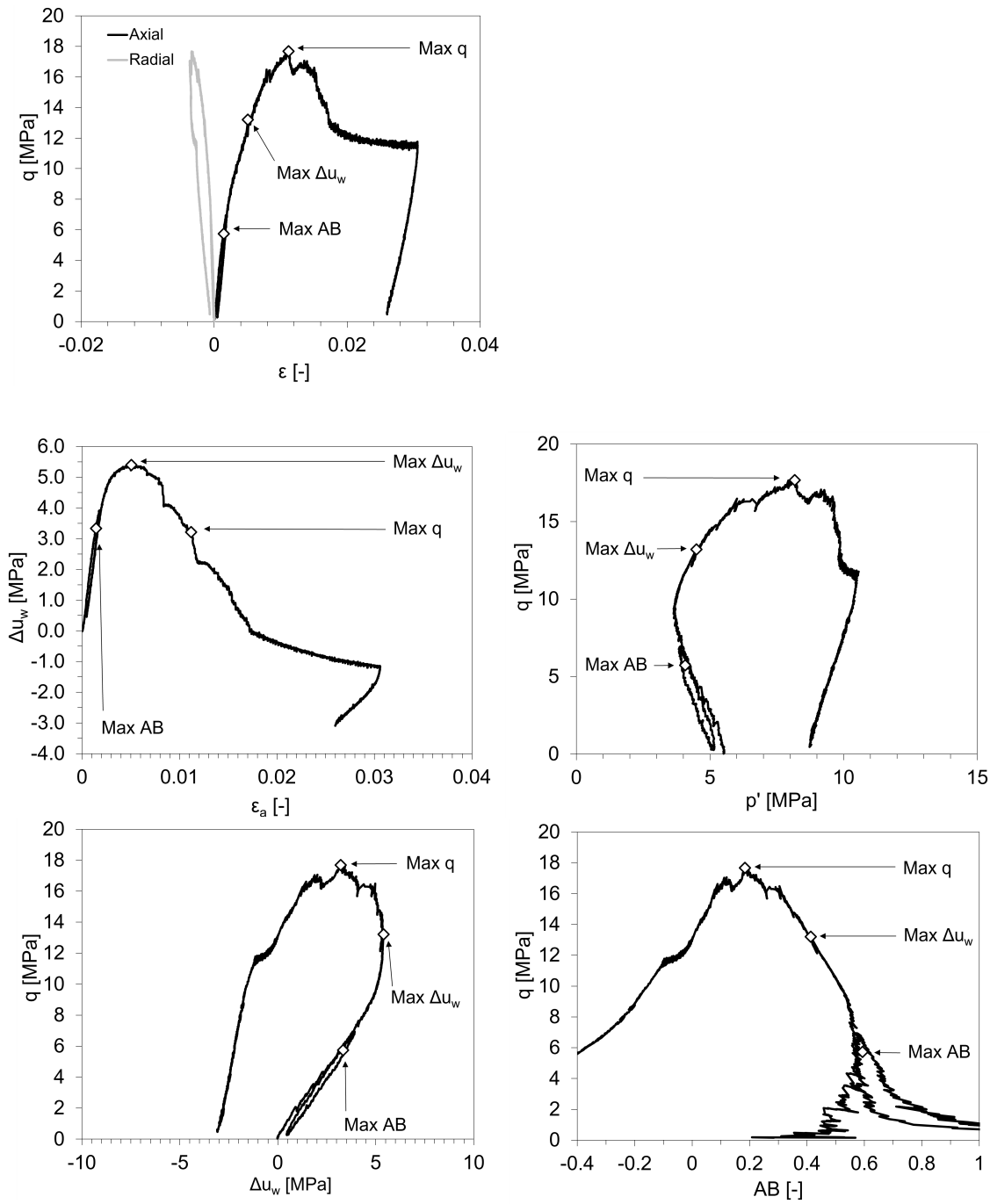
C12_MAR1_1_647_84P5CTC



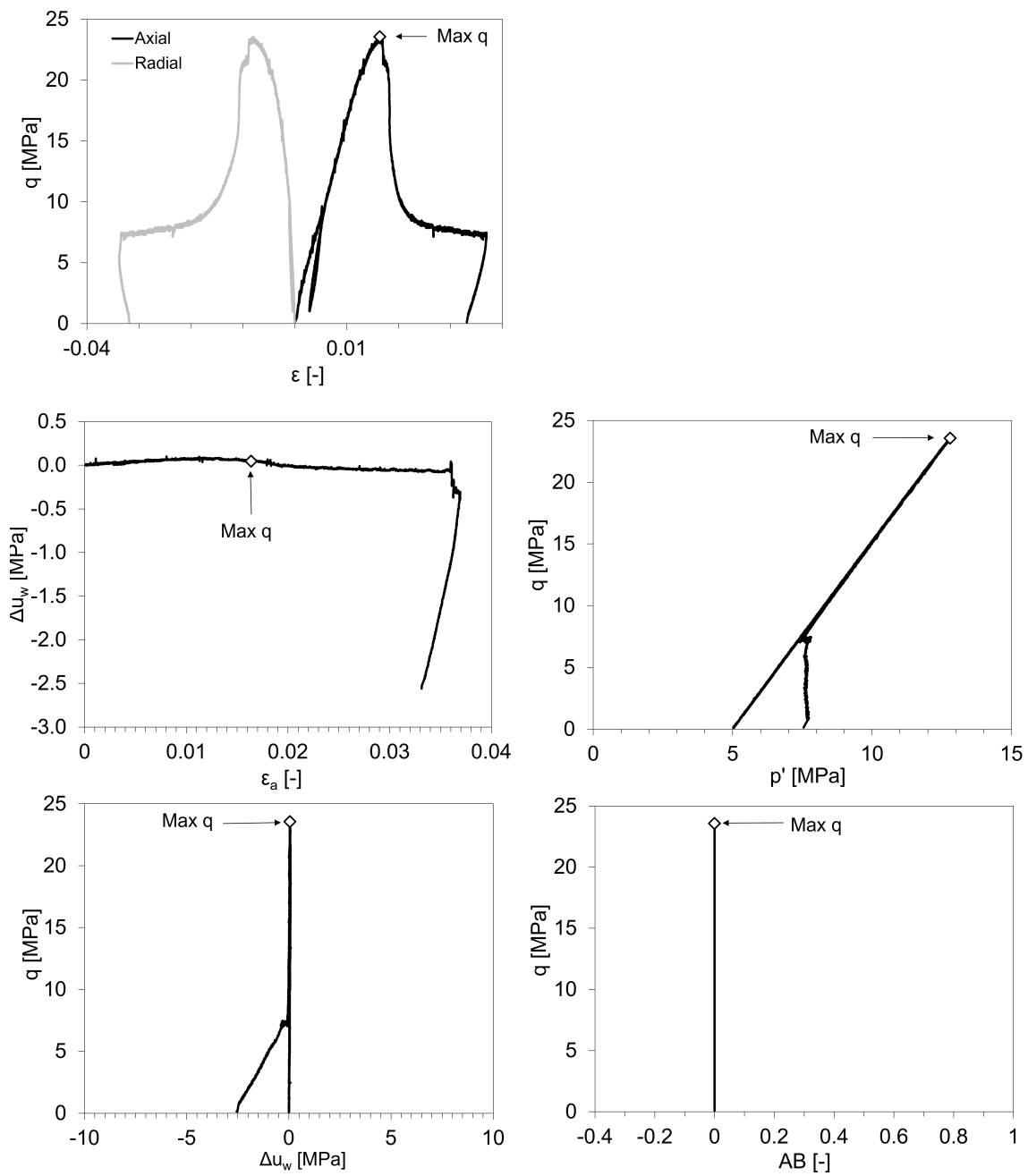
C4_MAR1_1_647_84P6CTCU



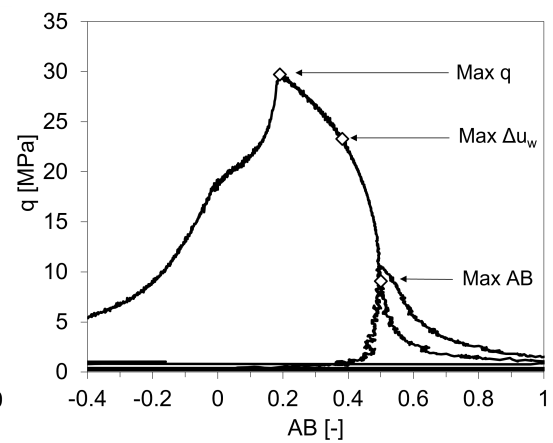
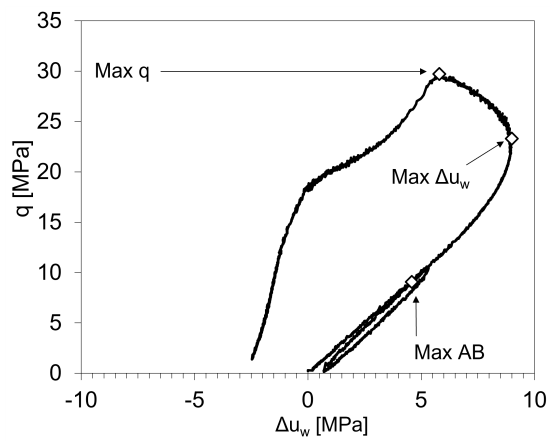
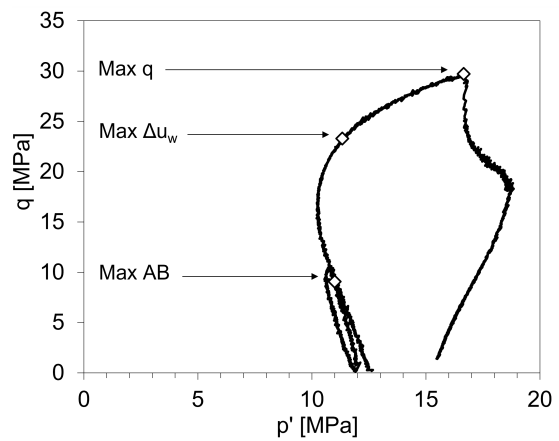
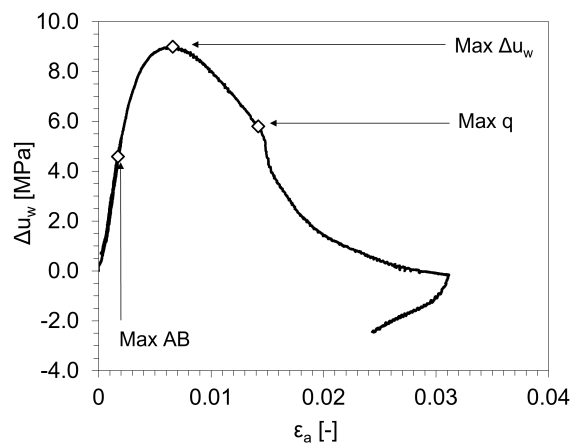
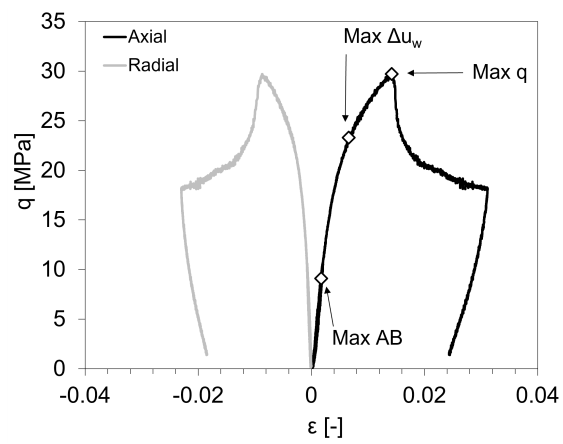
C3_MAR1_1_647_84S6CTCU



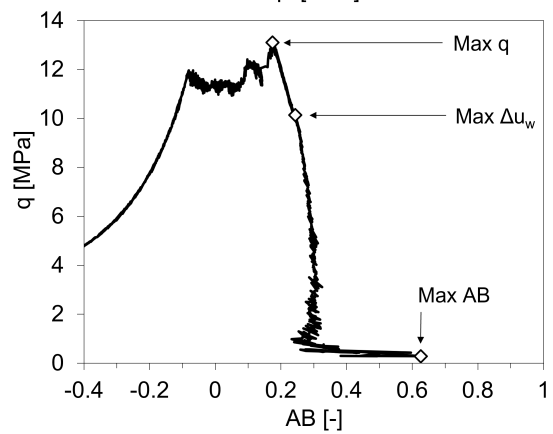
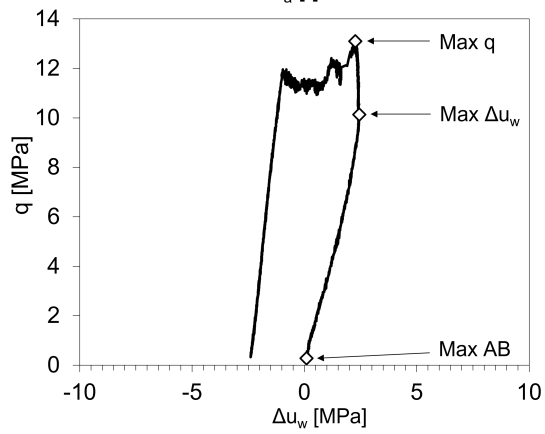
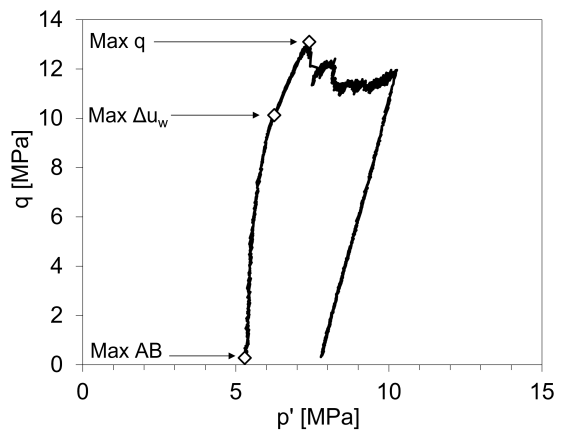
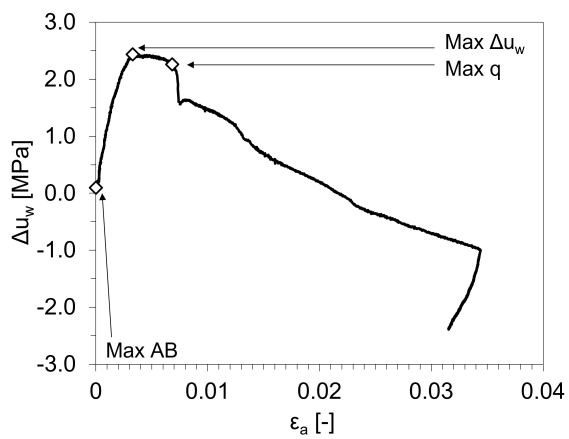
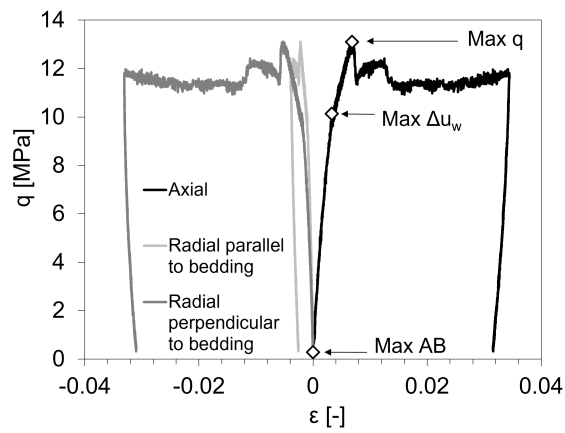
C11_MAR1_1_647_84S5CTC



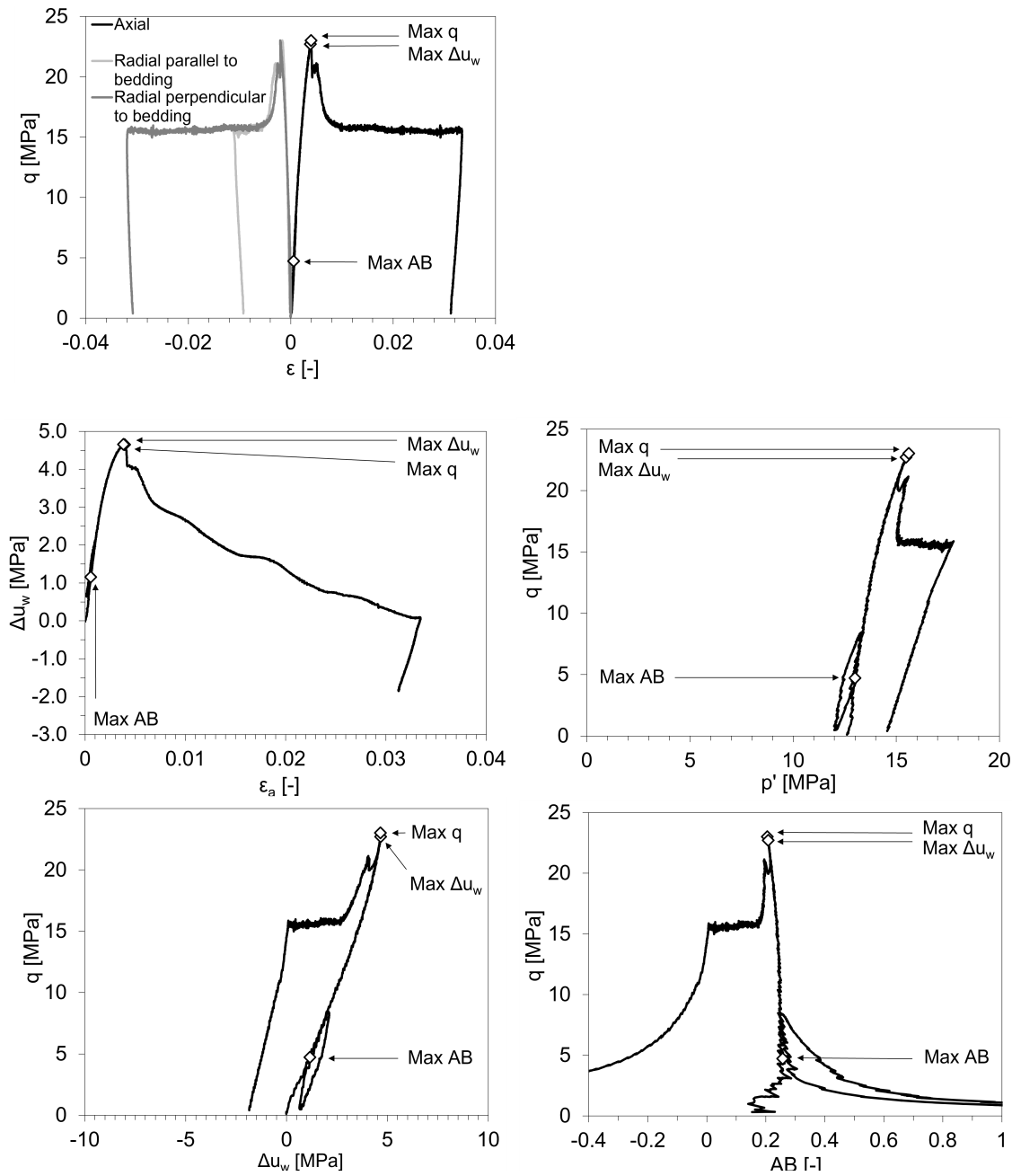
C1_MAR1_1647_84S13CTCU



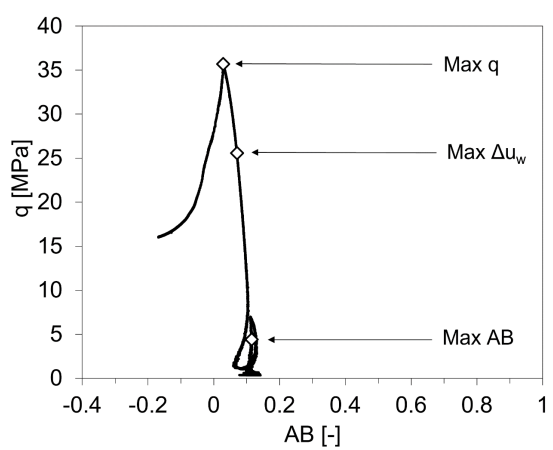
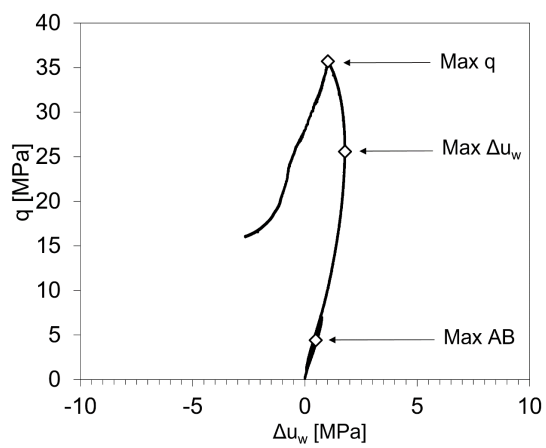
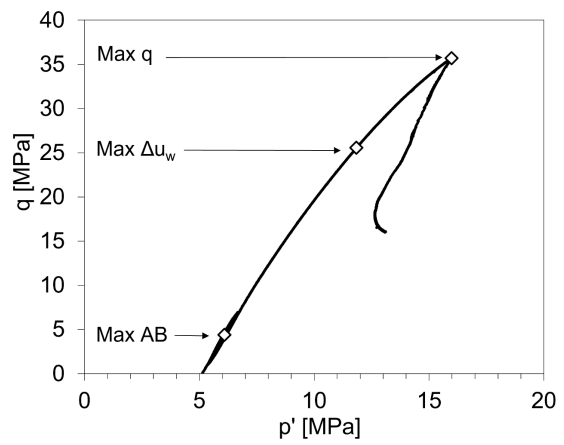
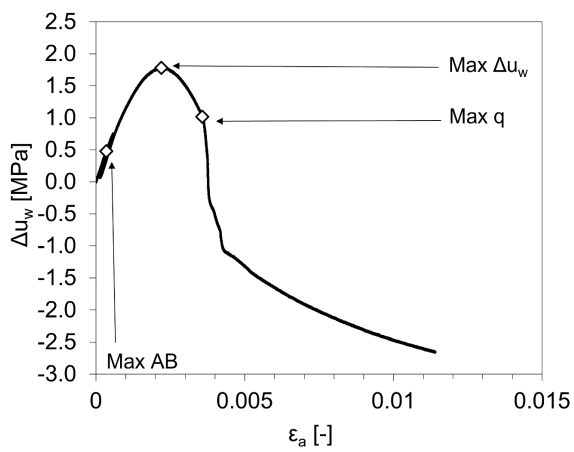
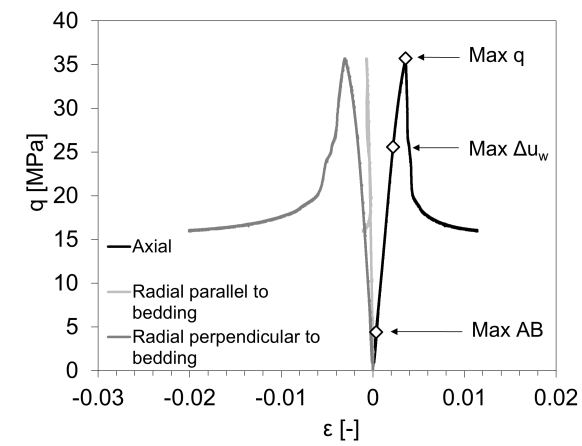
C9_MAR1_1_647_84Z6CTCU



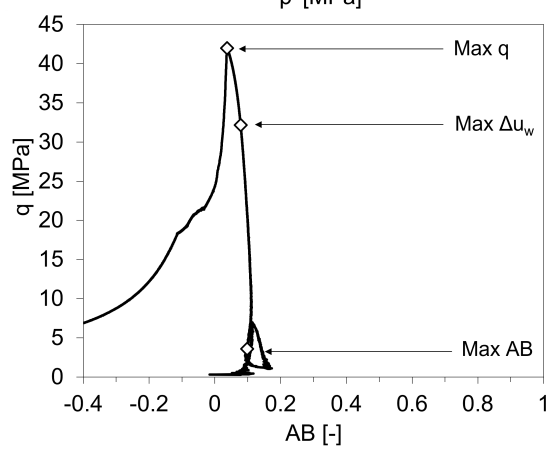
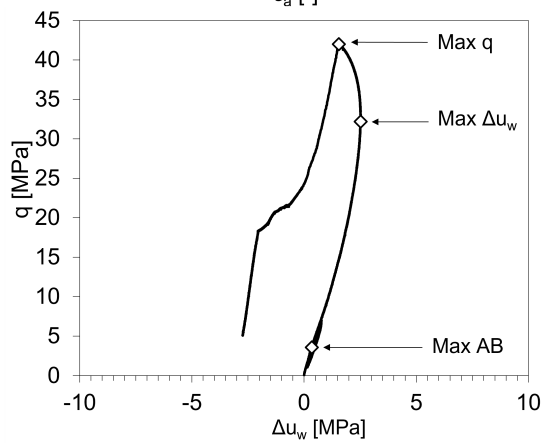
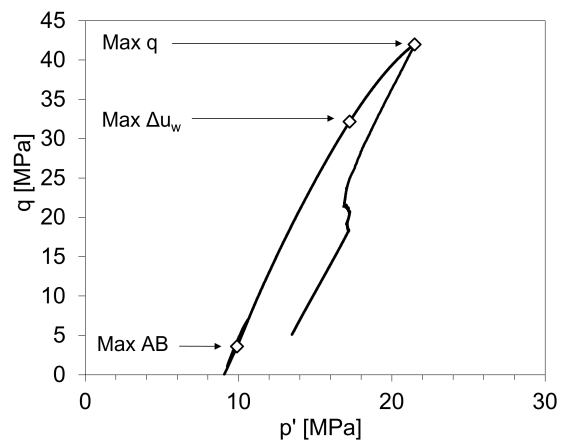
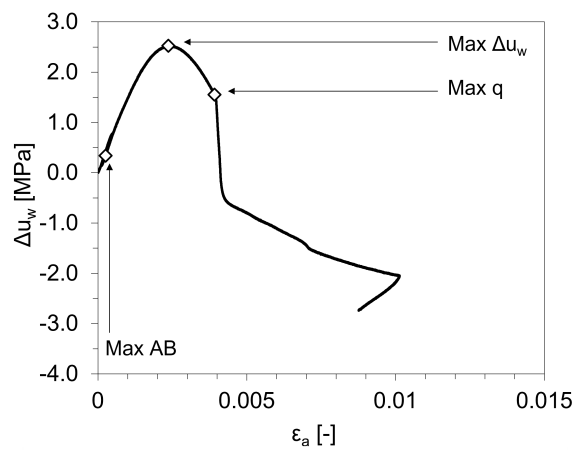
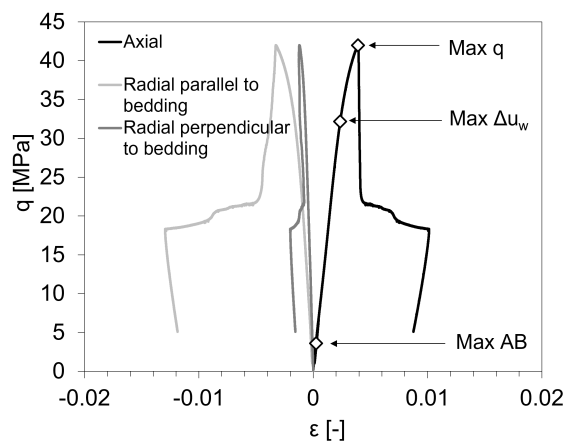
C10_MAR1_1_647_84Z13CTCU



B3_MAR1_1_663_46P5CTCU



B4_MAR1_1_663_76P9CTCU



Appendix D Mineralogical analysis of the tested samples

Mineralogical analysis and grain density performed by the University of Bern on a series of specimens. Part of the specimens had been previously mechanically tested in triaxial or oedometric conditions; other mineralogical analyses were performed on specimens that were not used in the mechanical programme. The specimens used for mineralogy-only are marked with *a*, and considered representative for the mineralogical composition of the portion of core from which it was extracted (i.e. lateral variability is negligible). The ID of one of the specimens sourced from the same depth is reported.

Specimen ID	Depth [m]	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 ⁻
D04_OED_MAR1_1 ^a	535.08	0.4	3.0	0.7	24.1	6.2	2.9	25.0	0.0	tr	0.8	40.3	2.689	6
C5_MAR1_1_608_34 S13CTCU ^a	608.35	0.2	0.8	1.2	17.2	5.2	2.8	6.7	0.0	0.0	0.4	66.6	2.675	2
B1_MAR1_1_617_42 S7CTCU ^a	617.43	0.3	1.5	1.0	29.3	5.2	3.5	10.4	0.0	2.3	0.5	47.8	2.670	5
D04_OS_MAR1_1 ^a	629.81	0.6	1.4	0.8	23.3	5.4	2.8	8.8	0.0	2.9	1.2	54.8	2.693	4
D05_OS_MAR1_1 ^a	629.85	0.5	1.2	0.8	25.2	5.7	3.5	7.7	0.0	2.6	0.9	53.6	2.697	9
C1_MAR1_1_647_84 S13CTCU ^a	647.84	0.2	1.4	1.0	21.2	4.5	3.2	8.6	0.0	3.9	0.3	57.4	2.695	5
Additional specimen ^a	663.42	0.2	1.3	1.1	24.6	5.4	3.2	7.9	0.0	2.9	0.3	54.5	2.662	7
Additional specimen ^a	663.76	0.1	1.2	1.1	23.2	5.6	3.5	8.7	0.0	1.8	0.1	56.0	2.663	9
A01_OED_MAR1_1 ^a	677.16	0.1	1.3	1.1	15.7	5.4	3.3	5.9	0.0	5.5	0.3	62.9	2.687	8
A01_OS_MAR1_1	677.22	0.9	1.2	1.0	17.3	5.5	3.4	6.9	0.0	3.4	1.7	60.9	2.692	7
A02_OED_MAR1_1 ^a	677.26	0.1	1.1	1.1	16.3	5.3	2.8	6.0	0.0	3.5	0.3	64.7	2.675	8
A02_OS_MAR1_1	677.30	0.1	1.7	1.0	14.4	5.4	3.6	5.9	0.0	9.8	0.2	59.8	2.720	6
D03_OS_MAR1_1	701.99	0.4	1.1	0.6	15.8	4.9	3.4	6.7	0.0	3.0	0.8	64.8	2.708	4
D01_OS_MAR1_1	702.05	0.4	1.1	0.8	16.3	5.1	2.8	7.1	0.0	2.8	0.8	64.3	2.692	6
D02_OED_MAR1_1	702.07	0.4	1.1	0.7	15.9	5.4	2.7	7.4	0.0	2.5	0.8	64.6	2.694	6

Appendix E Triaxial test results

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

Specimen ID, specimen geometry, mean effective and deviatoric stress (if applicable) at which the measurements were performed, P and S waves in the cylinder axis direction (V_p , V_s), P wave in the direction perpendicular to the axis direction (V_{ph}). For specimen geometry see Fig. 4-1. ^d Drained shearing.

Specimen ID	Specimen geometry	p' [MPa]	q [MPa]	V_p [m/s]	V_s [m/s]	V_{ph} [m/s]
C8_MAR1_1_608_35P6CTCU	P	5.9	7.8	3'880	1'992	
C6_MAR1_1_608_34P13CTCU	P	14.1	8.1	3'970	2'228	
C5_MAR1_1_608_34S13CTCU	S	11.5	7.1	2'994	1'531	
C7_MAR1_1_608_34S7CTCU	S	5.9	5.2	2'864	1'485	
B5_MAR1_1_617_28Z13CTCU	Z	13.1		3'672	1'862	3'425
B1_MAR1_1_617_42S7CTCU	S	7.1		3'400	1'480	
B2_MAR1_1_617_42S5CTC	S	5.0			1'970	
B6_MAR1_1_617_55Z5CTCU	Z	5.1		3'537	1'976	3'391
B7_MAR1_1_617_55Z9CTCU	Z	9.1		3'624	2'058	3'358
B8_MAR1_1_617_68P5CTCU	P	5.1		3'917	2'332	
C2_MAR1_1_647_84P13CTCU	P	14.0	9.5	4'055	2'040	
C12_MAR1_1_647_84P5CTC	P	8.4	10.2	4'003 ^d	2'070 ^d	
C4_MAR1_1_647_84P6CTCU	P	6.6	7.1	4'105	1'900	
C3_MAR1_1_647_84S6CTCU	S	4.3	5.2	3'042	1'509	
C11_MAR1_1_647_84S5CTC	S	7.1	6.4	3'039 ^d	1'654 ^d	
C1_MAR1_1647_84S13CTCU	S	11.3	7.0	3'202	1'678	
C9_MAR1_1_647_84Z6CTCU	Z	5.7	6.2	3'583	1'967	
C10_MAR1_1_647_84Z13CTCU	Z	13.2	5.6	3'797	2'076	
B3_MAR1_1_663_46P5CTCU	P	5.1		3'859	2'205	3'300
B4_MAR1_1_664_15P9CTCU	P	9.1		3'914	2'270	3'337

Specimen 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:

- Specimen ID
- Core depth
- Formation
- Specimen 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)

For specimen geometry see Fig. 4-1.

Specimen ID	Specimen geometry	ρ_{bulk} [g/cm ³]	e_0 [-]	Water content		B check		Swelling pressure		Consolidation phase		Strain rate	Stress path
				Initial [%]	Final [%]	p' [MPa]	B [-]	Axial [MPa]	Radial [MPa]	p' [MPa]	c_v [mm ² /s]	$\dot{\epsilon}$ [s ⁻¹]	
C8_MAR1_1_608_35P6CTCU	P	2.47	0.158	6.08	6.00	2	0.94					1×10^{-7}	CTCU
C6_MAR1_1_608_34P13CTCU	P	2.47	0.160	6.16	6.09	4	0.93			13.1	0.00014	1×10^{-7}	CTCU
C5_MAR1_1_608_34S13CTCU	S	2.46	0.156	5.52	5.57	8	0.88					1×10^{-7}	CTCU
C7_MAR1_1_608_34S7CTCU	S	2.47	0.157	6.01	5.83	3	0.93			7	0.00020	1×10^{-7}	CTCU
B5_MAR1_1_617_28Z13CTCU	Z	2.50	0.132	4.98	5.12	18.5	0.82	18.2	16.8	13	0.00091	7×10^{-8}	CTCU
B1_MAR1_1_617_42S7CTCU	S	2.51	0.144	6.46	6.76	17.4	0.83	17.2	9.4	7	0.00118	9×10^{-8}	CTCU
B2_MAR1_1_617_42S5CTC	S	2.52	0.119	4.53	4.99	18	0.83	17.8	12.2	5	0.00129	2×10^{-8}	CTC
B6_MAR1_1_617_55Z5CTCU	Z	2.51	0.128	4.91	5.11	17.7	0.84	17.6	15.4	5	0.00289	7×10^{-8}	CTCU
B7_MAR1_1_617_55Z9CTCU	Z	2.51	0.131	5.08	5.17	18	0.83	17.8	15.1	9	0.00566	7×10^{-8}	CTCU
B8_MAR1_1_617_68P5CTCU	P	2.49	0.143	5.42	5.78	20.7	0.86	20.7	19.6	5	0.00222	7×10^{-8}	CTCU
C2_MAR1_1_647_84P13CTCU	P	2.52	0.131	5.40	5.32	4.5	0.91			13	0.00007	1×10^{-7}	CTCU
C12_MAR1_1_647_84P5CTC	P	2.49	0.141	5.39	5.42	2.5	0.94			5	0.00012	2×10^{-8}	CTC
C4_MAR1_1_647_84P6CTCU	P	2.51	0.115	3.73	3.59	3	0.95			5.8	0.00058	1×10^{-7}	CTCU
C3_MAR1_1_647_84S6CTCU	S	2.52	0.129	5.27	5.27	3	0.92					1×10^{-7}	CTCU
C11_MAR1_1_647_84S5CTC	S	2.48	0.147	5.51	5.37	3.2	0.91					2×10^{-8}	CTC
C1_MAR1_1647_84S13CTCU	S	2.51	0.130	5.16	5.18	3.5	0.90			13	0.00017	1×10^{-7}	CTCU
C9_MAR1_1_647_84Z6CTCU	Z	2.49	0.144	5.66	5.49	2	0.94			6	0.00052	1×10^{-7}	CTCU
C10_MAR1_1_647_84Z13CTCU	Z	2.50	0.136	5.34	5.23	9	0.85					1×10^{-7}	CTCU
B3_MAR1_1_663_41P5CTCU	P	2.48	0.142	4.93	5.25	20.5	0.83	20.2	20.1	5	0.00222	7×10^{-8}	CTCU
B4_MAR1_1_663_76P9CTCU	P	2.51	0.133	5.16	5.34	22.7	0.82	22.3	22.2	9	0.00196	7×10^{-8}	CTCU

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

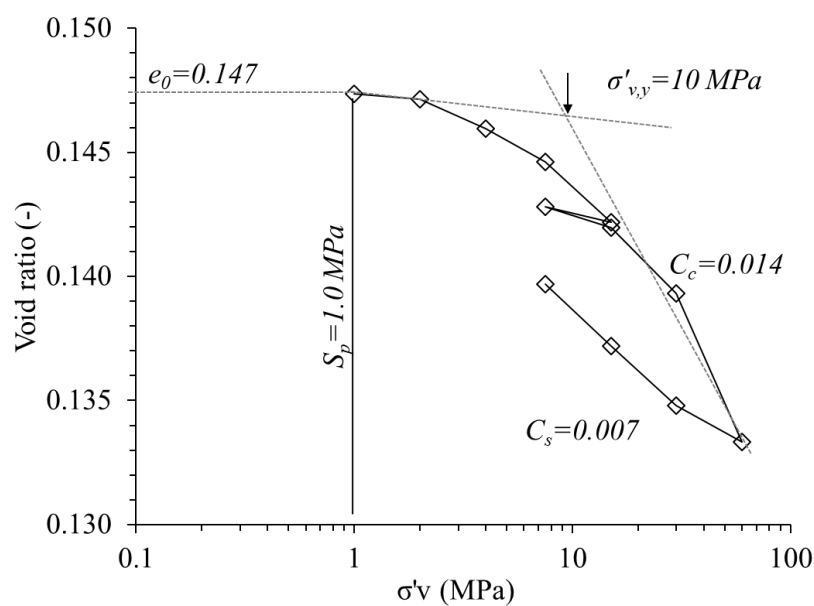
^dDrained elastic modulus

Specimen 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]	$\nu_{\perp}$ [-]	$\nu_{\parallel}$ [-]	q [MPa]	p' [MPa]	q [MPa]	p' [MPa]	q [MPa]	p' [(MPa)]	q [MPa]	p' [MPa]	p' [MPa]	E [GPa]
C8_MAR1_1_608_35P6CTCU	7.9	10.5	14.04	0.54	0.23					24.4	11.4	1.4	4.6	14.6	8.0	12.0	11.0	11.0	6.6
C6_MAR1_1_608_34P13CTCU	15.0	13.9	21.9	0.34	0.39					31.8	19.7	3.0	13.1	31.6	19.6	22.0	19.9	19.7	11.1
C5_MAR1_1_608_34S13CTCU	10.6	6.35	7.94	0.36						22.5	11.0	10.4	10.6	21.0	10.3	11.5	14.8	14.8	2.5
C7_MAR1_1_608_34S7CTCU	5.2	4.99	6.9	0.27						13.9	5.2	7.3	4.8	12.7	4.5	9.5	8.5	8.6	3.0
B5_MAR1_1_617_28Z13CTCU										23.1	17.2	3.8	13.6	23.0	17.2	12.7	15.9		
B1_MAR1_1_617_42S7CTCU	6.5	8	10.1							21.5	10.2	2.9	6.8	17.9	7.9	16.2	13.7		
B2_MAR1_1_617_42S5CTC						7.3	4.9	0.2		23.9	12.9	0.0	5.0	10.2	5.0	10.2	8.4	8.4	2.6
B6_MAR1_1_617_55Z5CTCU										15.0	8.0	3.0	5.9	14.3	7.7	8.1	7.9		
B7_MAR1_1_617_55Z9CTCU										20.9	13.2	1.0	9.2	19.9	12.8	10.0	12.0		
B8_MAR1_1_617_68S5CTCU	6.7	13.3	16.3	0.48	0.4					27.9	13.6	0.4	5.5	19.7	10.4	13.3	11.3	11.4	9.4
C2_MAR1_1_647_84P13CTCU	14.3	23.45	32	0.48	0.55					35.4	19.9	10.3	14.0	34.1	19.5	18.6	20.0	20.3	9.2
C12_MAR1_1_647_84P5CTC						9.8	7.8	0.59	0.23	21.7	12.2	0.5	5.2	9.7	8.1	10.6	8.9		
C4_MAR1_1_647_84P6CTCU	7.7	8.8	13.4	0.57	0.25					17.0	8.8	3.9	6.1	16.4	8.5	14.0	9.5	10.4	7.3
C3_MAR1_1_647_84S6CTCU	3.8	5.11	6.05	0.33						17.7	8.1	5.7	4.1	13.2	4.3	11.5	10.5	10.5	3.4
C11_MAR1_1_647_84S5CTC						7.9	4.2	0.27		23.6	12.7	0.2	7.0	13.4	9.4	7.2	7.5		
C1_MAR1_1647_84S13CTCU	10.8	7.5	9.4	0.38						29.7	16.6	9.1	11.0	23.3	11.3	18.1	18.8	18.8	3.7
C9_MAR1_1_647_84Z6CTCU										13.1	7.3	0.3	5.3	10.1	6.2	12.0	10.2		
C10_MAR1_1_647_84Z13CTCU	13.4	9.7	13.4					0.34	0.41	23.0	15.2	4.7	13.0	22.7	15.5	15.5	15.6	17.6	8.9
B3_MAR1_1_663_41P5CTCU	6.7	13.5	19.8	0.55	0.2					35.7	16.0	4.4	6.1	25.6	11.8	16.0	13.1	0.0	0.0
B4_MAR1_1_663_76P9CTCU	10.6	15.4	19.2	0.56	0.32					42.0	21.5	3.6	9.9	32.2	17.3	18.4	17.2	17.2	10.6

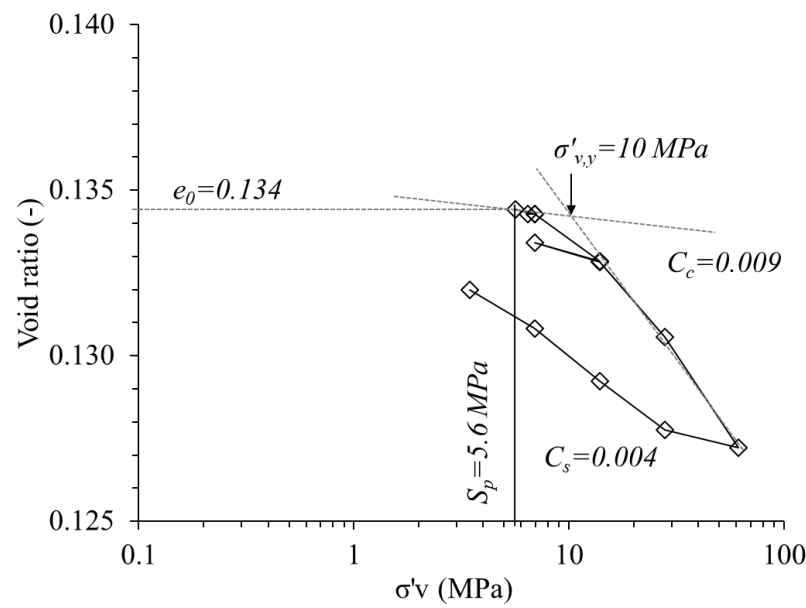
Appendix F Test results in oedometric conditions

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 (σ'_y) obtained through the Casagrande's construction, the compressibility (C_c) and the unloading (C_s) indexes. Results ordered by samples source depth.

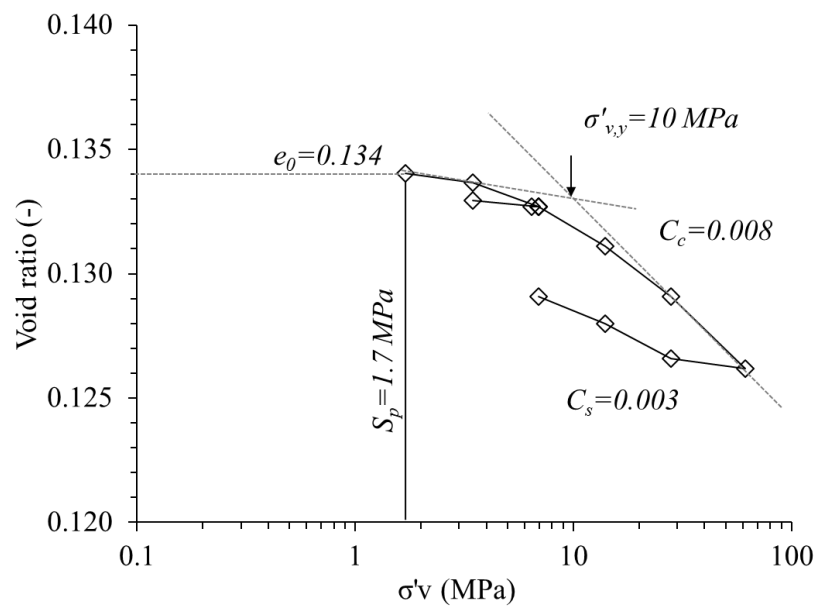
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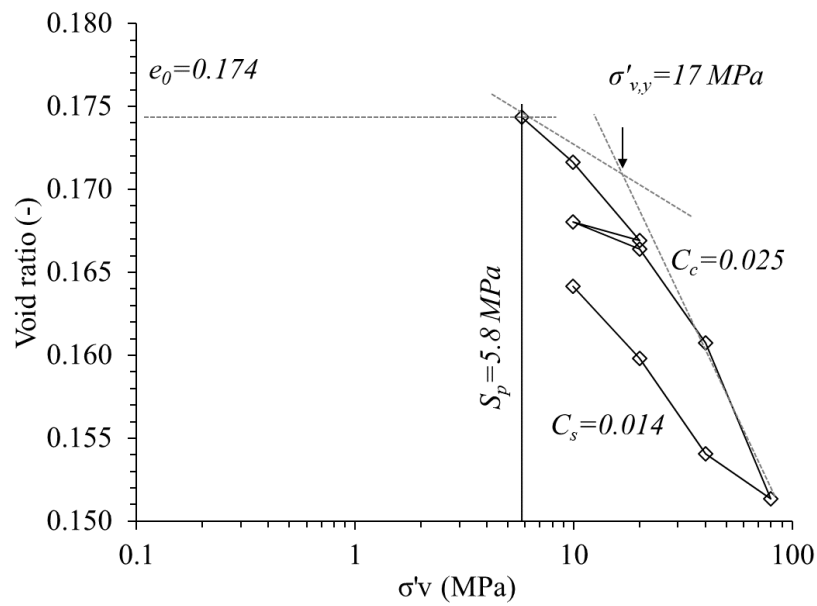
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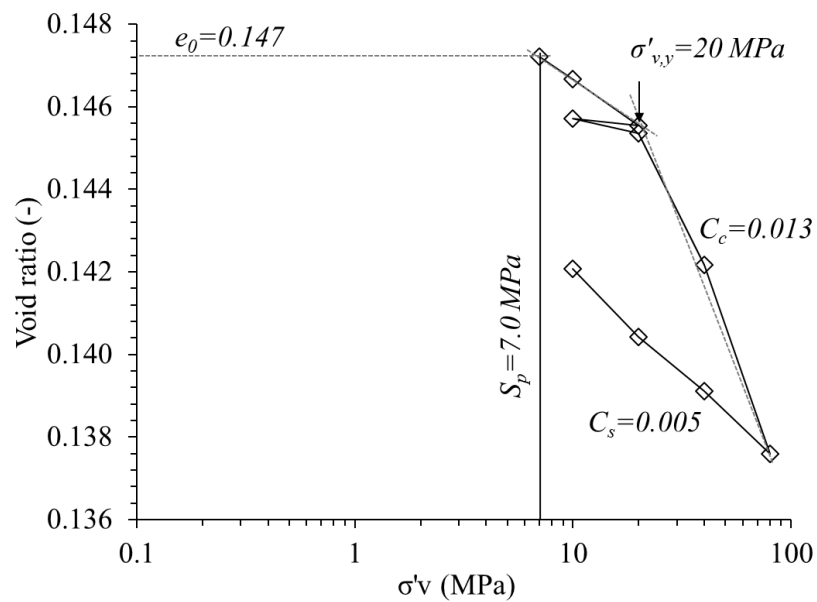
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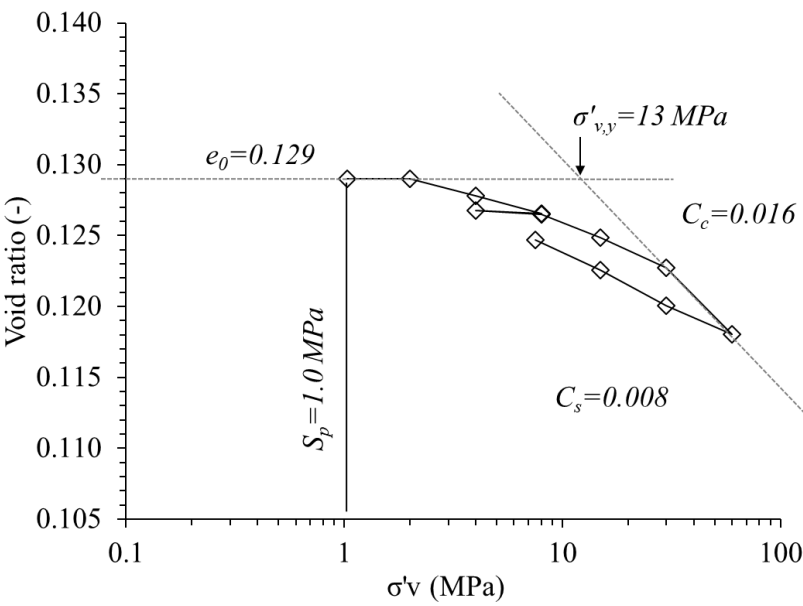
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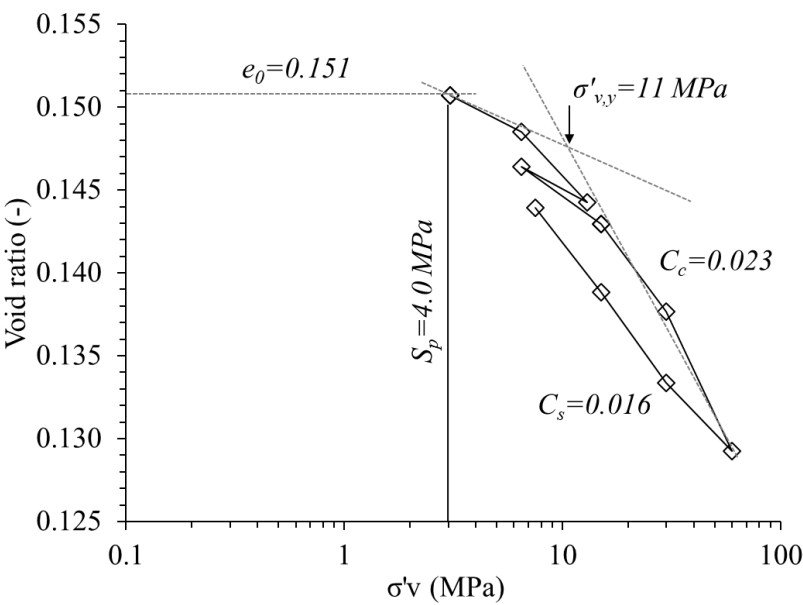
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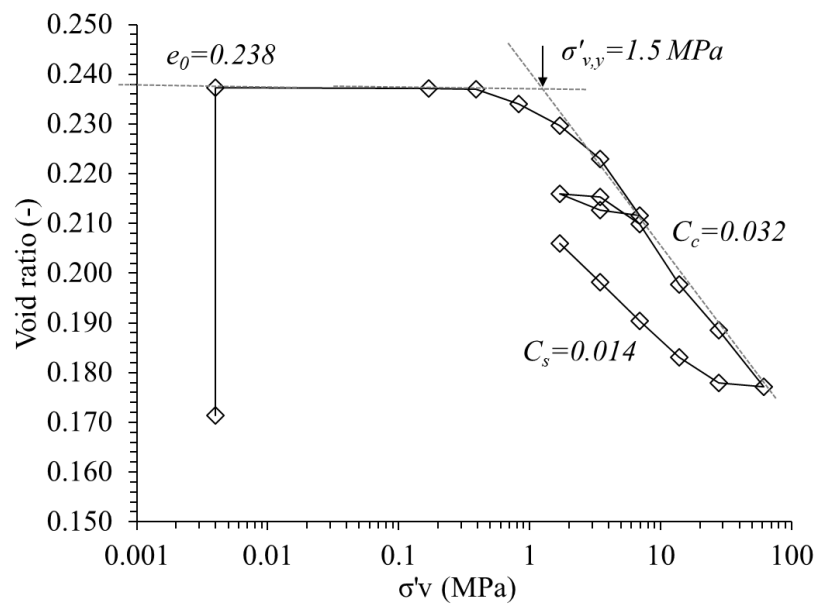
A01OED_MAR1_1



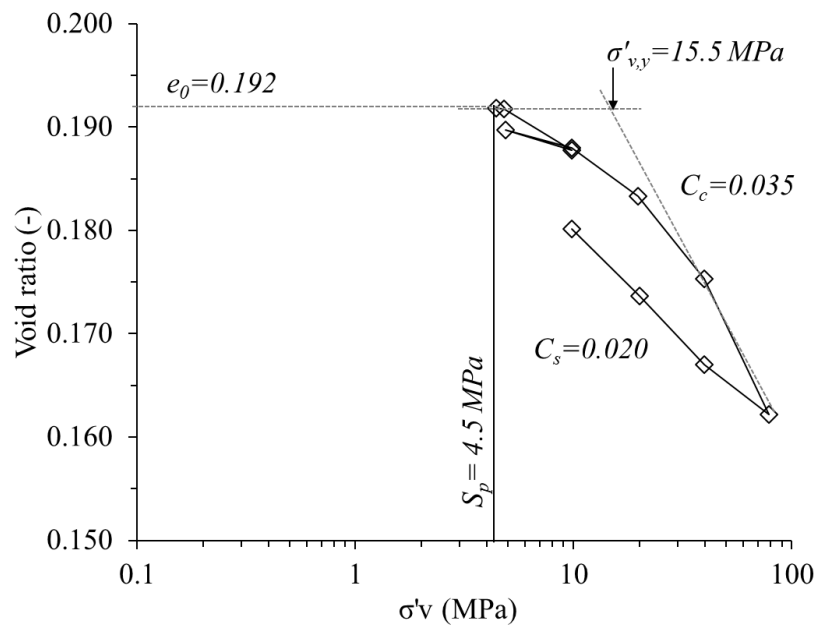
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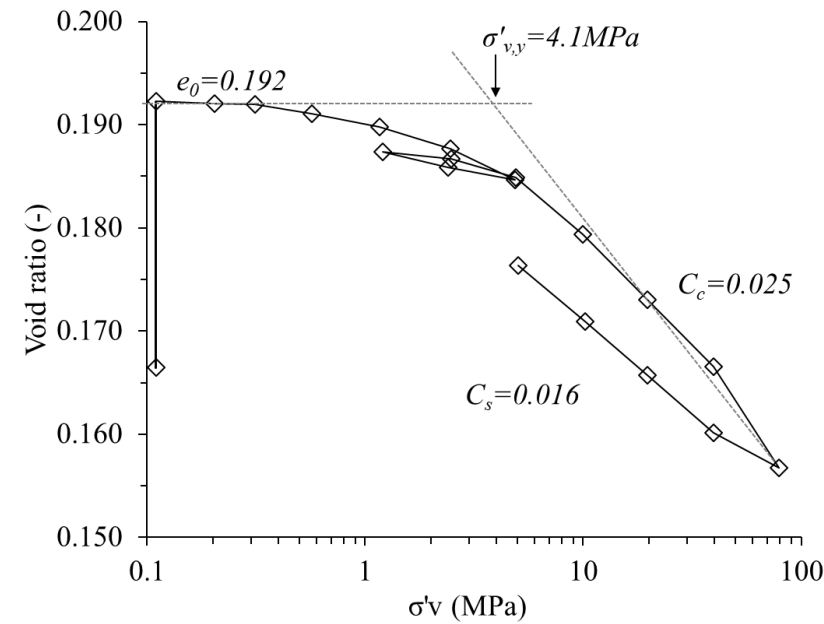
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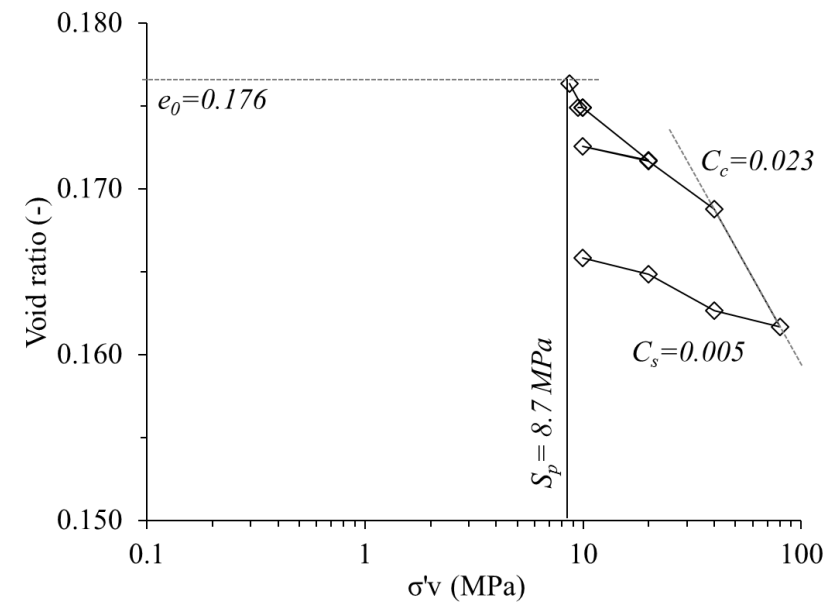
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D03OED_MAR1_1

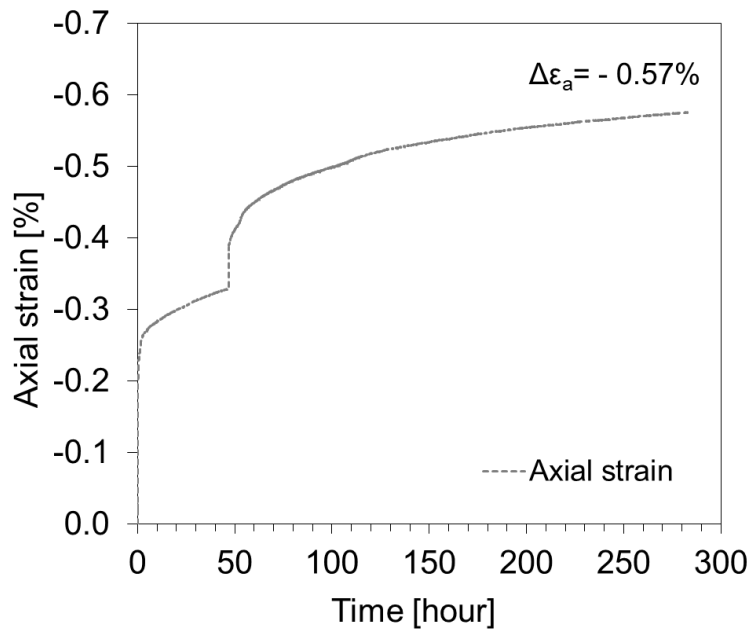


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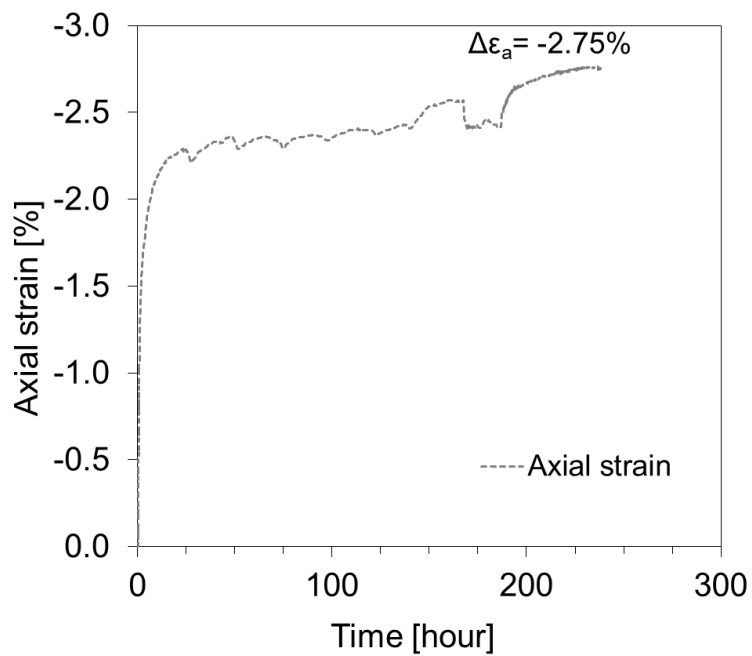


The results of the one-dimensional swelling tests are reported below.

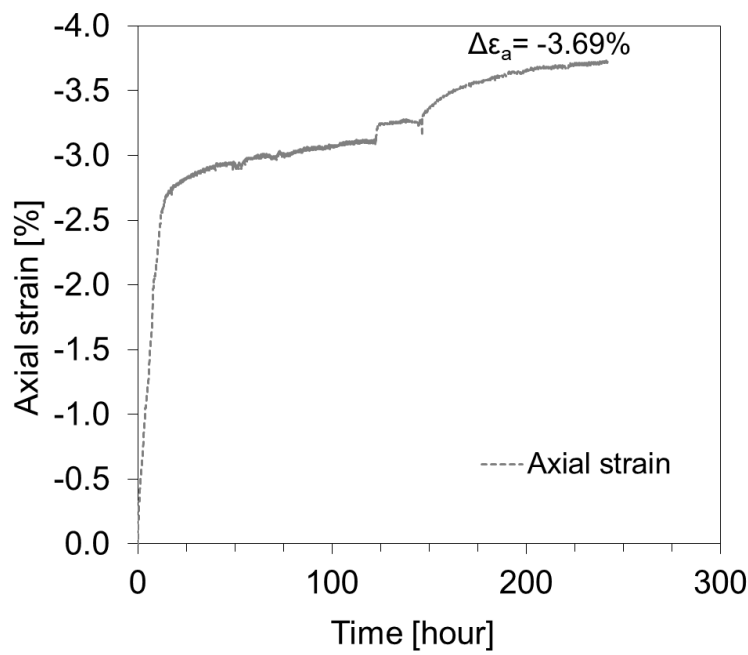
A03_OS_MAR1_1



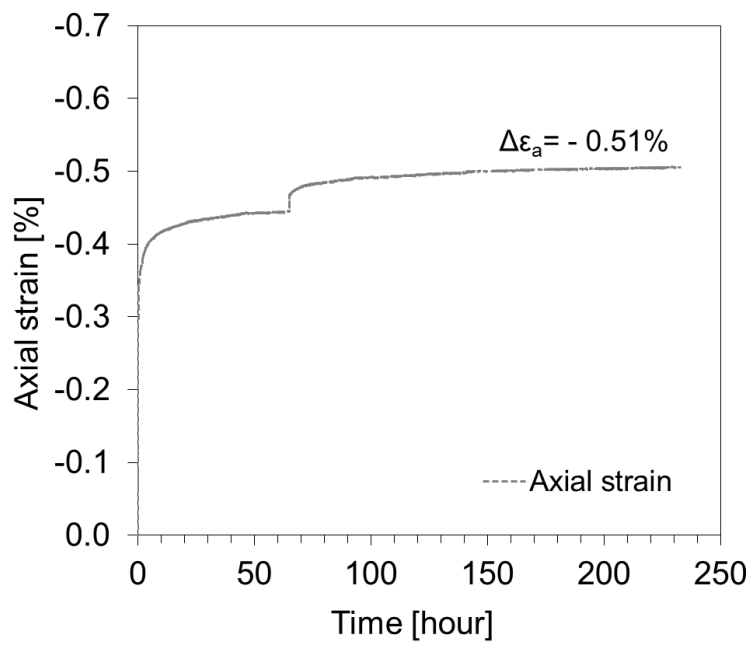
D04_OS_MAR1_1

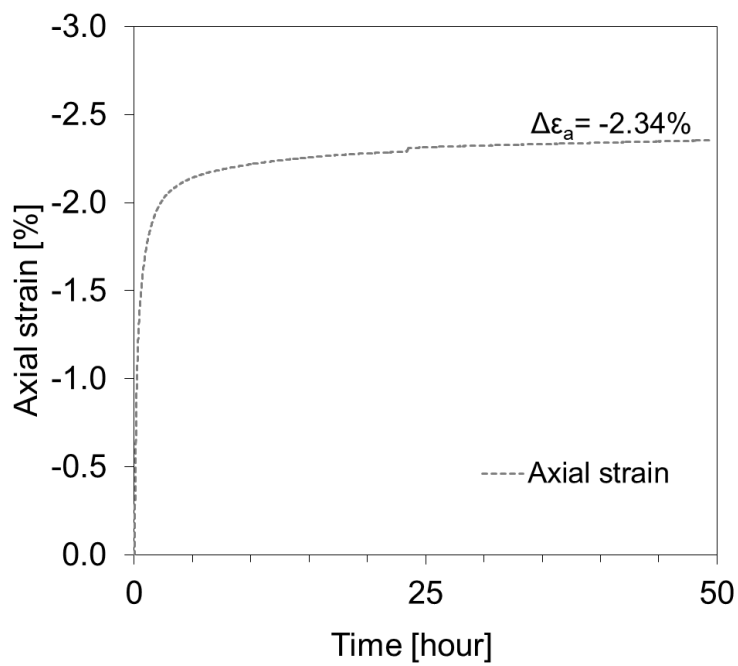
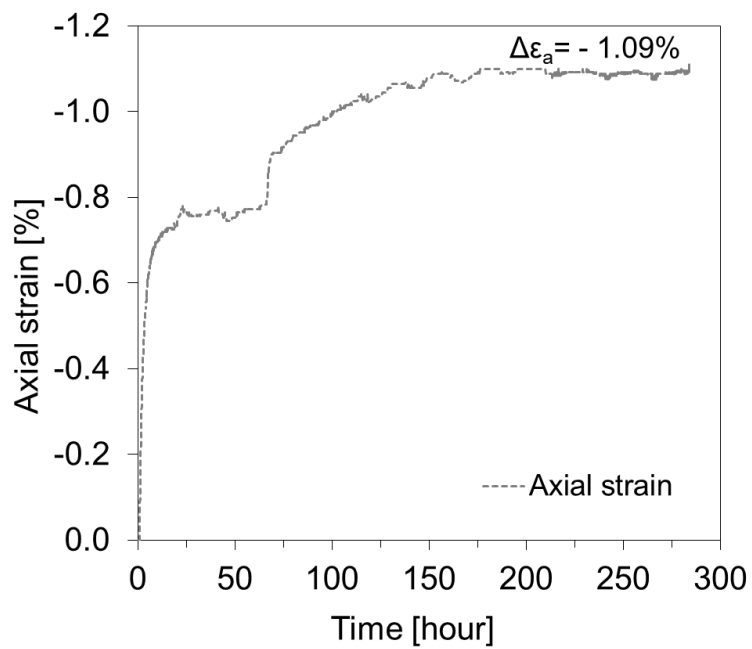


D05_OS_MAR1_1

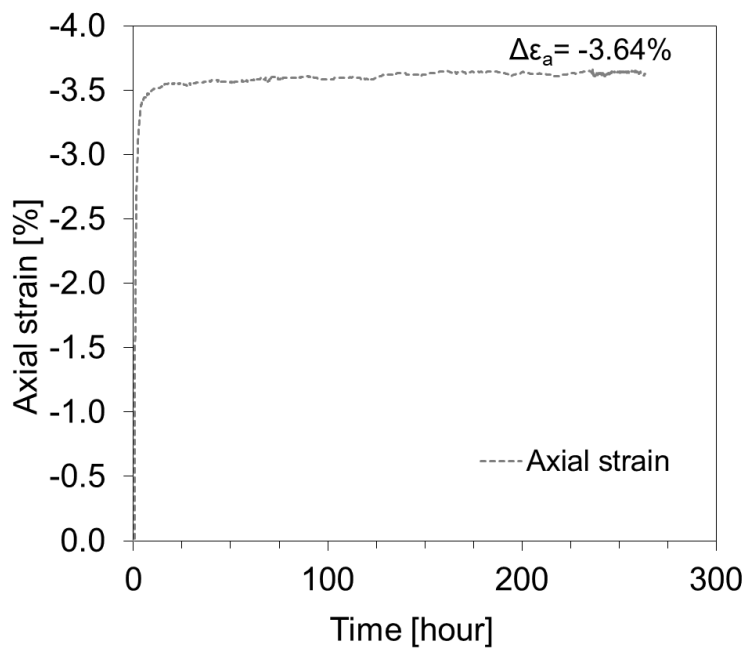


A01_OS_MAR1_1



A02_OS_MAR1_1**D01_OS_MAR1_1**

D02_OS_MAR1_1



D03_OS_MAR1_1

