Doctoral Thesis

Evaluation of the hydro-mechanical properties and behavior of Opalinus Clay

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Evaluation of the hydro-mechanical properties and behavior of Opalinus Clay

A thesis submitted to attain the degree of
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presented by

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Abstract

Argillaceous rock formations are considered as favorable host rocks for nuclear waste repositories due to their low permeability, high sorption capacity, and potential for self-sealing. Damage caused by the construction of underground excavations, however, may alter the beneficial properties of the rock and affect its natural barrier function negatively. In order to assess the performance of a repository and its safety, it is therefore essential to understand the processes that occur during and after excavation and to predict the behavior of the rock mass around man-made underground cavities. Due to the low hydraulic conductivity and complete saturation in the in-situ state, argillaceous rocks usually show a strong hydro-mechanical coupling. Thus, the characterization of the hydro-mechanical properties and behavior is of particular importance.

In Switzerland, Opalinus Clay, a Mesozoic clay shale formation, has been chosen as host rock formation for a nuclear waste repository. In this dissertation, consolidated drained and undrained triaxial tests on back-saturated specimens of Opalinus Clay were conducted to study the hydro-mechanical properties and gain a better understanding of the processes that control the rock mass behavior around an excavation in this formation. Of particular interest were:

- the characterization of the poroelastic properties,
- the determination of the drained and undrained elastic properties,
- the description of the stress-strain or pore pressure evolution under compressive loading conditions and their dependency on the stress path,
- the measurement of the effective strength properties.

To highlight the importance of back-saturating the specimens and to deepen the understanding of the impact of suction (or partial saturation) on the geomechanical properties of Opalinus Clay, a preliminary study was conducted. Specimens were equilibrated to different levels of relative humidity to establish the water retention characteristics, the influence of suction on ultrasonic p-wave velocity and geomechanical properties (e.g. Young’s modulus, Poisson’s ratio, onset of dilatancy, unconfined compressive strength, and Brazilian tensile strength). The results of this study showed that suction has a significant influence on these properties. Both stiffness and strength, for example, increased significantly (by a factor of 2-3) with increasing suction. In addition to the importance for the performance of a nuclear waste repository during the open-drift and long-term phase, this influence also emphasizes the need for tests on back-saturated specimens to characterize the material properties and behavior for in-situ conditions before and during an excavation.

In order to back-saturate the specimens, a multi-stage testing procedure is necessary. Before the actual shearing phase, a saturation and a consolidation phase are required. For the purpose of elaborating a testing procedure that allows the determination of representative material properties, a review of the major key aspects and challenges associated with laboratory testing of low permeable clay shales was performed. Theoretical background for each stage (i.e. saturation, consolidation, and shearing) is given.
and the testing procedure was applied to a series of Opalinus Clay specimens to illustrate the applicability.

The specimens were back-saturated with the help of elevated back pressure, consolidated at effective confinements between 0.5 and 16 MPa and subsequently sheared under drained or undrained conditions using a standard triaxial stress path. P-specimens (loaded parallel to bedding) and S-specimens (loaded normal to bedding) were used to characterize the influence of the anisotropy of the material. The measured pore pressure response during undrained tests and the effective geomechanical properties were additionally analyzed with respect to their dependency on the confinement.

The results show that Skempton’s pore pressure parameter $A$ and $B$ and the Young’s moduli depend on both the confinement and the anisotropy. Additionally, a change from overconsolidated to normally consolidated behavior was observed with increasing confinement. Specimens consolidated at lower effective stresses (i.e. heavily overconsolidated specimens) show a dilatant behavior in the pre-peak region and a significant post-failure stress drop. Specimens consolidated at higher effective stresses (i.e. slightly overconsolidated to normally consolidated specimens) show compaction from initial loading until post-peak and a brittle-ductile post-failure behavior. The changes could be correlated to a non-linear appearance of the effective peak strength envelope. The dilatant structure of the material is proposed as possible explanation for this non-linearity.

To investigate the influence of the stress path on the hydro-mechanical behavior of Opalinus Clay, a second series of specimens was subjected to different stress paths approximating a tunnel excavation. Three different stress paths were applied to both P- and S-specimens: a two-dimensional stress path with isotropic initial stress conditions (i.e. pure shear compression), a two-dimensional stress path with anisotropic initial stress conditions, and a three-dimensional stress path. An influence of the stress path and the anisotropy on the hydro-mechanical response of the specimens was observed. The application of the two-dimensional stress paths on S-specimens revealed positive excess pore pressure whereas P-specimens showed negative excess pore pressure. Dilation associated with yielding was observed for all specimens. The results of the tests using the three-dimensional stress path conceptually reproduced the pore pressure evolutions usually observed during tunnel excavations. With regards to the effective peak strength of the specimens, no significant difference compared to the standard triaxial tests was found. However, the state of failure is affected by the transversal isotropy, which is related to the different hydro-mechanical response of the specimens and is reflected by different effective stress paths. S-specimens fail at lower effective stresses compared to P-specimens.

This dissertation aims at a systematic, experimental characterization of the hydro-mechanical properties and behavior of Opalinus Clay. The findings, including (poro)elastic properties, stress-strain behavior in combination with the pore pressure response, the dependency of these properties and behavior on the confinement and the stress path, and the non-linear effective peak strength failure envelope, present a valuable contribution to the geomechanical understanding of the material itself and the interpretation of observations around test galleries at the Mont Terri Underground Rock Laboratory. The newly gained
understanding of the hydro-mechanical properties and behavior of Opalinus Clay contributes to the development, the implementation and the validation of new constitutive models which are able to adequately represent and predict the short- and long-term behavior of the rock mass around underground excavations in the Mont Terri Underground Rock Laboratory. Furthermore, such a constitutive model would substantially add value to the geotechnical and safety assessment of a nuclear waste repository in Opalinus Clay.
Zusammenfassung


Opalinuston, ein mesozoisches Tongestein, wurde in der Schweiz als Wirtsgestein für ein Tiefenlager für radioaktive Abfälle bestimmt. Zur Untersuchung seiner hydro-mechanischen Eigenschaften und um ein detaillierteres Verständnis für die Prozesse während des Tunnelvortriebes zu erhalten, wurden in der vorliegenden Doktorarbeit konsolidierte, drainierte und undrainierte Triaxialversuche an rückgestützten Proben durchgeführt. Im Fokus standen dabei:

- das Bestimmen der poroelastischen Eigenschaften,
- die Beschreibung des drainierten und undrainierten elastischen Verhaltens während des Scherens,
- die Charakterisierung des Spannungs-Dehnungs-Verhaltens und der Porenwasserdruck-Entwicklung unter kompressiver Differentialspannungserhöhung,
- das Ermitteln der Spitzenfestigkeit des Opalinustons.

Diese Eigenschaften wurden auch auf ihre Abhängigkeit gegenüber der Einspannung und des Spannungspfades hin untersucht.

In einer Vorstudie wurden mehrere Prüfkörper in Exsikkatoren mit verschiedenen relativen Luftfeuchtigkeiten ins Gleichgewicht gebracht. Damit konnte der Einfluss von partieller Sättigung auf die p-Wellengeschwindigkeit und die geomechanischen Eigenschaften (Elastizitätsmodul, Poissonzahl, einaxiale Druckfestigkeit und indirekte Zugfestigkeit) gezeigt werden. Mit zunehmender Saugspannung (bzw. zunehmender Entsättigung der Probe) nehmen beispielsweise sowohl die Steifigkeit als auch die Festigkeit signifikant zu (um einen Faktor von 2-3). Dies ist einerseits relevant in Bezug auf das längerfristige Verhalten des Gesteins um einen Lagerunstollen. Andererseits impliziert es die Notwendigkeit einer kompletten Sättigung der Probe zur Bestimmung von hydro-mechanischen Materialeigenschaften, die als repräsentativ für die Beurteilung des Verhaltens des Opalinustons vor und während des Tunnelvortriebs herangezogen werden können.

Die Prüfkörper wurden mithilfe von erhöhtem Wasserdruck gesättigt und bei effektiven Einspannungen zwischen 0.5 und 16 MPa konsolidiert, bevor die axiale Spannung unter gleichbleibender radialer Spannung erhöht und der Prüfkörper somit geschert wurde. Um den Einfluss der Anisotropie des Materials auf die Eigenschaften und das Verhalten untersuchen zu können, wurden sowohl P-Proben (Hauptbelastungsrichtung verläuft parallel zur Schichtung) als auch S-Proben (Hauptbelastungsrichtung verläuft senkrecht zur Schichtung) getestet. Zudem wurde der Einfluss der Einspannung auf die Porenwasserdruck-Entwicklung während der undrainierten Versuche und auf die effektiven geomechanischen Eigenschaften studiert.


beobachtet wird, konzeptionell nachgebildet werden. In Bezug auf die Spitzenfestigkeit konnte kein signifikanter Unterschied im Vergleich zu der vorhergehenden Testreihe (d.h. den Triaxialversuchen unter Verwendung eines Standard-Spannungspfades) festgestellt werden. Jedoch wird das Eintreten des Bruches stark durch die Anisotropie des Opalinustons beeinflusst und kann auf das unterschiedliche hydro-mechanische Verhalten der P- und S-Proben zurückgeführt werden, welches sich in den unterschiedlichen Spannungspfaden zu erkennen gibt. Obwohl im Ganzen gesehen ein gemeinsames Bruchkriterium festgestellt wurde, brechen S-Proben im Vergleich zu P-Proben grundsätzlich bei geringeren effektiven Spannungen.

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1. Introduction

Radioactive waste is produced by nuclear power plants, industry, research and healthcare applications, and has to be stored safely to protect the biosphere. In Switzerland, the waste is nowadays kept in secure containers in halls at ground level. However, this is only an interim solution. Based on today’s knowledge, deep geological disposal sites are considered the safest methodology for the long-term storage of nuclear waste. The design concept of a repository in Switzerland relies on a multi-barrier system, consisting of engineered barriers (from inside to outside: canisters, buffers, backfill material, and seals) and natural geological barriers (host rock and surrounding formations). For the latter, argillaceous rock formations are often considered due to their low permeability, high sorption capacity, and the potential for self-sealing (i.e. the ability to close existing or excavation induced fractures by hydro-chemical and hydro-biological processes (Bernier et al. 2007)). In Switzerland, Opalinus Clay, a Mesozoic clay shale formation, has been chosen as host rock for the disposal of high-level nuclear waste (BFE 2011).

To investigate the hydrogeological, geochemical and rock mechanical characteristics of Opalinus Clay, the Mont Terri Project has been established in 1996. For its purpose, an underground rock laboratory (URL) has been built within the Jura Mountains near the village of St-Ursanne in Canton Jura. Several galleries and niches have been excavated, starting from a motorway reconnaissance tunnel at a depth of approximately 300 m below surface. The underground facilities have been built for research purpose only, and a significant number of experimental studies has been conducted so far, both in situ and in laboratories. ETH Zurich acts as a research partner of the Swiss Federal Nuclear Safety Inspectorate (ENSI) which is a formal partner of the Mont Terri Consortium. Within this relationship, the “HM-experiment” was released. It generally aims at characterizing the hydro-mechanical properties and behavior of Opalinus Clay and builds the framework of this thesis.

1.1 Problem definition and motivation

Opalinus Clay in its natural state shows very favorable conditions for a host rock for nuclear waste disposal (e.g. low permeability, good retention capacity of radionuclides due to its high surface area). However, these beneficial properties may be altered due to the damage (inelastic deformations) induced by the construction of an underground repository and the natural barrier function of the host rock can be affected negatively (Blümling et al. 2007). For the construction and design of underground excavations for repository purposes, it is therefore important to study the behavior of the intact rock and the rock mass in great detail. Opalinus Clay shows a strong hydro-mechanical coupling caused by its low hydraulic conductivity and complete saturation in its in-situ state. Thus, for assessing the performance of an underground excavation in Opalinus Clay during the excavation phase (short-term, hours) and the open-drift phase (0-1 year, Nagra 2016), one has to consider how mechanical changes affect the hydraulic behavior of the rock (mass) and vice versa. Since no waste will be emplaced until the end of
the open-drift phase and thus mechanisms such as thermal processes (e.g. heat release from the radioactive material) and or chemical interactions (e.g. between pore water and canisters) do not have to be taken into account. Furthermore, it can be assumed that the short-term and the open-drift phase represent the time of the maximum pressure and stress gradient.

1.1.1 Hydro-mechanical behavior around an excavation

Due to the low hydraulic conductivity of Opalinus Clay ($10^{-12}$-$10^{-14}$ m/s, Marschall et al. 2004) and the high advance rate (up to 6 m/day, Nagra 2016), seepage flow is insignificant during the excavation phase and the short-term response of the rock mass is essentially undrained (i.e. the mass of the pore fluid within the porous medium remains constant upon the application of a stress increment) (Anagnostou and Kovári 1996). Thus, induced stress perturbations and associated mechanical reactions of the rock mass during the excavation of underground drifts cause changes in pore pressure around the tunnel, which in turn influences the mechanical processes. A review of pore pressure measurements reported by various authors (e.g. Neerdael et al. 1999, Verstricht et al. 2003, Corkum and Martin 2007, Yong 2007, Martin et al. 2011, Wileveau and Bernier 2008, Armand et al. 2012, Morel et al. 2013, Giger et al. 2015) revealed similarities in the pore pressure evolution during an excavation in clay shale (Wild et al. 2015, see Appendix A). An example for typical pore pressure responses is shown in Figure 1.1. In all observations, the pore pressure increases as the tunnel face approaches the location of the monitoring sensor and culminates in a peak value shortly before the tunnel face passes. As the tunnel face passes the sensor, the pore pressure drops. Atmospheric pressure has been recorded by sensors close to the tunnel wall. For most of the sensors located farther away, the pore pressure leveled off at values below the initial pore pressure but higher than atmospheric pressure.

![Figure 1.1: Example of a typical pore pressure response observed in the vicinity of an excavation at the Mont Terri URL (data from Martin et al. 2011) (figure from Wild et al. 2015).](image-url)
The fundamental geomechanical processes underpinning these pore pressure responses around underground excavations are not well understood. In general, pore pressure changes are solely associated with changes in volumetric strain. Thus, various factors, including the poroelastic response, the inelastic and dilatant yielding behavior, the stiffness and strength of the material, the in-situ stress conditions, as well as the stress path, can contribute to the pore pressure evolution and create challenges for its interpretation. Based on analytical solutions and conceptual numerical models, Wild et al. (2015) concluded that three-dimensional models are needed to cover the full pore pressure evolution during excavation advance. Such models suggest that the typically observed pore pressure responses are related to an anisotropic in-situ stress field (i.e. $\sigma_1 > \sigma_2 > \sigma_3$), the transversal isotropic stiffness of clay shales, and the orientation of the excavation with respect to the plane of isotropy of the material or the in-situ stress state. These factors mostly determine the magnitude of the pore pressure peak occurring as the tunnel face approaches but can also account for the drop in the pore pressure after the face passes the monitoring section. Additionally, dilatancy accompanying failure can lead to the observed decrease in pore pressure, especially in cases where the pore pressure drops to atmospheric pressure, which indicates that new fractures created a connection to the tunnel environment.

The state of the rock mass after excavation with respect to the effective stresses and damage sets the basis for the evolution during the open-drift phase (which includes pore pressure dissipation and associated processes like consolidation and swelling) and it is therefore important to characterize, understand and predict the induced pore pressure perturbation. The models shown in Wild et al. (2015) enabled reproducing the observed pore pressure response conceptually and revealed important factors that are relevant for understanding the pore pressure evolution in low permeable clay rocks. They showed that the hydro-mechanical behavior of the material surrounding the excavation is strongly influenced by the characteristics of the material. Hence, to fully understand the processes that are relevant for the pore pressure response around a tunnel and to being able to quantitatively back-calculate the observations, it is important to quantify the basic properties of the material (i.e. effective strength and poroelastic properties) representative for the in-situ conditions and study their dependency on the stress path.

1.1.2 Development of an excavation damage zone

As indicated by a drop in pore pressure to atmospheric pressure close to the tunnel (see above), fractures can form during tunnel excavation due to induced stress redistribution. These fractures add up to the so-called excavation damage zone (EDZ). Within this fractured zone, shear and extensional failure are typically observed (Bossart et al. 2002, Marschall et al. 2006, Marschall et al. 2008, Nussbaum et al. 2011, Yong 2013, Thoeny 2014, Kupferschmied et al. 2015). The extent of the EDZ in Opalinus Clay in the short term is strongly dependent on the orientation of the tunnel with respect to the bedding plane orientation and the in-situ state of stress, the excavation size, the strength and stiffness of the material, the existence and frequency of natural fractures, and the excavation method (Bossart et al. 2002, Yong
Kupferschmied et al. (2015) conducted an overcoring experiment to characterize the time-dependent evolution of the damage zone around a borehole drilled parallel to bedding in the Opalinus Clay at the Mont Terri URL. The borehole was left unsupported for 12h and was then impregnated with fluorescent resin to capture the fractures around it. Afterwards, this pilot-borehole was overcored, the core was cut into slices and analyzed macroscopically under UV-light and microscopically with thin sections under a UV-light embedded microscope. A short-term borehole damage zone (BDZ) with an extent of about half the radius of the pilot borehole was observed (Figure 1.2). It is characterized by tangential shear fractures on opposing sides of the borehole, emanating from the borehole mostly in one direction only (Figure 1.2a). Starting from these bedding parallel shear fractures, extensional type fractures and secondary shear fractures were observed, propagating towards the borehole (Figure 1.2b). The observations revealed that initial fracturing occurs as shear fracturing along the bedding planes and is therefore strongly affected by the presence of them as well as their orientation with respect to the in-situ stress state. Similar findings have been reported by Thoeny (2014) who analyzed the structural and kinematic influence of natural fractures on the EDZ around a mine-by section at the Mont Terri URL. In cases where the bedding plane shear was kinematically free and the rock mass was only sparsely faulted (i.e. 0-1 fault/m²), shearing along the bedding planes and extensional brittle failure were observed. In cases where the bedding shear was kinematically constrained, extensional fracturing was found to be the dominating fracture mechanism. Furthermore, Kupferschmied et al. (2015) concluded that the localized and relatively small short-term BDZ is related to the effective stress state that develops due to stress redistribution during drilling (see also above). Unloading and dilatant failure lead to a significant reduction in pore pressure which favors the stability of the borehole.

Figure 1.2: Section of the overcore BHM-3 photographed under UV-light (S₀ indicates the orientation of the bedding planes): a) tangential fractures propagating from the borehole in opposite directions, b) close-up view of the lower tangential fracture showing smaller extensional fractures and secondary shear fractures bending towards the borehole. (Photos N. Kupferschmied).
The behavior of the rock mass around a repository drift in clay shales during the open-drift phase, on the other hand, can be characterized by a drained behavior (i.e. dissipation of excess pore pressures). Starting from the state after excavation (including a relatively small and distinct EDZ and the corresponding effective stress state), time-dependent processes such as consolidation or mechanical swelling, creep, and environmental degradation can lead to further deformation and failure of the rock mass and extension of the EDZ (Bobet and Einstein 2008, Amann et al. 2012a, Corkum 2004, Naumann et al. 2007, Pineda et al. 2014, Möri et al. 2010, Kupferschmied et al. 2015). Kupferschmied et al. (2015) compared the initial BDZ described above to two further BDZs that developed around two boreholes that were left unsupported for 6 and 30 days respectively and were studied with the same methods. The main finding was that the BDZ extended both laterally and radially and formed a chimney-like fracture network with buckled slabs. After 6 days, the extent of the BDZ was grown to at least one, after 30 days to two borehole diameters. Kupferschmied et al. (2015) concluded that this relatively fast transition is most likely related to pore pressure dissipation that destabilizes the borehole rather than to creep or environmental degradation.

1.1.3 Shear strength of Opalinus Clay and the influence of the effective stress state

The discussion of the pore pressure evolution around an excavation and the development of an EDZ/BDZ shows that there is a need for the determination of effective rock properties (e.g. elastic properties such as Young’s modulus or pore pressure coefficients, effective peak strength, and stress-strain/pore pressure evolution) of Opalinus Clay which are representative for the in-situ condition. This is important in order to understand the processes and to be able to predict the hydro-mechanical behavior of the rock mass around a tunnel. A special focus lies on the short-term undrained behavior because, as mentioned previously, the state of the rock mass after excavation builds the basis for all subsequent phases of an underground excavation, including the open-drift and the long-term phase where further (thermo)-hydro-mechanical-(chemical)-processes set in.

Many studies have been conducted to characterize the undrained strength properties of Opalinus Clay from the Mont Terri URL (e.g. Olalla et al. 1999, Rummel and Weber 2004, Schnier 2005, Schnier and Stührenberg 2007, Lux et al. 2007, Amann et al. 2011, Amann et al. 2012b). Furthermore, Jahns (2007) and Jahns (2010) conducted drained triaxial tests to characterize the drained strength. Figure 1.3 compiles the different peak strength values obtained in the different studies. The test results are subdivided into P-specimens (for which the load is applied parallel to bedding) and S-specimens (for which the load is applied normal to bedding). In general, S-specimens tend to show slightly lower peak strength in comparison to P-specimens. However, a significant scatter can be identified with differences in peak strength of up to about 20 MPa for the same confining stress.

Variations in the results may be related to the different testing procedures used and the initial specimen properties. Specimens with diameters ranging between 17.5 and 100 mm have been utilized. Their water content varied between 3.4 and 7.9 %. Most of the tests were conducted on unsaturated (or not
specifically back-saturated) specimens (Schnier 2005, Schnier and Stührenberg 2007, Lux et al. 2007, Amann et al. 2012b). Olalla et al. (1999) tested some of the specimens with back pressure but no clear description of the saturation procedure is provided. 0.3 MPa back pressure has been applied by Rummel and Weber (2004) and 0.6 MPa back pressure has been used by Jahns (2007) and Jahns (2010). In all cases, however, saturation has not been proven after the application of the back pressure (Amann and Vogelhuber 2015). Except for the data set reported by Amann et al. (2012b), all specimens have been consolidated to a certain degree before the differential stress has been increased. The consolidation time, however, ranged between 1 and 4 days and the completeness of consolidation has, except for Olalla et al. (1999), not been confirmed by measuring for example pore pressure or strains (Amann and Vogelhuber 2015). Shearing has then been performed with constant strain rates between $10^{-5}$ and $10^{-7} \text{s}^{-1}$ under undrained conditions and between $10^{-6}$ and $10^{-7} \text{s}^{-1}$ under drained conditions. No significant difference between drained tests and undrained tests can be identified.

![Figure 1.3: Peak strength values from different tests on Opalinus Clay from the Mont Terri URL.](image)

In contrast to these consolidated tests, Amann et al. (2012b) tested specimens by means of unconsolidated undrained tests (the results of those tests are also plotted in Figure 1.3). These tests give a quick
estimate of the undrained shear strength. Pore pressures are usually not measured and the specimens are not consolidated. The confinement is increased immediately before shearing. Results from such tests on isotropic, saturated specimens result in an undrained shear strength that is independent on the confinement and such specimens therefore exhibit usually an undrained friction angle of 0°. The results of Amann et al. (2012b) for Opalinus Clay, however, indicate a bi-linear failure envelope with an undrained friction angle of 43° for confinements below 1 MPa and a lower friction angle of 11° for confinements exceeding 1 MPa. Undrained friction angles > 0° have been discussed for example by Golder and Skempton (1948), Bishop and Eldin (1950), Penman (1953), Lambe and Whitman (1979), Fredlund and Vanapalli (2002), Amann et al. (2015). Without going into details, entrapped air within the specimen, negative pore pressure developing upon dilation during shearing, or non-isotropic materials (i.e. materials for which Skempton’s pore pressure coefficient $B$ is smaller than 1) can account for undrained friction angles > 0°. Thus, as for the consolidated undrained and drained tests described before, the results seem to be strongly dependent on the initial saturation of the specimens and a more reliable interpretation would require the knowledge of the effective stress state before undrained loading.

The mean effective stress within core samples of a homogeneous, isotropic elastic, saturated material, with a compressibility of the rock matrix which is much lower than that of water, remains constant upon sampling (i.e. unloading) since the pore pressure would decrease by the same amount as the mean total stress. However, this is not true for a transversely anisotropic material like Opalinus Clay in combination with the anisotropic stress state observed at the Mont Terri URL. In this case, the decrease in pore pressure upon unloading of the core during sampling and thus the effective stress state depends primarily on the sample orientation with respect to the bedding and the principal stress components (Amann et al. 2017). Further processes such as desaturation due to gas escaping from solution upon unloading, air-entry, capillary effects, and desaturation by cavitation can contribute to the desaturation of the sample (Hight 2003, Pei 2003). Additionally, disturbances during sampling (e.g. vibration of the drilling machine, heat generation during drilling, interaction with the cooling fluid) and during storage, core-dismantling and specimen preparation (e.g. bumping during transportation, temperature change, contact with air) can lead to further modification of the effective stress state (Pei 2003). Due to the complex interaction of these effects, the specimen’s saturation degree is probably lower than the saturation degree of the in-situ rock and/or suction may act on the specimen’s surface. Thus, the effective stress state of the specimen is unknown.

The influence of moisture content on the rock mechanical properties, such as strength and deformability, have been demonstrated in various studies (e.g. Fredlund et al. 1987, Escario et al. 1989, Dyke and Dobereiner 1991, Hsu and Neson 1993, Lashkaripour and Passaris 1995, Rahardjo et al. 1995, Cui and Delage 1996, Valès et al. 2004, Feuerharmel et al. 2006, Chae et al. 2010, Wild 2010, Nam et al. 2011, Schnellmann et al. 2013). An increase in strength or stiffness with decreasing moisture content has been consistently observed and has been associated with capillary forces (i.e. total suction) acting in a partly
saturated system (Fredlund et al. 1978, Schmitt et al. 1994, Birle 2012). Especially for rock types with predominantly micro-pores, such as clay shales, a small decrease in saturation from an initially saturated state may lead to considerable suction and thus, the mechanical properties of partially saturated specimens may not be representative for the in-situ properties. Since the effective stress state of the specimens of Opalinus Clay tested so far has not been quantified and therefore remains unknown, the test results shown in Figure 1.3 are ambiguous and leave room for interpretation. Therefore, triaxial tests on fully saturated specimens are required to characterize the basic hydro-mechanical properties underlying the in-situ conditions and behavior of the rock mass during the excavation phase.

1.2 Study objectives

The scope of this thesis is to obtain a better understanding of the hydro-mechanical processes that control the rock mass behavior around an excavation in Opalinus Clay. Of particular interest is the quantification of effective rock mechanical properties and the hydro-mechanically-coupled behavior of Opalinus Clay on the laboratory scale. The main focus of this thesis is on a systematic experimental analysis of poroelastic properties such as the Skempton’s pore pressure coefficients, the drained and undrained elastic properties, the stress-strain behavior under drained and undrained compressive loading conditions, the bulk pore pressure evolution during compressive loading and its dependency on the stress path, as well as the effective strength properties. The project aims are associated with hydro-mechanically coupled phenomena relevant for the excavation phase (hours-days) and open-drift phase (0-1 year) of a future nuclear waste repository.

1.3 Thesis organization

This thesis is organized in seven main chapters with the aim of comprising the topics associated with the investigated problem. Relevant background information for the research of this thesis is given in Chapter 2, including a discussion of clay shale classification systems, the description of the Opalinus Clay at the Mont Terri URL (i.e. a description of the material examined in this study, a review of the in-situ stress state, and a summary of the sampling for the laboratory specimens). Additionally, the core logs of the used samples are shown in Appendix B. Chapter 3 is dedicated to the study dealing with the influence of partial saturation (or suction) on the mechanical and petro-physical properties of Opalinus Clay. This work has been conducted at the beginning of the doctoral study and shows the importance of the knowledge of the initial effective stress state for the interpretation of laboratory testing results. At the same time, it gives the motivation for testing fully back-saturated specimen, which requires a multi-stage testing procedure.
The details of the testing procedure for the consolidated drained and undrained triaxial tests on back-saturated specimens and the arising challenges of such laboratory experiments on low permeable clay shales are given in Chapter 4.

Chapter 5 and Chapter 6 analyze and discuss the results of the laboratory tests. The results and discussion of the standard triaxial tests are given in Chapter 5. Especially the influence of confinement and anisotropy on the pore pressure response and the effective geomechanical properties of Opalinus Clay are examined. Chapter 6 presents the outcome from the triaxial tests utilizing various stress paths approximating an excavation of a circular tunnel. The influence of the stress path on the hydro-mechanical behavior of Opalinus Clay is discussed. More detailed data of the consolidated drained and undrained triaxial tests can be found in Appendix C.

Finally, Chapter 7 summarizes the main conclusions of this study and gives recommendations for future work.

1.4 References


2. Background

2.1 Clay shale classification

Fine-grained, inorganic, clastic sedimentary materials are most common at Earth’s surface. The classification of such material, however, remains ambiguous until today and there is no generally accepted classification (Stead 2016). Some researchers define all argillaceous sediments (i.e. fine-grained materials, composed predominantly of clay and silt-sized particles and clay-minerals) including claystone, siltstone, mudstone as shales, whereas for others consider shales as a subgroup of mudstone/mudrock (Morgenstern and Eigenbrod 1974). Furthermore, there is a difference in usage of the term shale between geologists and engineers, which has its reason in the different work fields (Farrokhrouz and Asef 2013). Geologist mostly work with outcrops where they find weathered and more soil-like argillaceous material and are more concerned with the descriptive characterization of the rock/soil. Geotechnical experts, working at middle depth ranges, deal with argillaceous material which can be considered more like a soft rock. Engineers, and especially petroleum engineers, are more concerned with a deeper depth range where those materials are more like a weak or even hard rock. These different points of view, but also the fact that argillaceous materials are highly transitional in their physical properties and behavior with state and time and cannot be clearly classified as either soil or rock (Deen 1981, Botts 1986), lead to immense confusion in the literature about classification of such materials.

A review of the early history of the term shale is given by Tourtelot (1960). The term probably is of Germanic origin (from “schälen” – to peel) and was used by English miners to describe laminated clayey rocks. Early attempts of classification of shales are mostly based on geological considerations including mineralogy, grain size, color, degree of compaction, type and degree of bonding, and breaking characteristics (Deen 1981). The main objective of such geological classification schemes is the determination of the geological history of the sedimentary material (Botts 1986). Wentworth (1922) distinguished between argillaceous materials (shale/mudstone) and the rest of clastic sedimentary rock based on grain size. The boundary was arbitrarily set to 0.0625 mm. Lewan (1979) proposed a classification based purely on textural and compositional criteria. He subdivided very fine grained rocks into mudstones and shales according to their content of microscopic material (< 5 μm). The nomenclature further includes root names (i.e. claystone, marlstone, micstone, and mudstone) based on their silicate fraction. The root names can be preceded by a primary adjective (indicating specific mineralogical information) which again may be preceded by a nominal adjective (indicating for example bedding structures, fissility, color, or fossil content). Classifications schemes that are based on grain size, however, may be misleading since it is difficult to separate the individual grains of argillaceous materials from each other. Furthermore, there is a lack of the information about bonding or lithification of the material and thus does not convey anything about their durability (Gens 2013). Fissility (the ability
of the material to split into more or less parallel planes), for example, is an important characteristic of a material, as it provides natural planes of weakness, increases the permeability of the bulk material and the surface area where weathering agents can act and thus enhances degradation (Botts 1986). Twenhofel (1939) extended the previously considered parameters and created a classification scheme based on composition, the degree of induration, and the level of metamorphism (Figure 2.1). Silt- and claystones are distinguished from silt and clay based on their higher degree of induration. Furthermore, a silt- or clay-stone can only be classified as shale if lamination or fissility is present.

Twenhofel (1939) extended the previously considered parameters and created a classification scheme based on composition, the degree of induration, and the level of metamorphism (Figure 2.1). Silt- and claystones are distinguished from silt and clay based on their higher degree of induration. Furthermore, a silt- or clay-stone can only be classified as shale if lamination or fissility is present.

**Figure 2.1: Classification scheme according to Twenhofel (1939).**

<table>
<thead>
<tr>
<th>unindurated</th>
<th>indurated</th>
<th>after incipient metamorphism</th>
</tr>
</thead>
<tbody>
<tr>
<td>Silt</td>
<td>Siltstone</td>
<td>Argillite</td>
</tr>
<tr>
<td>Clay</td>
<td>Claystone</td>
<td>metamorphic equivalent</td>
</tr>
<tr>
<td>+ H₂O = mud</td>
<td>+ fissility = Shale</td>
<td></td>
</tr>
<tr>
<td>mudstone</td>
<td>slate</td>
<td></td>
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</tbody>
</table>

The classification of Ingram (1953) also relies on the predominance of silt or clay minerals and includes fissility (Figure 2.2). Starting from mud-/silt-/clayrock where no connotation as to breaking characteristics can be made, the sedimentary rocks with more than 50% clay or silt minerals can be divided into mud-/silt-/claystone or mud-/silt-/clayshale depending on their fissility. The individual rocks can further be described by their breaking types: massive (blocky fragments without any preferred direction), flaky-fissile (splitting along irregular surfaces parallel to bedding into uneven flakes/thin chips/wedge like fragments with a length smaller than 7.6 cm), or flaggy-fissile (fragments have a length which is many times greater than their thickness and two essentially flat sides which are approximately parallel). Furthermore, Ingram (1953) tried to correlate properties such as color, carbonate content,
organic matter, or cementing agents to the type of fissility. Folk (1974) refined the classification scheme of Ingram (1953) and included a more precise declaration of the correspondent clay or silt content (Figure 2.2). Similar classification based on layering properties like fissility or stratification are given for example by Stow (1980) and Spears (1980).

Mead (1936) defined shales as “fine-grained sedimentary rocks which were originally deposited in the form of clay or mud and have undergone various degrees of solidification and lithification by processes of compaction, crystallization and cementation”. For him, cementation is the major parameter for further classification of shales. He subdivided shales into compaction shales which experienced some consolidation by compaction and therefore show only little or no cementation and cemented shales which have experienced maximum compaction resulting in a cementation. The presence of cementation implies differences with respect to bearing strength and weathering (Mead 1936). While compaction shales slake and disintegrate when immersed in water after drying, well-cemented shales don’t show this phenomenon. Shales intermediate between compacted and cemented shales build fractures along bedding planes but do not disaggregate completely. Thus, although mostly geological (based on grain size), the classification by Mead (1936) already implies some engineering behavior with respect to the behavior of shales in contact with water.

Engineering classifications aim at classifying materials according to different engineering behavior and are therefore often bound to a specific application (Botts 1986). Nevertheless, engineering classifications may be closely controlled by the geological parameters of the material (Philbrick 1969). Terzaghi (1936) proposed a classification for the purpose of investigating the shearing resistance of undisturbed clay. A distinction between 1) “soft, intact clays, free from joints and fissures” for which “the results of shearing tests are consistent with those obtained from triaxial compression tests”, 2) “stiff, intact clays, free from joints and fissure” for which, however, “the results from shearing tests appear to be incompatible with those of triaxial compression tests”, and 3) “stiff, fissured clays like 2)” expect that they show joints/fissures and break into polyhedral fragments (Terzaghi 1936).

Underwood (1967) took up the classification from Mead (1936) and subdivided shales into compaction (i.e. not cemented) or “soil-like” shales and cemented or “rock-like” shales. This classification is a first approach to the concept of rock- and soil-like behavior that has been adapted by later authors (e.g. Bjerrum 1967, Deo 1972, Wu 1991, Leddra et al. 1992, Horsrud et al. 1998, Pei 2003). Furthermore, Underwood (1967) postulates that a satisfactory engineering classification for shales should be based on significant engineering properties such as compressive strength, elastic properties, moisture, density, void ratio, permeability, swelling potential, activity ratio, etc. However, no specific scheme of evaluation with respect to these properties is given.

In the context of progressive failure of slopes in plastic clays and clay shales, Bjerrum (1967) emphasizes the importance of properties of overconsolidated plastic clays and clay shales and subdivides them into 1) overconsolidated plastic clays with weak or no diagenetic bonds, 2) clay shales with well-developed diagenetic bonds, and 3) shales with strongly developed diagenetic bonds. Bjerrum’s
statements suggests that shales with weak diagenetic bonds are more similar to overconsolidated clays, and shales with strong diagenetic bonds are more similar to rocks (Pei 2003). Thus, shales are again seen as a material somewhere between soils and rocks.

This transitional behavior was accounted for by Gamble (1971) who investigated correlations between durability of clay shales and their moisture content, liquid limit, and density. He found that the material can be grouped best when comparing the two-cycle slake durability index with the plastic index. However, no correlation to established terminology was provided. Furthermore, the classification can be criticized with respect to the usage of the plastic index. Tests to determine plasticity are inappropriate for materials like clay shales since they hardly disaggregate (Grainger 1984, Corominas et al. 2015).

A similar investigation has been conducted by Deo (1972) who created a rating system for shales to be used as embankment materials (Figure 2.3). Based on simple tests (slaking, slake durability, sulfate soundness), shales are subdivided into “rock-like”, “soil-like”, “intermediate-1”, and “intermediate-2”. Similar to Mead (1936), Morgenstern and Eigenbrod (1974) noticed a major difference in deterioration characteristics of argillaceous materials when they are exposed to relative humidity changes or immersed in water. Well-lithified, cemented shales may remain intact when kept in a moist environment, exposed to wetting-drying cycles, or when immersed in water. In contrast, compacted shales with weak diagenetic bonds may contain clay minerals that swell and diagenetic bonds that disintegrate rapidly when exposed to the same wetting-drying cycles or immersed in water. Morgenstern and Eigenbrod (1974) proposed a classification scheme based on slaking characteristics and the undrained shear strength as well as the strength loss and change in water content due to softening (Figure 2.4).

Argillaceous materials are first divided into soils (clay) or rocks (mudstone) depending on their undrained strength at natural water content (determined in an unconsolidated undrained tests with 0.3 MPa confinement). For soil-like materials, the slaking characteristics then determine if they are clay shales.

A more extensive engineering geological classification was suggested by Grainger (1984) who combined basic material characteristics which have a fundamental influence on the mechanical properties and performance of mudrocks. Based on soil and rock index parameters (grain size distribution, composition, texture, compressive strength), he first differentiated between non-mudrocks and mudrocks and then subdivided mudrocks into different subgroups according to their compressive strength, durability, plasticity, grain size distribution, anisotropy. The individual testing procedures have been adapted with respect to the material (e.g. anisotropy was expressed by the flakiness ratio for soils but evaluated from the strength ratio for durable rocks).
Figure 2.3: Classification scheme proposed by Deo (1972). \( (l_d)_d \): durability index for dry samples, \( (l_d)_s \): durability index for soaked samples, \( l_s \): soundness index.

Figure 2.4: Classification of argillaceous materials according to Morgenstern and Eigenbrod (1974). \( c_{uo} \): undrained shear strength, \( \Delta c_u \): strength loss due to water immersion, \( t_{so} \): time needed to achieve softening to 50% of \( c_{uo} \).
The overview of classifications of clay shales discussed above shows that there is no consensus on a classification scheme for argillaceous materials. Differences arise between geological and engineering classifications but also among themselves. Generally, it can be concluded that from a geological point of view, clay shale is a laminated or fissile rock which is rich in clay-size components. From an engineering point of view, clay shales are located somewhere between stiff clays and soft rocks and can change in their behavior when interacting with water leading to a transformation from a clayey rock into a soil. Thus, in the further part of this thesis, clay shales are considered as transitional materials that exhibit a high clay content, show fissility and have a certain degree of bonding. Their properties are dependent on the texture, state, composition, geological loading history. To characterize these properties and the behavior of clay shales both soil and rock mechanics principles have to be applied.

2.2 Opalinus Clay at the Mont Terri URL

2.2.1 Geological description

Opalinus Clay is a clay shale that has been deposited about 180 Mya during the middle Jurassic (Aalenian) in a shallow marine environment in the area of Southwestern Germany and Northern Switzerland (Figure 2.5). It generally can be described as dark-grey to black colored clay shales with thin sand- and carbonate layers (Wetzel and Allia 2003). The name “Opalinus Clay” was given due to the ammonites “Leioceras Opalinum” that is often found in the formation.

Figure 2.5: Paleographic situation during the early Aalenian (modified from Wetzel and Allia 2003).
At the Mont Terri URL, Opalinus Clay is part of the Mont Terri anticline which formed during the folding of the Jura Mountains. The formation shows a thickness of about 150 meter and dips approximately 45° to the south-east (Thury and Bossart 1999) (Figure 2.6). The present overburden at the Mont Terri URL ranges between 230 and 330 m (Thury and Bossart 1999) but it is estimated to have reached 1350 m in the late Tertiary (Mazurek et al. 2006).

Figure 2.6: Geological profile along the Mont Terri motorway tunnel (modified from Freivogel and Huggenberger 2003).


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<tbody>
<tr>
<td>clay minerals</td>
<td>50-80 %</td>
</tr>
<tr>
<td>illite</td>
<td>15-25 %</td>
</tr>
<tr>
<td>illite/smectite mixed layers</td>
<td>10-15 %</td>
</tr>
<tr>
<td>chlorites</td>
<td>5-15 %</td>
</tr>
<tr>
<td>kaolinite</td>
<td>20-30 %</td>
</tr>
<tr>
<td>quartz</td>
<td>10-20 %</td>
</tr>
<tr>
<td>carbonate bioclasts</td>
<td>13 %</td>
</tr>
<tr>
<td>feldspar, pyrite, organic matter</td>
<td>0-5 %</td>
</tr>
<tr>
<td>water loss porosity</td>
<td>15-19 %</td>
</tr>
<tr>
<td>hydraulic conductivity</td>
<td></td>
</tr>
<tr>
<td>parallel to bedding</td>
<td>$10^{-12}$ m/s</td>
</tr>
<tr>
<td>perpendicular to bedding</td>
<td>$10^{-14}$ m/s</td>
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</table>

Lithologically, Opalinus Clay at the Mont Terri URL can be divided into a carbonate-rich, a sandy, and a shaly facies (Thury and Bossart 1999) (Figure 2.7). In this thesis, only the shaly facies of Opalinus Clay is considered. Its mineralogical composition and some hydraulic properties are summarized in Table 2.1. The shaly facies at the Mont Terri URL mainly consists of clay minerals (50-80%) which can be subdivided into illite (15-25%), illite/smectite mixed layers (10-15%), chlorites (5-15%), and kaolinite (20-30%). Besides clay minerals, quartz (10-20%), feldspar (0-5%), pyrite (0-3%), and organic matter of continental or marine origin (0-1%) can be found (Mazurek 1998, Thury and Bossart 1999, Klinkenberg et al. 2009, Nagra 2002, Bossart 2005). Furthermore, carbonate bioclasts and fossils are
often observed (Figure 2.8). The fossils, however, are not homogeneously distributed but rather occur in lenses and the fossil content can vary along the bore core. Additionally, sandy and carbonate lenses can be found. The water loss porosity of the shaly facies at the Mont Terri URL lies between 15 and 19% (Bossart 2005, Amann et al. 2011, Amann et al. 2012, Wild et al. 2015, Wild et al. (in preparation, see Chapter 4) and its hydraulic conductivity is $10^{-12}$ and $10^{-14}$ m/s parallel and normal to bedding, respectively (Marschall et al. 2004).

Figure 2.7: Geological map of the Mont Terri Underground Rock Laboratory (modified from Nussbaum et al. 2011) including the different galleries and niches. Also plotted is a stereo plot with the mean orientations of the main fracture sets and the bedding planes. The material used in this study was taken from the boreholes BFE-A3 (drilled parallel to bedding), BHM-1 (drilled parallel to bedding) and BHM-2 (drilled perpendicular to bedding). The location of the boreholes is indicated approximately. Furthermore, the locations where stress measurements have been conducted are indicated.
Due to the compaction of the clay minerals during sedimentation and tectonic processes, Opalinus Clay shows a distinct macroscopic bedding and its physical behavior is highly anisotropic. The bedding is the most pronounced structural feature at the Mont Terri URL and dips towards southeast with an angle between 22° at the northernmost contact and 55° at the southernmost contact (Bath and Gautschi 2003) (Figure 2.7). Furthermore, three minor tectonic shears and a larger thrust fault (main fault) can be identified. For further details of the tectonic setting at the Mont Terri URL, the reader is referred to Nussbaum et al. (2011).

2.2.2 Sampling

The material utilized in this thesis, was taken from a 38 m long borehole (BFE-A3, core diameter: 76 mm) and two 25 m long boreholes (BHM-1, BHM-2, core diameter: 67.5 mm) drilled in the shaly facies of Opalinus Clay at the Mont Terri URL. The location of the boreholes is approximately indicated in Figure 2.7. BFE-A3 was drilled parallel to bedding from the FE-A-Niche using a single-tube core barrel. BHM-1 (parallel to bedding) and BHM-2 (normal to bedding) were drilled from the Gallery 08 using a triple-tube core barrel. Compressed air cooling was used for all drillings. After extracting the cores, they have been mapped and photographed, if necessary cut into pieces with a length of one meter (to fit into the core boxes), and have been immediately sealed with aluminum foil to avoid desaturation. The core boxes are stored in a cellar room at ETH Zurich. The geological and structural maps of the cores are given in Appendix B.

2.2.3 In-situ state of stress

The stress at a specific location is influenced by the topography/geomorphology, the structural and tectonic setting as well as the lithostratigraphy. From a topographical point of view, the Mont Terri URL is surrounded by several valleys: a moderately cut valley in the South East, a deeply cut valley created by the river Doubs in the South West, and another rather shallow cut valley in the North West (Enachescu 2011). The URL is tectonically located in the Mont Terri anticline. The regional tectonic
stress in the area is mainly determined by the formation of the Alps and the Jura Mountains and is characterized by a NNW–SSE to NNE–SSW orientation of the maximum horizontal stress (compressive) in the region of the Mont Terri URL (Becker 2000). Lithostratigraphically, the stiffness contrast between the different formations (limestones, sandy and shaly facies of Opalinus Clay, and marls), the distinct anisotropy of the Opalinus Clay as well as the fault zone influence the stress state. The combination of these geological effects (topography, tectonics, and lithostratigraphy) motivates the use of direct in-situ stress measurements but complicates the applicability and interpretation of them (Enachescu 2011, Doe and Vietor 2015). The state of stress at the Mont Terri URL has been studied intensively in the past 20 years utilizing different approaches.

Hydraulic fracturing is one of the stress measurement methods that has been utilized at the Mont Terri URL. In the Opalinus Clay formation, it has been applied for example in the sandy facies in the IS-Niche by Evans et al. (1999) and the “DS-experiment” by Enachescu (2011). By generating a tensile fracture in an isolated interval due to the high-pressure injection of water, the magnitudes of the minimum and maximum horizontal stresses can be determined. The magnitude of the maximum stress, however, is often questionable due to the uncertainty in tensile strength and the effects of pore pressure. Furthermore, the orientation of the maximum stress can be determined by observing the fracture direction, given that the borehole is oriented not far off the orientation of one of the principal stresses (Doe and Vietor 2015).

Evans et al. (1999) conducted hydraulic fracturing measurements in a vertical (oblique to bedding) and an inclined borehole (normal to bedding) drilled from the IS-Niche located in the sandy facies of Opalinus Clay in the southern part of the Mont Terri URL (Figure 2.7). An opening of bedding plane features has been observed in the vertical borehole by an impression packer survey conducted after injection and it was concluded that the stress normal to bedding ($\sigma_n$) is 4.2 MPa (i.e. equal to the instantaneous shut-in pressure). For the inclined borehole (normal to bedding), instantaneous shut-in pressures between 2.8 and 2.9 MPa were measured. Assuming that the bedding was opened in both boreholes (however, no satisfactory impressions that would proof the opening of the bedding were revealed from the inclined borehole), the differences in shut-in pressure between the two boreholes can be related to a heterogeneous stress field. Alternatively, assuming that the stress field is homogeneous, the difference may be explained by the presence of natural or induced fractures which cut the bedding and are more normal to the minimum stress. In this case, it can be concluded that the minimum stress cannot be greater than 2.9 MPa (Evans et al. 1999).

Further hydraulic fracturing tests were conducted as part of the “DS-experiment” (Enachescu 2011). A vertical borehole has been drilled in Gallery 04, crossing the shaly, sandy, and sandy-carbonate facies of Opalinus Clay (Figure 2.7). The main aim of the tests was to determine the orientation of the maximum horizontal stress since almost all previous hydraulic fracturing tests may have opened bedding parallel features (the opening of the bedding in the inclined borehole in the IS-niche could not have been proofed and therefore remains uncertain (Evans et al. 1999)). Measurements with an impression packer,
however, revealed distinct fractures parallel to bedding. No vertical fractures have been observed, an assumption that would be needed to determine the minimum and maximum horizontal stress. In addition to the hydraulic fracturing method, breakouts from the borehole have been observed with an acoustic televiewer log. Although the hydraulic fracturing method could not be applied in its classical way, stress indicators from hydraulic fractures, breakouts and the impression packer interpretations have been taken to constrain the stress tensor. These indicators suggest a maximum principal stress ($\sigma_1$) of 8.6 MPa with an orientation (trend/plunge) of 033/00, an intermediate principal stress ($\sigma_2$) of 6.7 MPa oriented 123/70, and a minimum principal stress ($\sigma_3$) of 3.9 MPa with an orientation of 303/20 (Enachescu 2011).

In the framework of the “IS-A experiment”, the in-situ stress state in the IS-Niche located in the sandy facies (Figure 2.7) has been measured by using the undercoring/underexcavation method (Bigarré 1996, Bigarré 1997, Cottur 1997, Homand et al. 1997, Bigarré and Lizeur 1997). The same method has been applied by Bigarré (1998) in the “ED-B experiment” in the shaly facies around Gallery 98 (Figure 2.7). The principle of the undercoring/underexcavation method is the measurement of strains caused by the excavation of a rock mass close to the sensor locations and their conversion into stresses by using stress-strain relationships for the rock. In the “ED-B experiment”, the excavation of Gallery 98 was monitored in boreholes that have been drilled from the safety gallery before. However, the measurements were not successful. In the case of the “IS-A experiment”, a borehole was drilled and the related strains were measured in nearby boreholes. For calculation of the stresses from the strain measurements in the IS-Niche, a linear elastic transversal isotropic behavior with Young’s moduli of 12.2 and 4.1 GPa parallel and perpendicular to bedding, respectively, have been considered (Homand et al. 1997). The proposed best fit solution includes a subvertical maximum principal stress between 6.5 and 8.0 MPa, a subhorizontal intermediate principal stress between 4.0 and 5.5 MPa oriented 320° NE, and a subhorizontal minimum principal stress between 0.6 and 1.1 MPa oriented 52° NE (Bigarré and Lizeur 1997).

The borehole slotter technique has been applied by König and Bock (1997) in the “IS-B experiment”. The measurements have been conducted in three boreholes (perpendicular and oblique to bedding) located in the sandy facies of Opalinus Clay around the IS-Niche (Figure 2.7). For the borehole slotter technique, radial slots are cut into the borehole wall and the strain of the borehole wall at the side of the slot is recorded as the slot is cut. Isotropy and homogeneity are assumed for the evaluation. The measurements revealed a $\sigma_1$ of 2.0-5.0 MPa oriented 194/40, a $\sigma_2$ of 1.0-2.9 MPa oriented 89/21, and a $\sigma_3$ of less than 0.4 MPa oriented 341/43 (König and Bock 1997).

Overcoring experiments with a CSIRO-Hi cell have been conducted by Lahaye (2005) in the “EZ-A experiment” in boreholes drilled oblique to bedding in the northern part of Gallery 98, i.e. in the shaly facies of Opalinus Clay (Figure 2.7). The CSIRO-Hi cell is glued into a pilot borehole which is overcored. During overcoring, the changes in strain due to relieving stresses are recorded. The elastic properties used for the conversion of the strains into stresses have been obtained from laboratory tests (Lahaye 2005). Linear elasticity and transversal isotropy with Young’s moduli of 22.5 and 9.0 GPa
parallel and perpendicular to bedding respectively have been assumed. Consistent values for the principal stress magnitudes were obtained: \( \sigma_1 = 9.5-12.9 \) MPa, \( \sigma_2 = 7.4-10.7 \) MPa, and \( \sigma_3 = 0.9-2.7 \) MPa have been reported (Lahaye 2005). The orientation of the minimum principal stress has been found to be perpendicular to bedding. The intermediate and maximum principal stress lie within the bedding plane but show, however, a high variability in their orientation.

Table 2.2 summarizes the magnitudes and orientations found by the various stress measurement campaigns conducted in Opalinus Clay at the Mont Terri URL. Differences in both magnitude and orientations are obvious. There are several factors and uncertainties that may influence the test results. For the strain relief methods (borehole slotter, undercoring, and overcoring), geomechanical parameters are needed to convert the strains into stresses. Opalinus Clay as a clay shale is very prone to moisture changes and thus both the parameters and the behavior around a borehole strongly depend on the state of the material (Wild et al. 2015). An example of the effect of the use of elastic properties can be seen in the different results obtained by the borehole slotter and undercoring methods applied in the IS-Niche. The difference in the magnitudes from these two test campaigns differ by a factor of 2, which is exactly the ratio in the Young’s moduli used in the two methods. If the same elastic parameters would have been considered, comparable results would have been obtained (Martin and Lanyon 2003).

<table>
<thead>
<tr>
<th>location</th>
<th>method</th>
<th>stress magnitudes (MPa)</th>
<th>orientation</th>
<th>reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>IS-niche</td>
<td>hydraulic fracturing</td>
<td>( \sigma_1 ) 4.2</td>
<td>330°</td>
<td>40°    Evans et al. (1999)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>( \sigma_3 ) &lt; 2.9</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Gallery 04</td>
<td>hydraulic fracturing</td>
<td>( \sigma_2 ) 8.6</td>
<td>033°</td>
<td>0°     Enachescu (2011)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>( \sigma_2 ) 6.7</td>
<td>123°</td>
<td>70°</td>
</tr>
<tr>
<td></td>
<td></td>
<td>( \sigma_3 ) 3.9</td>
<td>303°</td>
<td>20°</td>
</tr>
<tr>
<td>IS-niche</td>
<td>undercoring</td>
<td>( \sigma_1 ) 6.5-8.0</td>
<td>subvertical</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>( \sigma_2 ) 4.0-5.5</td>
<td>subhorizontal</td>
<td>320° NE</td>
</tr>
<tr>
<td></td>
<td></td>
<td>( \sigma_3 ) 0.6-1.1</td>
<td>subhorizontal</td>
<td></td>
</tr>
<tr>
<td>IS-niche</td>
<td>borehole slotter</td>
<td>( \sigma_1 ) 2.0-5.0</td>
<td>190° \pm 13°</td>
<td>42°    König and Bock (1997)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>( \sigma_2 ) 1.0-2.9</td>
<td>83° \pm 13°</td>
<td>19°</td>
</tr>
<tr>
<td></td>
<td></td>
<td>( \sigma_3 ) &lt; 0.4</td>
<td>334° \pm 9°</td>
<td>42°</td>
</tr>
<tr>
<td>EZ-A</td>
<td>overcoring</td>
<td>( \sigma_1 ) 9.5-12.9</td>
<td>within bedding plane</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>( \sigma_2 ) 7.4-10.7</td>
<td>within bedding plane</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>( \sigma_3 ) 0.9-2.7</td>
<td>perpendicular to bedding</td>
<td></td>
</tr>
</tbody>
</table>

Furthermore, the glue used to fix the strain measuring devices to the borehole walls can change the properties of the material. For the hydraulic fracturing method, the pore water chemistry can affect the
measurement as the swelling potential may increase when purer water compared to the natural pore fluid is used. This method but also the others may furthermore be significantly influenced by the bedding orientation and the resulting anisotropy of the rock mass. As bedding planes represent planes of weakness, fracture propagation (during excavation of a borehole, created through hydraulic fracturing) and thus the resulting interpretation of the measurements are strongly biased. The presence of bedding planes also requires the consideration of transversal isotropic elasticity, which was for example not the case for the borehole slotter measurements. Another factor influencing the measurements is the general assumption of linear elastic behavior. However, yielding long before peak strength was observed in undrained compression tests on Opalinus Clay conducted by Amann et al. (2011), Amann et al. (2012), and Corkum (2006). Non-elastic behavior caused by drilling of a borehole was also manifested by the development of a distinct borehole damage zone within twelve hours after drilling (Kupferschmied et al. 2015). Thus, the assumption of linear elasticity for evaluation of the results from the stress relief methods might not be valid.

Despite all the uncertainties, Martin and Lanyon (2003) used numerical models in combination with the field measurements from the IS-Niche to constrain the in-situ stress tensor for the Mont Terri URL. They particularly used the stress tensor from the undercoring campaign (ROSAS1) as an input for a numerical model to calculate the stress distribution around tunnels and boreholes. The results have been compared to the observed rock mass behavior and have been found in good agreement. Furthermore, Martin and Lanyon (2003) discussed the relatively low magnitude of the minimum principal stress, especially when it is compared to the undisturbed in-situ pore pressure which is assumed to range between 1 and 2 MPa. Nevertheless, no justification for disregarding the reported result could have been identified.

K.F. Evans (personal communication, July 29, 2016) on the other hand sees firm physical reasons based on the observations from the hydraulic fracturing campaign in the IS-Niche (which has been excluded from the analysis by Martin and Lanyon 2003) that support that the value of the minimum principal stress from the undercoring campaign in the IS-Niche is too low. Using the stress tensor from the undercoring campaign as suggested by Martin and Lanyon (2003) and applying it to the inclined borehole of the hydraulic fracturing campaign in the IS-Niche results in favorable conditions for axial fracture development with an expected instantaneous shut-in pressures of 0.6-1.1 MPa. This is significantly lower than the measured shut-in pressure of 2.9 MPa. Furthermore, a breakdown pressure of 15 MPa has been observed in the lower part of the inclined borehole in the IS-Niche (Evans et al. 1999), which is not consistent with the large differential stress in the plane normal to the borehole axis as suggested by ROSAS1 (K.F. Evans, personal communication, July 29, 2016). The large differential stress would generate a state of circumferential tension around the borehole and the rock could probably not resist a pressure increase of 15 MPa before failing (K.F. Evans, personal communication, July 29, 2016). Thus, the value of $\sigma_3$ given by ROSAS1 is too low and the exact value of the minimum principal stress remains uncertain. However, an upper bound of 2.9 MPa is given by the results of the hydraulic

Corkum (2006) slightly modified the stress tensor proposed by Martin and Lanyon (2003). Especially the value of $\sigma_3$ has been discussed and further constrained. In addition to the upper bound of 2.9 MPa, Corkum (2006) suggests a lower bound of 0.6 MPa based on the hydrofracture results. Furthermore, his laboratory tests on Opalinus Clay specimens showed a non-linear stress-strain behavior at low confining stress, which is contradicting to the assumption of linear elasticity used in the evaluations of the stress tensor from the stress relief methods and implies an uncertainty in the reliability of the test results. Corkum (2006) gives geophysical and geological interpretations (e.g. extent of EDZ, ultrasonic p-wave velocity measurements, hydro-geological history of the Mont Terri region) and further observations of the behavior of Opalinus Clay (e.g. non-linearity in the stress-strain behavior, self-sealing) to support a value of $\sigma_3$ around 2 MPa. Thus, a value between 2 and 3 MPa is suggested for $\sigma_3$ (Corkum 2006).

The proposed stress tensor by Martin and Lanyon (2003) including the modifications from Corkum (2006) is given in Table 2.3. This tensor will be used within this thesis although the author is aware of the ambiguities and uncertainties that are related to it.

<table>
<thead>
<tr>
<th>stress magnitudes (MPa)</th>
<th>orientation</th>
<th>trend</th>
<th>plunge</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\sigma_1$</td>
<td>6.0-7.0</td>
<td>210°</td>
<td>70°</td>
</tr>
<tr>
<td>$\sigma_2$</td>
<td>4.0-5.0</td>
<td>320°</td>
<td>7°</td>
</tr>
<tr>
<td>$\sigma_3$</td>
<td>2.0-3.0</td>
<td>52°</td>
<td>18°</td>
</tr>
</tbody>
</table>

2.3 Review of previous studies of low permeable soils/rocks in triaxial tests

In shale testing for problems related to nuclear waste repository design, conventional and unconventional oil/gas extraction and CO$_2$-sequestration, three main tests are commonly used to study the drained and undrained behavior of clays/clay shales in triaxial testing conditions: the unconsolidated undrained (UU) test, the consolidated undrained (CU) tests and the consolidated drained (CD) test (Lambe and Whitman 1979). In the UU tests, pore pressures are not measured and not considered in the analysis. The advantage of such tests is that no back-saturation of the specimens is required, which usually is challenging and time-consuming. However, the outcome of UU tests is strongly influenced by the dilatant behavior of the material and the initial effective stress state of the specimen (Amann et al. 2015). Thus, the interpretation of the results may offer room for speculation. Furthermore, the total stress results of UU tests can usually only be used for specific and empirical applications and are therefore not suitable for long-term calculations where effective stress properties are required (Nakken et al. 1989). CU and CD tests allow for measuring pore pressure responses and the determination of effective properties. Both consider a consolidation phase before shearing. In CD tests, drainage of excess pore pressures upon shearing is permitted, i.e. the differential load during shearing is applied such that
the pore pressure is kept constant. In CU tests, the specimen is sheared under undrained conditions, i.e.
no drainage is permitted and excess pore pressure is allowed to build up.

This section gives a short review of consistencies and discrepancies in methods and findings from
previous laboratory triaxial tests on low permeable geotechnical materials. Only consolidated tests
(CU/CD tests) on intact (saturated) specimens are considered.

2.3.1 Sampling, specimen preparation and testing procedures

There is a general agreement in literature that care has to be taken for sampling, sample handling, and
specimen preparation. Otherwise, a change in mechanical and physical properties upon drying and
rewetting of the specimens will occur (e.g. Wichter 1979, Chiu et al. 1983, Peters 1988, Botts 1998,
Hight 2003, Wild et al. 2015, Ewy 2015). Different sampling techniques have been applied. Most
commonly, block samples are obtained from which core samples for triaxial tests are drilled (e.g. Bishop
al. 2013). Subsampling has also been applied to gain cylindrical specimens from bigger cores (e.g.
have also been cut directly from field cores (e.g. Chiu et al. 1983, Aristorenas 1992). Different fluids or
gas have been used for cooling during cutting and drilling: brine (e.g. Wu 1991, Marsden et al. 1992,
Petley et al. 1993, Ewy et al. 2003), inert fluids like mineral oil (e.g. Steiger and Leung 1991a, Steiger
and Leung 1991b, Horsrud et al. 1998, Islam and Skalle 2013), water (e.g. Swan et al. 1989, Leddra et
The samples have usually been protected from the environment by sealing them in plastic bags, tubes,
or bottles (e.g. Chiu et al. 1983, Wu 1991, Steiger and Leung 1991a, Ewy et al. 2003, Deng et al. 2011)
or by coating them in wax (e.g. Steiger and Leung 1991a, Steiger and Leung 1992, Aristorenas 1992,
typically range between 12.5 and 76 mm. Usually, a height to diameter ratio of 2:1 has been used.

Different procedures have been applied during testing. Some researchers (e.g. Steiger and Leung 1989,
al. 2003, Islam and Skalle 2013) conducted tests comprising three steps: 1) loading to a predetermined
level of pore pressure and confinement, 2) consolidation of the specimens, and 3) axial loading at a
constant axial strain/displacement rate. In those tests, saturation has been achieved through
consolidation but has often not been explicitly confirmed. In some of these tests, the specimens have
initially been placed into a desiccator to equilibrate with a constant level of relative humidity and thus
achieve a specific water content (e.g. Chiu et al. 1983, Steiger and Leung 1991b, Ewy et al. 2003). Other
researchers additionally included a saturation phase at the beginning of the tests utilizing back pressures
2011, Yu et al. 2012, Dong et al. 2013, Bésuelle et al. 2013, VadenBerge et al. 2014). In some studies,
the saturation of the specimens has been confirmed by measuring Skempton’s pore pressure coefficient $B$ (Skempton 1954, Baracos et al. 1980, Bellwald 1990, Wu 1991, Aristorenas 1992, Taylor and Coop 1993, Barla 1999, Yu et al. 2012, Dong et al. 2013, VandenBerge et al. 2014). A specimen was assumed to be saturated when the $B$-value was higher than a certain value or constant for subsequent measurements. Others considered a specimen saturated when the fluxes of water stabilized (Bésuelle et al. 2013) or the pore pressure at the outlet and inlet equilibrated (Wu et al. 1997, Hu et al. 2014).

Depending on the permeability and size of the specimen, different time ranging from several hours to several days has been allocated for consolidation. Pore pressure changes and strains have been used to confirm complete consolidation of the specimens (e.g. Wu 1991, Taylor and Coop 1993, Amorosi and Rampello 2007).

The reported axial strain rates also cover a wide range of values: CU tests have been conducted with strain rates from the order of $10^{-8}$ s$^{-1}$ (e.g. Steiger and Leung 1991a) to $10^{-4}$ s$^{-1}$ (e.g. Graham and Li 1985, Marsden et al. 1992). Strain rates between $10^{-7}$ s$^{-1}$ and $10^{-5}$ s$^{-1}$ have been used for CD tests (e.g. Amorosi and Rampello 2007, Deng et al. 2011, Bésuelle et al. 2013, Dong et al. 2013).

2.3.2 Specimen behavior during shearing

Despite the different materials and testing procedure, similarities in the behavior of the clays/clay shales can be identified. Bishop et al. (1965) conducted CU and CD tests on London Clay and found a progressive change from brittle to ductile behavior with increasing effective confinement. Brittle behavior for overconsolidated specimens and rather ductile behavior for normally consolidated or slightly overconsolidated specimens have also been reported by Baracos et al. (1980), Graham and Li (1985), Marsden et al. (1992), Ewy et al. (2003), Nygård et al. (2006), Hu et al. (2014). Further differences in the behavior between overconsolidated and normally consolidated specimens has been found in the occurrence of dilatancy, which is reflected in the volumetric behavior or the pore pressure evolution. Bishop et al. (1965) noticed a marked dilatancy at the peak of the stress-strain curves for lower effective confinements that decreased with increasing effective confinement. They related this observation to a possible greater break down of particle aggregates during shear at higher effective confinements. A transition from compaction to dilatancy at low confinements (i.e. for overconsolidated specimens) has also been observed by Baracos et al. (1980). After a linear pore pressure increase at the beginning of the test, specimens consolidated at low effective stresses (i.e. overconsolidated specimens) showed dilatancy towards the maximum shearing resistance and thus pore pressure started to decrease whereas normally consolidated specimens still showed an increasing pore pressure response in the pre- and post-peak stage. Similar findings have been reported by Wu (1991), Horsrud et al. (1998), Ewy et al. (2003), Amorosi and Rampello (2007), Islam and Skalle (2013), and Hu et al. (2014).

Wu (1991) concluded from the observed behavior that overconsolidated shale specimens behave more like a rock-like material whereas normally consolidated specimens are more soil-like. Similarly, Horsrud et al. (1998) used the critical state soil mechanics theory (Schofield and Wroth 1968) to explain
the difference between clay shales and clays. In the overconsolidated range, the post-peak effective stress path for a uniform clay follows the Hvorslev surface up to the critical state. Overconsolidated and cemented rock will behave in a non-uniform manner. For an overconsolidated clay shale, strain localization will take place within the specimen and shear bands will develop, which will finally lead to a shear plane through the specimen. This is manifested by the brittle post-failure response. Thus, the post-peak effective stress path of a clay shale does not follow the Hvorslev surface. The strain localization and formation of a shear band is supported by the findings of Wu (1991) who tested specimens of a smectite shale and found that all specimens failed along single shear plane/zone. The behavior for an overconsolidated clay shale specimen after peak strength is more dependent on the characteristics that are typical for a rock (e.g. cementation, fabric) rather than a soil (Horsrud et al. 1998).

2.3.3 Strength

2.3.3.1 Undrained shear strength

For clays, the SHANSEP procedure (Stress History and Normalized Soil Engineering Properties) developed by Ladd and Foot (1974) is often used to characterize the undrained shear strength. According to this empirical relationship, the undrained shear strength ($s_u = q/2$) obtained from an isotropically consolidated undrained triaxial compression test, normalized with respect to the effective consolidation stress is related to the overconsolidation ratio by a power law (Ladd et al. 1977)

$$\frac{s_u}{\sigma_c} = a(OCR)^b,$$

(2.1)

where $a$ and $b$ are parameters, $\sigma_c$ is the isotropic consolidation stress, and $OCR$ is the overconsolidation ratio defined as $OCR = \sigma_{cm}'/\sigma_c'$, where $\sigma_{cm}'$ is the maximum past effective isotropic stress.

The empirical relationship of the normalized behavior can be used to predict the undrained shear strength of a clay/clay shale. However, Nygård et al. (2006) recommend to use it only to get a rough estimate of the undrained shear strength when no data based on actual field material is available. Furthermore it should be noted that the “$\phi = 0$”-concept might not be applicable for clay shales (Amann et al. 2015, Amann and Vogelhuber 2015). Ladd and Foot (1974) showed curves for different clays relating the normalized undrained shear strength with the overconsolidation ratio. Analyses of these curves gave values of $a = 0.2-0.3$ and $b = 0.7-0.8$. Tests from Bellwald (1990) and Aristorenas (1992) on Opalinus Clay and Lias Alpha shale revealed $a = 0.74$ and $b = 0.60$. Based on tests from different shales, Gutierrez et al. (1996), Nygård et al. (2006), and Gutierrez et al. (2008) demonstrated the applicability of SHANSEP for clay shales and therefore the applicability of the correlation even for materials showing apparent pre-consolidation due to diagenetic bonding and cementation. Gutierrez et al. (1996) reported mean values of 0.29 and 0.93 for $a$ and $b$, respectively. Nygård et al. (2006) assembled normalized undrained shear strength values from 40 types of mudrocks and shales from the North Sea and found a mean value of 0.39 for $a$ and 0.89 for
Values from triaxial tests on 25 types of shales from different locations were reported by Gutierrez et al. (2008). Mean value of $a = 0.37$ and $b = 0.87$ were found. The data has additionally been differentiated into individual clay shales (e.g. Kimmeridge shale: $a = 0.47$, $b = 0.66$, Barents Sea shale: $a = 0.40$, $b = 0.91$, Fuller’s Earth shale: $a = 0.41$, $b = 0.72$, Weald Clay: $a = 0.49$, $b = 0.47$).

2.3.3.2 Effective strength
Non-linear failure envelopes for the effective peak strength of clays and clay shales have been reported by many authors (e.g. Bishop et al. 1965, Graham and Li 1985, Nakken et al. 1989, Burland 1990, Wu 1991, Petley 1999, Amorosi and Rampello 2007, Deng et al. 2011). Generally, a decrease in slope (i.e. effective friction angle) has been observed when passing from low to higher effective confinement, which was related to different phenomena. Bishop et al. (1965) conducted CU and CD tests on specimens of undisturbed London Clay and found a non-linear failure envelope, a progressive change from brittle to ductile behavior with increasing effective confinement, and a marked dilatancy around peak strength limited to the range of lower effective confining stresses. Graham and Li (1985) related the non-linear failure envelope for natural Winnipeg clay to the micro- and macrostructure of the material. Petley (1999) reviewed undrained shear test results from different clays and clay shales and proposed that the non-linearity of the envelopes represents the initiation of the brittle-ductile transition. Nygård et al. (2006) demonstrated the transition from brittle to ductile behavior by analyzing the undrained shear behavior of shales and mudrocks from the North Sea and adjacent area. Normally consolidated or slightly overconsolidated specimens showed a ductile behavior while overconsolidated specimens with high overconsolidation ratios showed brittle behavior. A transition from brittle to ductile behavior with increasing confinement has also been reported by Hu et al. (2014) for a Callovo-Oxfordian clay shale from Meuse-Haute/Marne. However, neither Nygård et al. (2006) nor Hu et al. (2014) report non-linear failure envelopes.

2.3.4 Anisotropy
Clay shales are generally anisotropic with respect to their mechanical properties and elastic behavior, which is related to the preferred orientation of clay platelets. Usually, transversal isotropy is applied as an approximation to describe this behavior. Sarg and Hazen (1987), for example, confirmed the validity of transversal isotropy for two grey shales by means of multiaxial tests. Nakken et al. (1989) conducted CU and CD tests on different shales and found higher values of elastic moduli parallel to bedding than normal to it. Similar findings have been reported by Niandou et al. (1997) and Horsrud et al. (1998).

Wu (1991) found for specimens of a smectite shale that the peak strength for overconsolidated specimens is higher in direction parallel to the bedding than in any other direction. For normally consolidated specimens, he found the opposite, i.e. the peak strength normal to bedding was higher than in the other directions. Stronger specimens tested parallel to bedding than specimens tested normal to bedding have also been reported by Horsrud et al. (1998) and Islam and Skalle (2013). Niandou et al.
(1997) reported no significant difference between P- and S-specimens. Specimens tested at other angles with respect to bedding, however, showed lower peak strength values.

Influence of the anisotropy on the pore pressure response has been reported by Bellwald (1990), Aristorenas (1992), and Islam and Skalle (2013). Bellwald (1990) and Aristorenas (1992) investigated the hydro-mechanical behavior of Opalinus Clay with the help of undrained pure-shear compression and extension tests. They demonstrated that excess pore pressures develop even under undrained pure shear conditions (i.e. under constant octahedral normal stress), which would not have been observed for an isotropic material. Furthermore, the pore pressure response was different for specimens tested under pure shear compression than for specimens tested under pure shear extension. At low differential stress, positive excess pore pressure developed within specimens tested under pure shear compression whereas specimens tested under pure shear extension showed negative excess pore pressure. Islam and Skalle (2013) conducted CU tests on specimens of Pierre-1 shale using standard triaxial stress paths. They observed that specimens tested normal to bedding compacted more and therefore showed higher pore pressure before the onset of dilation (i.e. at low differential stresses). Less pore pressure was built up in specimens tested parallel to bedding.

2.3.5 Influence of structure


Burland (1990) reviewed test results from triaxial tests on intact and reconstituted specimens of Todi Clay and London Clay and concluded the microstructure affects the peak strength. The properties or the reconstituted specimens are referred to as intrinsic properties as they are inherent to the soil and independent of the structure at its natural state. The intrinsic properties therefore can be seen as a reference for the comparison to the intact properties and thus for the identification of the influence of the structure. Leroueil and Vaughan (1990) noted that the structure may not only be removed by remolding but can also be lost due to unloading of specimens that were consolidated beyond their yield stress. Similarly, Amorosi and Rampello (2007) reported a significant change in structure for specimens isotropically consolidated to stresses beyond the preconsolidation pressure. Furthermore, swelling of the specimens can lead to the destruction of diagenetic bonds and a decreased influence of the structure (Bjerrum 1967, Calabresi and Scarpelli 1985, Graham and Au (1985), Leroueil and Vaughan 1990, Takahashi et al. 2005, Picarelli et al. 2006).

The strength of intact specimens has been found to be higher than the intrinsic strength and the intact specimens often showed brittle behavior (Burland 1990, Leroueil and Vaughan 1990, Leddra et al. 1992,
Horsrud 1998, Deng et al 2011). Leddra et al. (1992) postulated that this brittleness can be related to the natural fabric (i.e. the preferred orientation of the clay minerals) and the intergranular bonding (due to cementation and chemical reaction between the clay minerals). A progressive destructuring at higher confinements will lead to an increase in plasticity and causes material behavior that is more closely to remolded specimens. Similar findings have been reported by Petley et al. (1993).

Deng et al. (2011) compiled strength values for intact and reconstituted specimens of Boom Clay and found a non-linear failure envelope for intact specimens that was higher than the linear failure envelope of reconstituted specimens. They related the difference in failure envelopes to the cementation effect of the carbonate that gets destroyed during reconstitution.

Favero et al. (2016) compared results from oedometer tests on remolded and natural specimens of Opalinus Clay and found differences with respect to stiffness, swelling tendency, porosity and creep. Remolded specimens are less stiff, show higher void ratio, higher porosity, and a higher swelling tendency. They related the findings to diagenetic features, i.e. cementation and diagenetic bonds that are present in the natural specimens and that are destroyed or cannot be reestablished in remolded specimens. No significant influence on permeability was found as connecting pores are not impacted by cementation or diagenetic bonding.

### 2.4 References


3. Water retention characteristics and state-dependent mechanical and petro-physical properties of a clay shale


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**Abstract:** A series of clay shale specimens in equilibrium with various humidity conditions were used to establish the water retention characteristics, the influence of suction on ultrasonic p-wave velocity and rock mechanical properties such as Young’s modulus, Poisson’s ratio, onset of dilatancy, unconfined compressive strength and Brazilian tensile strength. Opalinus Clay, a clay shale considered as host rock for the disposal of nuclear waste in Switzerland was utilized. The results showed that the p-wave velocity normal to bedding ($v_{p,n}$) dropped sharply upon desaturation until suction approached the air-entry value. The sharp decrease was associated with desiccation cracks solely oriented parallel to bedding. For suction in excess of the air-entry value, $v_{p,n}$ was constant, indicating no further desiccation damage. The suction at the shrinkage limit and at the air-entry point are similar in magnitude. The p-wave velocity parallel to bedding ($v_{p,p}$) remained constant in the entire range of suction investigated in this study. The constant $v_{p,p}$ with increasing suction might be associated with the disproportional decrease of the Poisson’s ratio and Young’s modulus, and its opposing effect on p-wave velocity. An almost linear increase in unconfined compressive strength, Brazilian tensile strength, stress at the onset of dilatancy and Young’s modulus with increasing suction was observed up to a suction of 56.6 MPa. For suction larger than 56.6 MPa, relatively constant strength and stiffness was observed. The increase is associated with the net contribution of suction to strength/stiffness, which decreases non-linearly with decreasing volumetric water content. The rate of increase in tensile strength and unconfined compressive strength with increasing suction is different depending on the rock anisotropy. Compared to the strength values (Brazilian tensile and uniaxial compressive strength) obtained from specimens loaded parallel to bedding, the tensile strength parallel to bedding and the unconfined compressive strength obtained from specimens loaded normal to bedding are considerably more affected by increasing suction or decreasing water content. The reasons for the different rates in strength increase are considered to be related to local variations in suction (i.e. local suction) as a consequence of zones of contrasting pore size distribution. These variations may influence the effect of suction on strength, especially when the load is applied parallel to bedding and crack growth occurs predominately along bedding layers with comparably low suction.
3.1 Introduction

Numerous experimental studies revealed that the moisture content of laboratory specimen can have a major influence on the strength and deformability of soils and rocks (Fredlund et al. 1987, Escario et al. 1989, Dyke and Dobereiner 1991, Hsu and Nelson 1993, Lashkaripour and Passaris 1995, Rahardjo et al. 1995, Cui and Delage 1996, Valès et al. 2004, Wild 2010, Feuerharmel et al. 2006, Chae et al. 2010, Nam et al. 2011, Schnellmann et al. 2013). It was consistently shown that the increasing strength or stiffness, along with decreasing moisture content, is associated with capillary forces arising from both the pressure difference between the wetting and the non-wetting fluid at the gas/liquid boundary (capillary suction), and osmotic processes (osmotic suction) in a partly saturated system composed of solids, air and water (Fredlund et al. 1978, Schmitt et al. 1994, Birle 2012). The total suction, further referred to simply as suction, is the sum of capillary and osmotic suction. Surface tension at the boundary between water and air in adjoining voids causes a compressive contact pressure between grains, which increases the shear resistance. This additional resistance has the same effect as if the grains were held together with a cohesive strength component, and is often called the apparent cohesive strength component (Fredlund et al. 1978, Peterson 1988, Schmitt et al. 1994, Papamichos et al. 1997).

West (1994) analyzed the effect of suction on the uniaxial compressive strength (UCS) of various sand- and limestones, and found that for suction between 0 and approximately 10 MPa, the UCS of these rock types is little affected. When the suction exceeds approximately 10 MPa, the UCS increased markedly. Similar results have been reported by Ramos de Silva et al. (2008) for low-porosity shales. Schmitt et al. (1994) investigated the dependency of UCS on suction for Tournemire shale, Vosges and Fontainebleau sandstone. For Vosges and Fontainebleau sandstone, where the pore space is associated with pore radii larger than 0.1 μm, suction larger than 1 MPa developed when the saturation dropped below approximately 10%. The increase in suction from 1 to 8-9 MPa (for a decrease in saturation from 10 to 2%) was accompanied with an increase in UCS (e.g. for Vosges Sandstone the UCS increased from 25 to 37 MPa\(^1\)). When the saturation was larger than 10%, the UCS was unaffected. For Tournemire shale, where the pore space is primarily associated with pore radii smaller than 0.1 μm, the water retention characteristics were substantially different, and suction larger than 1 MPa developed already when the saturation dropped slightly below 100% (Schmitt et al. 1994). The increasing suction in the shale was associated with a continuous increase in UCS (i.e. the UCS at a saturation of 66.5% was 19.8 MPa, and at 24.5% it was 60 MPa). Based on pore size distribution and water retention characteristics, Schmitt et al. (1994) proposed a saturation threshold for which decreasing saturation becomes significant for the UCS. For rock types with predominately micro- and meso-pores, such as Tournemire shale, this threshold is already exceeded by little changes in saturation (from an initially

\(^1\) In this paper, the geomechanics convention is used, with tension denoted as negative number, and compression denoted as positive number.
saturated state). These little changes can cause a significant increase in suction accompanied by a measurable increase in the unconfined compressive strength.

The aforementioned studies consistently showed that suction can have a substantial influence on rock mechanical properties, such as the compressive strength and stiffness. This is particularly relevant for rock types with predominately micro-pores, such as mudrocks or clay shales. In such rocks a small decrease in saturation from an initially saturated state may lead to considerable suction. Thus the strength and stiffness properties of these partially saturated specimens may not be representative for the in-situ properties.

In this study, a series of Brazilian tensile and unconfined compressive strength tests on Opalinus Clay (OPA) specimens were used to establish water retention characteristics and a relationship between suction and rock mechanical properties. The study focuses on the influence of suction on the Young’s modulus, Poisson’s ratio, the stress threshold at the onset of dilatancy, the uniaxial compressive strength, and the tensile strength. The latter was investigated both normal and parallel to bedding, while the other properties were determined parallel to bedding. The influence of increasing suction on the ultrasonic p-wave velocity parallel and perpendicular to the bedding was also investigated.

### 3.2 Material description

For this study, samples from the shaly facies of Opalinus clay at the Mont Terri research laboratory in Switzerland were utilized. The mass fractions of the predominant mineralogical components of the shaly facies are clay minerals (50-66%), quartz (10-20%), carbonates (8-20%), and feldspar (3-5%) (Thury and Bossart 1999; Klinkenberg et al. 2009). The mass fraction of clay minerals is composed of 20-30% 2:1 layer and mixed layer silicates, 7-8% chlorite, and 20-25% Kaolinite (Klinkenberg et al. 2009). Clay platelets are tabular-shaped and lie sub-parallel with the macroscopic bedding, which is made up of siderite concretions and silt and sandstone lenses. The water loss porosity of the shaly facies varies between approximately 13-20% (Thury and Bossart 1999). The hydraulic conductivity of the shaly facies varies between $10^{-12}$ m/s parallel to bedding and $10^{-14}$ m/s normal to bedding (Thury and Bossart 1999).

Houben et al. (2013) analyzed the pore-size distribution of the shaly facies using various methods. They suggest that the majority of equivalent pore radii are in a range between approximately 0.003-0.1 μm. They also found that the representative element area ($REA$) for the mineralogical composition and pore-size distribution is $100 \times 100$ μm$^2$ (i.e. for areas larger than the $REA$, fluctuation in mineralogical composition and porosity are negligible).
3.3 Sampling and testing methods

3.3.1 Sampling, specimen handling and specimen preparation

Opalinus Clay samples were taken from 89 mm diameter cores obtained from a 38-meter-long borehole (BFE-A3) in the shaly facies at the Mont Terri Underground Rock Laboratory. Double-tube core barrels with compressed air cooling were utilized, and specimens were hermetically sealed in vacuum-evacuated foil immediately after core extraction.

All specimens were cut under dry conditions at the Institute of Structural Engineering at the ETH Zurich, using a rigid prismatic specimen-holder and an electronically controlled diamond-saw (WELL, Model 6234). The constant band rotating speed, the constant feed rate and the thin metal string (0.3mm) populated with diamonds allows for vibration-less cutting. Specimens for Brazilian tensile strength testing were cut to a diameter-height ratio of 2:1, those for the uniaxial compressive strength tests to 1:2. After cutting, the end-faces were polished. Parallelism of the end-faces met the requirements of the ISRM suggested methods (ISRM 1978, ISRM 1979a). The environmental exposure time of the specimens was minimized through a rigorous preparation procedure and immediate sealing of the specimens between subsequent preparation steps.

3.3.2 Water content and degree of saturation

The mass fraction of water ($\omega$) and saturation were determined according to ISRM suggested methods (ISRM 1979b). The volumetric water content ($\theta$) was calculated using the following equation

$$\theta = \omega \rho_d, \quad (3.1)$$

where $\rho_d$ is the dry density which is related to the bulk density ($\rho_{\text{bulk}}$) and the water content ($\omega$) by the following relationship

$$\rho_d = \frac{\rho_{\text{bulk}}}{1 + \omega}. \quad (3.2)$$

The water loss porosity ($\phi$) was derived from the dry density ($\rho_d$) and grain density ($\rho_s$) using

$$\phi = 1 - \frac{\rho_d}{\rho_s}. \quad (3.3)$$

The grain density is given as 2.7 ± 0.2 g cm$^{-3}$ (Bock 2008). The degree of saturation ($S_w$) was calculated using

$$S_w = \frac{\omega \rho_d}{\phi \rho_w}, \quad (3.4)$$

where $\rho_w$ is the density of water.

3.3.3 Water retention curve

Water retention characteristics were established from specimens exposed to various levels of humidity in desiccators under stress free conditions. Constant predefined relative humidity was achieved and maintained by utilizing supersaturated salt solutions (Table 3.1). While exposing the specimens to these
different environments, the air temperature in the laboratory was held constant at 21 ± 0.5°C, and the relative humidity in the desiccators was monitored. The correspondent suction pressure was calculated according to Kelvin’s relationship

\[ \psi = -\frac{RT}{V_{w0} \omega_w} \]  (3.5)

where \( \psi \) is the suction in Pa, \( R \) the ideal gas constant in J/mol/K, \( T \) the absolute temperature in Kelvin, \( V_{w0} \) the specific volume of water (i.e. the inverse of the water density) in m³/kg, \( \omega_w \) the molecular mass of water vapor, \( p \) the vapor pressure of water in the system in MPa, and \( p_0 \) the vapor pressure of pure water in MPa. The term \( p/p_0 \) is the relative humidity as given by the experimental setup. The suction values utilized in this study are given in Table 3.1 and range from 1.4 to 224.9 MPa.

Table 3.1: Saline solutions and the corresponding theoretical and achieved relative humidity. The suction values correspond to the achieved relative humidity.

<table>
<thead>
<tr>
<th>Salt solution</th>
<th>Relative humidity (-) theoretical (%)</th>
<th>achieved (%)</th>
<th>Suction (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>K₂SO₄</td>
<td>&gt; 97</td>
<td>97 / 99</td>
<td>4.1 / 1.4</td>
</tr>
<tr>
<td>KCl</td>
<td>84-85</td>
<td>85</td>
<td>22.0</td>
</tr>
<tr>
<td>NaCl</td>
<td>75</td>
<td>75</td>
<td>39.0</td>
</tr>
<tr>
<td>NaN₂</td>
<td>66</td>
<td>66</td>
<td>56.3</td>
</tr>
<tr>
<td>Ca(NO₃)₂*4(H₂O)</td>
<td>51-54</td>
<td>52 / 53</td>
<td>88.6 / 86.0</td>
</tr>
<tr>
<td>K₂CO₃</td>
<td>43</td>
<td>43</td>
<td>114.3</td>
</tr>
<tr>
<td>CaCl₂</td>
<td>29-31</td>
<td>31 / 38</td>
<td>158.6 / 131.0</td>
</tr>
<tr>
<td>LiCl</td>
<td>15</td>
<td>19</td>
<td>224.9</td>
</tr>
</tbody>
</table>

The specimens were dried or saturated to constant weight, and the degree of saturation at each applied suction was determined as outlined above. The experimentally observed relationship between saturation and suction was fitted to the Van Genuchten equation (equation (3.6), with fitting parameters \( P_0 \) and \( \beta \), according to Van Genuchten 1980) to establish the water retention curves for both, drying and wetting path.

\[ S_w = \left[ 1 + \left( \frac{\psi}{P_0} \right)^{1/(1-\beta)} \right]^{-\beta} \]  (3.6)

### 3.3.4 Ultrasonic p-wave velocity measurements

P-wave velocity was measured utilizing high frequency ultrasonic pulse technique. A proceq ultrasonic pulse device was used, which is composed of an impulse generator, transducer (transmitter and receiver), time mark generator and a cathode ray oscilloscope (Figure 3.1). A pulse width of 1-10 seconds and a frequency of 54 kHz were utilized. The repetition frequency mounts up to 10-10³ repetitions per second. The velocity was derived by dividing the measured travel time by the specimen’s length or diameter, respectively. Before each measurement, the sensor array was calibrated with a calibration rod. Ultrasonic p-wave velocity parallel (\( v_{p,p} \)) and perpendicular (\( v_{p,n} \)) to the bedding plane orientation were obtained
from specimens in equilibrium with the applied suction (Figure 3.2). Additionally, one specimen was
dried at ambient conditions (i.e. 37% relative humidity and 22°C on average) in the laboratory with
simultaneous monitoring (in intervals of 15 minute) of the weight loss and the p-wave velocity normal
to bedding.

3.3.5 Mechanical testing procedure
Brazilian and uniaxial compression tests were performed at the rock mechanical laboratory at the ETH
Zurich (Chair of Engineering Geology). A modified 2000 kN Walter and Bai servo-hydraulic rock
testing device with digital feedback control was utilized.
Brazilian tests were performed with a constant loading rate of 0.1 kN/min. The load was applied parallel
or normal to bedding (Figure 3.3). For the unconfined compressive strength tests, load was applied in
direction parallel to bedding in such a way as to maintain a constant circumferential displacement rate.
The selected rate for the tests was 0.04 to 0.08 mm/min for specimens conditioned at high relative humidity (i.e. 75, 95, and 99%), and 0.01 mm/min for specimens exposed to lower relative humidity. Since the loading rates were high and the hydraulic conductivity of this clay shale is low, the tests are considered undrained. Axial and circumferential strain gages were mounted onto the specimen at half of the specimen height to eliminate the influence of end effects on the strain measurements (Figure 3.4). Two axial strain gages (Type BD 25/50, DD1), each with a base-length of 50 mm, were firmly attached on opposite sides of the specimens. The radial strain ($\varepsilon_{\text{radial}}$) was calculated from the displacement measured by a single gage (Type 3544-150M-120m-ST), which was attached to a chain wrapped tightly around the specimen (Figure 3.4).

**Figure 3.3:** Loading configurations for Brazilian tests with respect to bedding: load is applied a) parallel or b) perpendicular. For configuration a) the Brazilian tensile strength parallel ($\sigma_{t,p}$) to bedding was obtained, for b) normal to bedding ($\sigma_{t,n}$).

**Figure 3.4:** Monitoring setup for unconfined compression tests.

### 3.3.6 Determination of the onset of dilatancy and elastic parameters
For unconfined compression tests, two strain-based methods were utilized to determine the onset of dilatancy ($\phi_f$). The two different strain-based approaches are illustrated in Figure 3.5a and Figure 3.5b:

1) Brace et al. (1966) suggested that the onset of dilation can be established by examining when the axial stress-volumetric strain curve deviates from its linear portion at low axial stress (Figure 3.5a). Volumetric strain ($\varepsilon_{\text{vol}}$) was calculated from the sum of the arithmetic mean of the two axial strains ($\varepsilon_{\text{axial}}$) and twice the radial strain ($\varepsilon_{\text{radial}}$)
\[ \varepsilon_{\text{vol}} = \varepsilon_{\text{axial}} + 2\varepsilon_{\text{radial}} \]  

(3.7)

2) Lajtai (1974) applied the same principle as Brace et al. (1966) to the axial stress-radial strain curve. The onset of dilatancy is taken at the point where the radial strain curve deviates from linearity (Figure 3.5b).

Both, Young’s modulus \((E)\) and Poisson’s ratio \((\nu)\) were obtained from the linear part of the stress-axial strain curve at low axial stresses (i.e. axial stress < \(C\!l\)).

![Figure 3.5: Methodology for determining the onset of dilatancy in unconfined compression tests by examining a) the volumetric strain response according to Brace et al. (1966) and b) the radial strain response according to Lajtai (1974).](image)

3.4 Results

3.4.1 Water retention characteristics

Figure 3.6 shows the relationship between water content and suction obtained from specimens with water loss porosities between 15 and 19%. The relationship between saturation and suction for the same specimens is shown in Figure 3.7.
Data points for desorption (drying) represent several individual specimens conditioned in desiccators with relative humidity between 19 and 85%. The scatter in the degree of saturation for the same applied suction is most probably associated with the natural variability in pore size distribution. The adsorption path (wetting) was obtained from specimens that were consecutively saturated in six desiccators with relative humidity ranging from 38 to 85%. Specimens which were placed in desiccators at 97 and 99% relative humidity, immediately after dismantling, showed a slight increase in water content in the order
of 1%, suggesting that the specimens were not saturated. These specimens are further considered to be representative for the wetting path.

Drying and wetting path are both non-linear (Figure 3.6). The data further shows that the water content and saturation degree are reversible, following a hysteresis loop. The hysteresis between wetting and drying path becomes clearer when saturation is plotted against suction as shown in Figure 3.7. The reversibility of water content and saturation suggests that air entrapment in the rock is negligible.

The experimental data in Figure 3.7 were fitted to the Van Genuchten equation (see Equation (3.6), Van Genuchten 1980). The corresponding fitting parameters are given in Table 3.2. The drying path was used to derive an estimate of the air-entry suction (i.e. the suction where air theoretically enters the larger pores of the specimen). The air-entry value was determined by extending the tangent of the central part of the Van Genuchten fit to its intersection with the saturation axis (i.e. where saturation equals 1.0, Figure 3.8). A value of 22 MPa was found. Data and water retention curves obtained in this study are in agreement with the water retention characteristics obtained from the shaly facies of OPA by Ferrari and Laloui (2013) using psychrometer measurements on specimens with different water contents (Figure 3.9).

Table 3.2: Fitting parameters for the water retention curves. The deviations given in parentheses correspond to the 95% confidence interval.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Drying Path</th>
<th>Wetting Path</th>
</tr>
</thead>
<tbody>
<tr>
<td>$P_0$</td>
<td>47.4 (± 3.0)</td>
<td>15.7 (± 1.45)</td>
</tr>
<tr>
<td>$\beta$</td>
<td>0.52 (± 0.02)</td>
<td>0.36 (± 0.01)</td>
</tr>
</tbody>
</table>

Figure 3.8: Determination of the air-entry point. The air-entry suction was found to be 22 MPa.
3.4.2 Variations in ultrasonic p-wave velocity

Figure 3.10 shows the mean values and standard deviations of ultrasonic p-wave velocities obtained from cylindrical specimens in equilibrium with the applied suction (Table 3.1). Also shown in Figure 3.10 are mean p-wave velocities measured immediately after core dismantling, and p-wave velocities reported by Amann et al. (2011) and Amann et al. (2012) obtained immediately after core dismantling. The suction values of the latter specimens were estimated using the relationship between water content and suction as shown in Figure 3.6 and are found to be approximately 12 and 7 MPa, respectively.

The p-wave velocities measured parallel to bedding ($v_{p,p}$) were nearly constant at a value of 3000 m/s for the entire range of suction tested, and remained constant for the drying and wetting path. This is in contrast to the p-wave velocity normal to bedding ($v_{p,n}$). A slight increase in suction from its initial stage (i.e. after core dismantling) was associated with a substantial drop in $v_{p,n}$ from 2250 m/s at 12 MPa suction to 1000 m/s at 22 MPa suction. This sharp drop in $v_{p,n}$ was also observed within 0-3 hours of drying under ambient conditions, and remained constant at 1000 m/s for an elapsed time of > 26 hours (Figure 3.11). For suction larger than 22 MPa, $v_{p,n}$ was nearly constant at 1000 m/s, for both, wetting and drying path (Figure 3.10). For the wetting path, $v_{p,n}$ remained at around 1000 m/s, even for applied suctions smaller than 22 MPa. This indicates that the major drop in $v_{p,n}$ is irreversible.
Figure 3.10: Relationship between p-wave velocity normal and parallel to bedding and suction.

Figure 3.11: Evolution of the p-wave velocity (measured normal to bedding) and relative water loss with time obtained from a specimen dried under ambient conditions (i.e., 22 °C and 37 % relative humidity on average).
3.4.3 Variations in elastic properties

Figure 3.12 and Figure 3.13 show variations in Young’s Modulus and Poisson’s ratio, respectively, related to changes in suction. Both, the mean values and the data range (i.e. the minimum and maximum measured values) are shown for the corresponding applied suction (see also Table 3.4). For specimens that were tested immediately after core dismantling and specimen preparation, the equivalent suction was estimated to be approximately 13 MPa, using the relationship between water content and suction (Figure 3.6).

The Young’s modulus increases almost linearly from 6.7 GPa after core dismantling to 23.3 GPa for a suction of 56.6 MPa. Between 56.6 and 224.9 MPa suction, \( E \) does not tend to further increase, even though the scatter is high in this range of suction. The Poisson’s ratio exhibits an opposing, but similar trend as \( E \). The Poisson’s ratio decreases from approximately 0.20 after core dismantling to 0.11 at a suction of 56.6 MPa. For suction larger than 56.6 MPa, the Poisson’s ratio varies around a mean value of 0.11.

![Figure 3.12: Relationship between the Young’s modulus and suction. Mean value and data range for each applied suction are given.](image)
Figure 3.14 shows the mean, minimum and maximum Brazilian tensile strength ($\sigma_t$) parallel ($\sigma_{t,p}$) and normal ($\sigma_{t,n}$) to bedding with increasing suction (see also Table 3.3). It can be seen that $\sigma_{t,p}$ is generally higher than $\sigma_{t,n}$, reflecting the transversal isotropy of Opalinus Clay. For the lowest suction applied in this study (i.e. 1.4 MPa), the anisotropy ratio ($\sigma_{t,p}/\sigma_{t,n}$) is approximately two. With increasing suction the anisotropy ratio increases, indicating dissimilar increase in $\sigma_{t,p}$ and $\sigma_{t,n}$. Even though the rate of increase with increasing suction is dissimilar, both, $\sigma_{t,p}$ and $\sigma_{t,n}$ exhibit an almost linearly increase up to a suction of 56.6 MPa. The maximum measured $\sigma_t$ was -1.5 MPa for $\sigma_{t,n}$, and -3 MPa for $\sigma_{t,p}$ (i.e. ~3 times the $\sigma_t$ at a suction of 1.4 MPa, Figure 3.14). For suction larger than 56.6 MPa, $\sigma_{t,n}$ did not tend to increase, and varied around a mean value of approximately -1.5 MPa. The $\sigma_{t,p}$, however, dropped in the range of 56.6 to 150 MPa suction to a value of approximately -2 MPa.
Figure 3.14: Relationship between the Brazilian tensile strength (normal and parallel to bedding (i.e., $\sigma_{t,n}$ and $\sigma_{t,p}$)) and suction. Mean value and data range for each applied suction are given.

Table 3.3: Results of Brazilian tensile strength tests. $\sigma_{t,p}$: Brazilian tensile strength parallel to bedding; $\sigma_{t,n}$: Brazilian tensile strength normal to bedding.

<table>
<thead>
<tr>
<th>Specimen number</th>
<th>Suction</th>
<th>$\sigma_{t,p}$</th>
<th>Specimen number</th>
<th>Suction</th>
<th>$\sigma_{t,n}$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(-) MPa</td>
<td>(MPa)</td>
<td></td>
<td>(-) MPa</td>
<td>(MPa)</td>
</tr>
<tr>
<td>BTS12a</td>
<td>4.1</td>
<td>-0.8</td>
<td>BTS12b</td>
<td>4.1</td>
<td>-0.6</td>
</tr>
<tr>
<td>BTS12c</td>
<td>4.1</td>
<td>-1.1</td>
<td>BTS17a</td>
<td>4.1</td>
<td>-0.5</td>
</tr>
<tr>
<td>BTS17b</td>
<td>4.1</td>
<td>-1.1</td>
<td>BTS22a</td>
<td>4.1</td>
<td>-0.5</td>
</tr>
<tr>
<td>BTS15a</td>
<td>22.0</td>
<td>-2.0</td>
<td>BTS7a</td>
<td>22.0</td>
<td>-0.8</td>
</tr>
<tr>
<td>BTS15b</td>
<td>22.0</td>
<td>-2.2</td>
<td>BTS15b</td>
<td>22.0</td>
<td>-0.8</td>
</tr>
<tr>
<td>BTS15c</td>
<td>22.0</td>
<td>-1.8</td>
<td>BTS29f</td>
<td>22.0</td>
<td>-0.8</td>
</tr>
<tr>
<td>BTS15d</td>
<td>22.0</td>
<td>-1.9</td>
<td>BTS29a</td>
<td>39.0</td>
<td>-1.0</td>
</tr>
<tr>
<td>BTS15e</td>
<td>39.0</td>
<td>-2.7</td>
<td>BTS29b</td>
<td>39.0</td>
<td>-0.7</td>
</tr>
<tr>
<td>BTS15f</td>
<td>39.0</td>
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3.4.4.2 Onset of dilatancy

Figure 3.15 shows the mean axial stress and axial stress range at the onset of dilatancy, with increasing suction (see also Table 3.4) obtained from the volumetric and radial strain response. For capillary suction smaller than 56.6 MPa, \( CI \) increases almost linearly with increasing suction. Beyond 56.6 MPa suction, \( CI \) was almost constant except for the results obtained from specimens at 224.9 MPa suction, which showed a large variability. The increase in \( CI \) with increasing suction (in this study, up to an applied suction of 56.6 MPa) is in agreement with results obtained by Dyke and Dobereiner (1991) on sandstone.

Table 3.4: Results from unconfined compressive strength tests. \( E \): Young’s Modulus; \( \nu \): Poisson’s ratio; \( UCS \): unconfined compressive strength; \( CI \) (vol): onset of dilatancy determined according to Brace et al. (1966); \( CI \) (rad): onset of dilatancy determined according to Lajtai (1974).

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<th>( \nu )</th>
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</table>

\( CI \)-values obtained from specimens with an applied suction of 1.4 MPa were 2 MPa on average. The stress at the onset of dilation obtained in this study is consistent with results reported by Amann et al. (2011) and Amann et al. (2012), which found an average \( CI \)-value of 2 MPa for OPA specimens loaded
normal to bedding. This suggests that \( CI \) is equal for both loading directions. \( CI \) is typically considered as a material property that is stress path independent. However, results obtained in this study show that \( CI \) is a state property and varies substantially with varying suction.

![Graph showing relationship between onset of dilation (CI) and suction. Mean value and data range for each applied suction are given.](image)

**3.4.4.3 Uniaxial compressive strength**

Figure 3.16 shows the relationship between UCS (mean, minimum and maximum for each applied suction) with increasing suction (see also Table 3.4). Three specimens were tested directly after core dismantling and specimen preperation, and the corresponding suction was estimated from the relationship between water content and suction (Figure 3.6), and was 13 MPa on average.

The data shows that the UCS increases almost linearly between 13 and 56.6 MPa suction. Specimens saturated in dessicators with relative humidity of 99\% (i.e. 1.4 MPa suction) revealed a considerably lower UCS. For suction larger than 56.6 MPa, the UCS increases non-linearly, and above a suction of 86 MPa no significant change in UCS can be identified.

The low mean UCS at 1.4 MPa suction is most probably associated with the conditioning of these specimens at a relative humidity of 99\%, which was difficult to maintain. Short-term humidity cycles (i.e. relative humidity variations between 96 and 99\%) were observed. As a consequence, a stable equilibrium could not be reached, and saturation of the specimens exceeded 100\%. Hence, UCS-values obtained from these specimens are not considered representative for 1.4 MPa suction. A back-extrapolation of a linear regression through representative data points between 13 and 56.6 MPa suction suggests a UCS of approximately 9 MPa for fully saturated specimens.
3.5 Discussion

3.5.1 Variations in p-wave velocity

Figure 3.6 and Figure 3.7 suggest that the suction in the specimens is reversible following a hysteresis loop. The p-wave velocity normal to bedding is, however, irreversible suggesting an irreversible alteration of bulk material properties, rather than a direct influence of suction. For the drying path, a sharp decrease in $v_{p,n}$ from 2250 m/s to 1000 m/s between 0 and 20 MPa suction was observed. For suction between 20 and 220 MPa, $v_{p,n}$ was almost constant at a value of 1000 m/s. The sharp decrease in $v_{p,n}$ occurred between 0 and 20 MPa suction and was irreversible for the wetting path. Seismic velocities measured parallel to bedding in the same range of suction were constant at approximately 3000 m/s.

Peron et al. (2009) found from desiccation experiments on saturated fine-grained soils that upon drying, the void ratio decreases sharply at relatively low suction, accompanied by an increase in deformation rate and desiccation damage. With further increase in suction, both, void ratio and deformation were almost constant. The suction at which the deformation rate sharply decreases is called the shrinkage limit. Peron et al. (2009) showed experimentally that the shrinkage limit and air-entry value are similar in magnitudes, suggesting that the majority of desiccation damage is associated with the early stage of desaturation where the suction increases from 0 to approximately the air-entry value. Desiccation cracking typically occurs when drying shrinkage is constraint, and/or tensile stresses in excess of the tensile strength are generated (Corte and Higashi 1960, Peron et al. 2009). The findings of Peron et al.
(2009) are consistent with the findings for \( v_{p,n} \) in this study, which decreased sharply between 0 and 22 MPa. In this stage of desaturation formation of desiccation, macro-cracks were observed for all test specimens. For suction larger than 22 MPa, \( v_{p,n} \) was almost constant at 1000 m/s indicating no further increase in desiccation damage. The transition between the sharp decrease in \( v_{p,n} \) and an almost constant \( v_{p,n} \) is consistent with the air-entry suction which was found to be 22 MPa.

The material tested in this study is anisotropic in strength, with a tensile strength parallel to bedding, which is approximately twice the tensile strength normal to bedding (i.e. tensile strength after dismantling of the specimens, Figure 3.14). The anisotropy in tensile strength may explain the contrasting behavior between \( v_{p,n} \) and \( v_{p,p} \) upon drying. During drying it was consistently observed that desiccation macro-crack solely formed parallel to bedding, suggesting that tensile stresses in excess of \( \sigma_{t,n} \) were generated while \( \sigma_{t,p} \) was not exceeded.

As mentioned above, \( v_{p,p} \) showed no major variation in the range of suction tested in this study (i.e. 1.4 to 224.9 MPa). In the same range of suction, the Young’s modulus and Poisson’s ratio changed substantially from approximately 5 to 25 GPa, and 0.25 to 0.1, respectively (Figure 3.12 and Figure 3.13). Assuming a linear-elastic material and a proportional relationship between dynamic and static elastic properties, there is evidence that the reason for the constant \( v_{p,p} \) throughout the range of suction tested can be related to an opposing evolution of the Young’s modulus and Poisson’s ratio with increasing suction. The relationship between \( v_p \) and the dynamic elastic moduli is given by

\[
v_p = \left[ \frac{E}{\rho \left( 1 - 2\nu^2 \right)} \right]^{\frac{1}{2}}. \tag{3.8}
\]

From Equation (3.8) it can be seen that for an increase in Young’s modulus, the Poisson’s ratio has to decrease to maintain a constant \( v_p \). Even though the material tested in this study can be considered as transversal isotropic, and the relationship between static and elastic moduli is not known for this material, it is reasonable to assume that the independence between \( v_p \) and suction is associated with the disproportional trend in Young’s modulus and Poisson’s ratio and their opposing effect on p-wave velocity.

### 3.5.2 Strength and stiffness increase

Fredlund et al. (1995), Vanapalli et al. (1996) and Fredlund and Vanapalli (2002) analyzed shear strength variation of soils associated with variations in suction. They found that the increase in shear strength with increasing suction is linear up to a suction equal to the air-entry value. For suction beyond the air-entry value the shear strength increase often becomes non-linear depending on the soil type. For highly plastic clays, the increase in shear strength can be linear for a wide range of suction. When the suction approaches the suction at residual water content the strength may remain constant or decrease. Based on their experimental findings and considerations on the effective stress law for shear failure in unsaturated porous media, they demonstrated that variations in shear strength are associated with the net
contributions of the effective normal stress and suction to the shear strength. The relation between soil shear strength and soil water retention characteristics is primarily based on the following equation, where changes in total stress and pore water pressure are handled independently by two stress state variables

$$\tau = c' + (\sigma_n - u_a) \tan \phi' + (u_a - u_w) \beta \tan \phi', \quad (3.9)$$

where $c'$ is the effective cohesion, $u_a$ is the pore-air pressure, $\phi'$ is the effective friction angle, $u_w$ is the pore-water pressure, and $\beta$ represents the decrease in effective stress resistance as suction increases. The factor $\beta$ equals 1 for saturated conditions and decreases with decreasing volumetric water content (Fredlund et al. 1995). Thus, the net contribution of capillary suction to shear strength decreases with decreasing volumetric water content (or wetted area between soil particles as suggested by Vanapalli et al. (1996)). At residual water content, this may cause a drop in shear strength. The above findings are consistent with observations in the present study: the tensile strength, the onset of dilatancy, the unconfined compressive strength, and the stiffness increased almost linearly up to a suction of 56.6 MPa. For suction larger than 56.6 MPa the scatter in the data points suggest only little variations in $C\!I$, $E$, $UCS$, and $\sigma_{t,n}$ whereas $\sigma_{t,p}$ dropped significantly. Due to the scatter in the data, however, the onset of a non-linear strength increase for suction beyond the air-entry value as suggested by Fredlund et al. (1995) could not be identified.

### 3.5.3 State-dependent strength anisotropy

The comparison between the increase in $\sigma_{t,p}$ and $\sigma_{t,n}$ with an increase in suction suggest that $\sigma_{t,p}$ is considerably more affected by changes in suction than $\sigma_{t,n}$. Similar tendencies can be found for the unconfined compressive strength on specimens loaded parallel and perpendicular to the bedding orientation. Amann et al. (2010) used both their own and published data to establish a relationship between water content and $UCS$ of OPA specimens loaded perpendicular to bedding (Figure 3.17). They found that in a range of water content between 8% and 5%, the $UCS$ increases with decreasing water content by a ratio of 5:1. Figure 3.17 also shows $UCS$ data obtained in this study from specimens loaded parallel to bedding. The $UCS$ increases with decreasing water content at a ratio of 1:1 in a range of water content between 7% and 1%. These finding suggest a state dependent strength anisotropy.

As shown in the previous sections desiccation damage evolves at high saturation (i.e. between 100-95%) causing bedding parallel desiccation macro-cracks associated with a sharp drop in $\nu_{p,n}$. Desiccation cracks parallel to bedding can affect $\sigma_{t,n}$, but do not or only marginally affect $\sigma_{t,p}$. However, desiccation cracking occurred at relatively high saturation or suction smaller than the air-entry value. For suction larger than the air-entry value, the data suggest no further damage accumulation (i.e. shrinkage limit). Thus, accumulated damage may not explain the dissimilar strength increase for suction greater than the air-entry suction.
State-dependent anisotropy in hydraulic conductivity of layered materials has been investigated by Mualem (1984), Yeh et al. (1985), and Green and Freyberg (1995). They consistently showed that the anisotropy in hydraulic conductivity increases with increasing suction. The reason for this increase is most probably related to the spatial variability in suction within a layered system, as illustrated in Figure 3.18a (Green and Freyberg 1995). Due to the bedding fabric of OPA, it is likely that the mean applied suction is not uniformly distributed in the specimens (mainly due to variability in pore space distribution). Houben et al. (2013) found from detailed analyses that the representative element area for mineralogical composition and pore size distribution is in the order of $100 \times 100 \, \mu m^2$. For smaller areas considerable variations in pore size distribution exist, suggesting that local variations in suction are to
be anticipated. They also found that the majority of pores are aligned parallel to bedding. According to the results obtained in this study, these variations may cause stiffness and strength contrasts between zones of different pore size distribution, and may influence the effect of suction on strength and strength increase. Cracks in brittle rock types, such as OPA, tend to grow predominately sub-parallel to the maximum applied load (Amann et al. 2011, Amann et al. 2012). For a load applied parallel to bedding, cracks may grow along bedding layers with comparably low suction. If the load is applied normal to bedding growing fractures sub-parallel to the applied load direction may arrest at layers with higher suction and thus strength and stiffness (Amann et al. 2011, Amann et al. 2013, Amann et al. 2014). The differences in crack propagation are illustrated in Figure 3.18b and Figure 3.18c for typical Brazilian tensile strength specimens loaded parallel and perpendicular to bedding. For the latter, the tensile fracture shows a stepped trace, indicating that fracturing is affected by heterogeneities in strength or stiffness between different regions. This is in contrast to fractures that form for tests with the load applied parallel to bedding, where typically a single fracture parallel to bedding formed.

3.6 Conclusions
A series of tests on specimens in equilibrium with various humidity (i.e. calculated suction between 1.4 MPa and 224.9 MPa) were used to investigate strength (unconfined compressive strength, onset of dilatancy, Brazilian tensile strength), elastic properties (Young’s modulus and Poisson’s ratio) and ultrasonic p-wave velocity variations of Opalinus Clay. The results show that the p-wave velocity normal to bedding drops sharply upon desaturation until suction approached the air-entry value. The sharp decrease is associated with desiccation cracks solely oriented parallel to bedding. The p-wave velocity parallel to bedding remained constant in the range of suction investigated in this study. The constant $v_{p,p}$ with increasing suction might be associated with the disproportional decrease of the Poisson’s ratio and $E$, and its opposing effect on p-wave velocity.

An almost linear increase in UCS, $\sigma_t$, CI, and $E$ with increasing suction was observed up to a suction value of 56.6 MPa. For suction exceeding 56.6 MPa, a relatively constant strength and stiffness was observed (except for $\sigma_{t,p}$ which showed a significant drop). The strength/stiffness increase is most probably associated with the net contribution of suction to strength, which decreases with decreasing volumetric water content non-linearly, following the water retention characteristics.

The rate of increase in $\sigma_t$ and UCS with increasing suction is different depending on the rock anisotropy. Both $\sigma_t$ parallel to bedding and UCS obtained from specimens loaded normal to bedding increase with a considerably larger rate when suction increases. The reasons for the different rates in strength increase are most probably associated with local variations in suction (i.e. local suction) as a consequence of the bedded nature of Opalinus clay with various zones of contrasting pore size distribution. These variations in pore size may influence the effect of suction on strength when the applied load is parallel to bedding and crack growth is predominately along bedding layers with comparably low suction.
Acknowledgement

This study was funded by the Swiss Federal Nuclear Waste Inspectorate. We are grateful to Dr. Keith Evans, Dr. Nicola Tisato, Dr. Benoit Valley (all ETH Zurich), and Dr. Paul Bossart (Swisstopo) for the fruitful discussions during execution and interpretation of the test results.

3.7 References


4. Challenges in lab testing of low permeable clay shales

Wild, K.M., Barla, M, Turinetti, G., Amann, F.

Prepared for publication in a modified form

Abstract: In many engineering applications it is important to determine both effective rock properties and the rock behavior which are representative for the problem’s in-situ conditions. For this purpose, rock samples are usually extracted from the ground and brought to the laboratory to perform laboratory experiments such as consolidated drained and undrained triaxial tests. For low permeable geomaterials such as clay shales, core extraction, handling, storage, and specimen preparation can lead to a reduction in the degree of saturation and the effective stress state in the specimen prior to testing remains uncertain. Suction or partly saturated conditions can have a substantial influence on the effective strength properties and stiffness. Therefore, a testing procedure is necessary that allows the re-establishment of full saturation. Re-establishing full saturation requires elevated back pressures and a procedure that allows demonstrating full saturation. Subsequently to a saturation phase, the specimen needs to be consolidated to constitute a uniform effective stress state that is relevant for the problem. After consolidation, a shearing phase is conducted with loading rates that are small enough to capture the specimens pore pressure response during consolidated undrained tests or to minimize pore pressure changes during consolidated drained tests. These three stages are challenging, in particular for clay shales due to their low permeability. In this contribution we describe key aspects that need to be considered during each stage. A theoretical background for each stage is given and a laboratory protocol is derived and applied to Opalinus Clay, a low permeable clay shale that was recently chosen as host rock for high level nuclear waste in Switzerland.
**Notation**

* A, B: Skempton’s pore pressure coefficients
* $c_f$: compressibility of pore fluid
* $c_d$: compressibility of rock skeleton
* $c_v$: coefficient of consolidation
* $\gamma_w$: unit weight of water
* $E_{oed}$: oedometer modulus
* $\eta$: factor
* $h$: half the height of specimen
* $H_s$: Henry’s coefficient of solubility
* $K$: bulk modulus
* $k$: hydraulic conductivity
* $L$: length of longest drainage path
* $n$: porosity
* $\nu$: Poisson’s ratio
* $P\%$: degree of pore pressure equalization
* $p_0$: initial gas pressure
* $R$: radius of specimen
* $S_0$: initial degree of saturation
* $\sigma_1$: major principal total stress
* $\sigma_3$: minor principal total stress
* $\sigma_{ax}$: axial total stress
* $\sigma_{conf}$: confinement
* $t$: time
* $T_v$: time factor for consolidation (or pore pressure equalization)
* $t_f$: time to failure
* $t_{95\%}$: time required to reach consolidation degree of 95%
* $U\%$: degree of consolidation
* $U_f$: pore pressure dissipation at failure
* $u$: pore water pressure
* $u_p$: pore water pressure at peak
* $u_{0f}$: initial pore water pressure
4.1 Introduction

In many engineering applications, such as nuclear waste repository design, conventional and unconventional oil and gas extraction, and CO₂ sequestration, it is of great interest to assess short- and long-term performance of underground structures like wellbores, repository drifts, and caverns. This requires the determination of effective rock properties and the rock behavior which are representative for the problem’s in-situ conditions. In both nuclear waste repository design and oil and gas industry, low permeable argillaceous rocks, especially clay shales are frequently encountered. To quantify the effective strength and to understand the deformation behavior of a clay shale, test specimens have to be extracted from the ground and brought to the laboratory. During this process, the samples will undergo a complex stress path and are exposed to atmospheric conditions. Because of the low permeability of clay shales and usually high drilling and extraction rates, the sampling procedure can be considered as undrained. Therefore, pore water pressure within the sample will drop due to unloading. In an ideal case of sampling (assuming a homogeneous, isotropic elastic, saturated material with a compressibility of the rock matrix which is much lower than that of water), the pore pressure will drop by the same amount as the mean stress changes and the mean effective stress within the sample remains unchanged (i.e. it stays equal to the in-situ conditions). Clay shales, however, exhibit a non-isotropic material behavior. Therefore, the mean effective stress within the extracted samples is likely not comparable to in-situ conditions. In addition, there are various processes that can lead to a further modification of the effective stress such as desaturation due to gas escaping from solution upon unloading, air-entry and capillary effects due to contact with air and desaturation by cavitation (Hight 2003, Pei 2003, Ewy 2015). During storage, core-dismantling and specimen preparation, the saturation of the samples may further change. The degree of desaturation is strongly dependent on the boundary conditions in the storage room or laboratory. The conditions during storage can be controlled by using vacuum-evacuated aluminum foil or wax to seal the samples. During specimen preparation the specimens inevitable have to be exposed to the laboratory environment. The relative humidity in the laboratory will impose a suction on the specimens according to Kelvin’s law. Wild et al. (2015a) showed that desaturation for clay shales occurs rapidly after sample dismantling and exposure to ambient conditions in the laboratory and may be accompanied by desiccation cracks.

Due to the reasons stated above, the saturation degree of a test specimen in the laboratory is likely to be lower than the saturation degree of the rock in situ. Once the specimen’s degree of saturation drops below 100%, the effective stress law for saturated porous media is no longer valid (Jennings and Burland 1962, Bishop and Blight 1963) and the effective stress in the specimen prior to testing remains unknown. Furthermore, the saturation may change during triaxial testing as a consequence of specimen compaction and dilation, which affects the reliability of the test results (Lowe and Johnson 1960, Bishop and Henkel 1962, Bishop and Blight 1963). Therefore, there is a need to re-establish full saturation within the specimens prior to triaxial testing in order to gain test results representative for the in-situ conditions.
Consequently, triaxial testing of low permeable soft rock types such as clay shales requires a multi-stage testing procedure (Chiu et al. 1983, Bellwald 1990, Steiger and Leung 1991, Aristorenas 1992, Taylor and Coop 1993, Horsem an et al. 1993, Ewy et al. 2003, Barla 2008, Bonini et al. 2009, Deng et al. 2011, Yu et al. 2012, Dong et al. 2013, Bésuelle et al. 2013, VandenBerge et al. 2014). In a first stage (saturation stage) the specimens will be saturated and saturation needs to be demonstrated. A second stage (consolidation stage) aims to establish a uniform effective stress state within the specimen. This ensures that the pore pressure that evolves upon shearing is not affected by a transient pressure signal. In a last stage (shearing stage) the loading/strain rate needs to be chosen in such a way that the pore pressure measured at the specimens faces (top and/or bottom) is representative for the pore pressure that develops within the specimen during consolidated undrained (CU) tests or that the pore pressure can be kept almost constant within the specimen during consolidated drained (CD) tests.

Substantial effort has been devoted to the establishment of a testing procedure for low permeable rocks during the past decades (e.g. Lowe and Johnson 1960, Bishop and Henkel 1962, Wissa 1969, Bellwald 1990, Steiger and Leung 1991, Aristorenas 1992, Head 1998, Barla 1999, Vogelhuber 2007, Dong et al. 2013). This contribution summarizes some major key aspects that have to be considered during the individual stages. It compiles both theoretical and data based considerations that allow proper planning of triaxial testing and gives guidelines for quality control. Furthermore, a laboratory testing procedure based on these considerations was applied to a series of Opalinus Clay specimens, a clay shale chosen as host rock for a high-level nuclear waste repository in Switzerland (BFE 2011), are presented and discussed to illustrate the proposed laboratory protocol for low permeable clay shales.

4.2 Theoretical background and ideal testing procedure

4.2.1 Saturation stage

4.2.1.1 Theoretical considerations on the back pressure needed to establish saturation

A flushing phase is considered as a preparatory phase for complete specimen saturation and is used to achieve saturation of the pore pressure lines, considering that the specimen is commonly set-up by the dry setting method, which makes dry circuits mandatory (i.e. no water is in contact with the specimen). Using de-aired water avoids bringing additional gas into the system (i.e. pressure lines and pore space). Furthermore, the use of pore water with a composition similar to the in-situ pore water is recommended since clay shales are prone to chemical reactions that may alter the geomechanical properties. This is especially important for long-lasting tests in order to keep the influence of the pore fluid purely mechanical.

A small pressure gradient is applied between the bottom (inlet) and the top pore pressure circuit (outlet) by leaving the exit valve open. This allows gas to escape from the pore space and from the circuit as pore water permeates the specimen. A confining pressure which exceeds the pore pressure within the
specimen and is large enough to avoid swelling and associated damage of the clay shale structure and diagenetic bonds (i.e. degradation of diagenetic bonds) is mandatory (Barla and Barla 2001, Barla 2008, Wild et al. 2015b).

The saturation procedure requires an increase of back pressure at the specimen faces. This decreases the volume of trapped gas bubbles according to Boyle’s law, which reduces the required time to dissolve the gas (Lee and Black 1972). At the same time, the amount of air which is soluble in water increases according to Henry’s law (Lowe and Johnson 1960). Theoretical relationships between the initial degree of saturation and the required change in back pressure necessary to completely saturate a specimen considering Henry’s law has been given by Bishop and Eldin (1950) and Lowe and Johnson (1960). The theoretical change in back pressure depends on the initial degree of saturation, Henry’s coefficient of solubility and the imposed boundary conditions. Two different methods can be used to achieve full saturation of the specimen: 1) the skeleton of the rock specimen is compressed without any change in water or total air content (i.e. undrained compression, no continuous supply of water) or 2) back pressure is built up and water is allowed to enter the pore space of the specimen (i.e. drained conditions, continuous supply of water). In both cases the pore pressure will increase and therefore more air will be dissolved in the pore water. The increase in pore water pressure $\Delta u$ necessary to saturate the specimen in case 1, starting from an initial degree of saturation $S_0$ including gas at pressure $p_0$, is given by the following equation (Bishop and Eldin 1950)

$$\Delta u = p_0 \left(1 - \frac{S_0}{S_0 H_s}\right), \quad (4.1)$$

where $H_s$ is Henry’s coefficient of solubility which can be taken as 0.02 volume of air per volume of water for room temperature.

For case 2, the pore pressure increase necessary to fully saturate the specimen can be estimated by using the following formula (Lowe and Johnson 1960, rearranged)

$$\Delta u = p_0 \frac{(1 - S_0) (1 - H_s)}{H_s}. \quad (4.2)$$

For simplicity reasons it is assumed for the derivation of equation (4.2) that the volume of the specimen remains constant during an increase of the back pressure with continuous supply of water. This might not be true for clay shales which may have compressible grains or a compressible rock skeleton. Furthermore, equations (4.1) and (4.2) reveal the minimum back pressure values required to achieve full saturation for the assumption that the gas bubbles in the pore space are at atmospheric pressure before applying a back pressure. This assumption might not be valid in case a flushing phase precedes the back pressure stage (Lowe and Johnson 1960).

The back pressure is typically increased in several stages on both specimen faces and is maintained for several hours to days (Lowe and Johnson 1960, Bishop and Henkel 1962, Wissa 1969). The confining
stress is increased simultaneously in such a way as to maintain the effective stress that has been established during the flushing phase.

4.2.1.2 Demonstration of saturation and validity of Skempton’s pore pressure parameter $B$

Skempton’s pore pressure coefficient $B$ is determined between each back pressure stage (so called $B$-check) and can be used to assure saturation of the specimen during the saturation stage. $B$ represents the ratio between a change in pore pressure and a change in confining stress (under undrained conditions) (Skempton 1954). Its value (between 0 and 1) is dependent on the porosity, compressibility of the skeleton, fluid and solid material (Bishop 1966). Gas bubbles in the pore water system or in the specimen will increase the compressibility of the pore fluid and therefore decrease $B$. For an ideal, isotropic porous media, with a compressibility of the mineral skeleton greater than $10^{-7}$ Pa$^{-1}$, $B$ equals unity when the specimen is saturated (Wissa 1969). For many rocks and soils, and especially for clay shales, $B$ can be significantly smaller than unity since the load is partly taken by the rock skeleton because the compressibility of the pore fluid is comparable to the compressibility of the rock skeleton (Skempton 1954, Wissa 1969). Furthermore, the value of $B$ is dependent on the effective confinement that affects the compressibility of the rock skeleton. A decrease in $B$ with increasing effective confinement has widely been observed for various rock types such as sandstone, limestone, marble, granite (e.g. Mesri et al. 1976, Green and Wang 1986, Hart and Wang 1999, Lockner and Stanchits 2002), and shales (Hart and Wang 1999, Mesri et al. 1976, Cook 1999; Wild et al. 2015b). Additionally, the value of $B$ is dependent on the compliance of the testing system (Wissa 1969, Bishop 1976). Pore pressure lines, transducers, and Darcy filters influence the compliance and add porosity to the system. The more compliant the system, the smaller the excess pore pressure that is measured, which decreases the $B$-value (Wissa 1969, Monfared et al. 2011, Hu et al. 2014). Furthermore, an undrained compression tests may lead to an instantaneous pore pressure change measured outside the specimen which is different than the pore pressure change in the specimen (Monfared et al. 2011). In case where the effective volume of the external system is comparable to the pore volume of the specimen, the assumption of an undrained response is not valid anymore (Bishop 1973, Ghabezloo and Sulem 2010, Monfared et al. 2011, Hu et al. 2014). A very rigid external system with a small free volume is therefore crucial. Correction calculation to account for the effects of the system are given by different authors (e.g. Bishop 1976, Bellwald 1990, Ghabezloo and Sulem 2010, Monfared et al. 2011). For the reasons stated above, it is not sufficient to target a high $B$-value (e.g. higher than 0.95) as a standalone criteria for demonstrating complete saturation in low permeable clay shales. A high value of $B$ is, however, an indicator for a high saturation degree.

Due to a high effective confinement, a high compressibility of the system or rock skeleton, $B$ may be smaller (e.g. 0.80) although the specimen is saturated. Wissa (1969) noted that for low permeable rocks, $B$ will stay constant for two subsequent undrained confining stress changes if the specimen is completely saturated. Aristorenas (1992) stated that it is almost impossible to reach identical Skempton’s pore
pressure coefficient $B$ from subsequent $B$-checks. Therefore, he assumed a specimen to be saturated if $B$ does not change significantly ($\Delta B$ in the order of $\pm 0.03$) for two subsequent steps. Hence, for demonstrating full saturation, the absolute value of $B$ and the assessment of the change between two subsequent $B$-checks can be used.

### 4.2.2 Consolidation

Subsequent to the saturation stage, a consolidation stage has to be considered to establish a uniform effective stress state within the specimen prior to shearing. Pore pressure and confinement will be maintained at desired values. The pore pressure valves on one or on both sides are opened and the specimen is allowed to consolidate against a back pressure which is equal or higher than the back pressure applied during the saturation phase in order to maintain full saturation.

The time $t$ theoretically required to consolidate a specimen can be estimated prior to testing from the dimensionless time variable or time factor $T_v$ of the one-dimensional consolidation theory by Terzaghi (1943) using the following general expression

$$T_v = \frac{ct^2}{L^2}, \quad (4.3)$$

where $c_v$ is the coefficient of consolidation and $L$ is the length of the longest drainage path (i.e. the total height of the specimen if drainage is allowed only on the bottom or top, half the height of the specimen if drainage is allowed on both ends of the specimen, and the radius of the specimen if both axial and radial drainage is allowed).

According to Terzaghi’s theory of one-dimensional consolidation (Terzaghi 1943), $c_v$ is related to the hydraulic conductivity $k$, the oedometer modulus $E_{oed}$, and the unit weight of water $\gamma_w$ by the following expression

$$c_v = \frac{kE_{oed}}{\gamma_w}. \quad (4.4)$$

The coefficient of consolidation for an isotropic consolidation in a triaxial cell, however, is not the same as the coefficient of consolidation for a one-dimensional consolidation described by equation (4.4) (Gibson and Henkel 1954, Bishop and Henkel 1962, Head 1998). As an approximation, Head (1998) proposed to use the bulk modulus $K$ instead of $E_{oed}$ in equation (4.4) to derive the coefficient of isotropic consolidation. Assuming an isotropic material with linear elastic behavior, the parameters $K$ and $E_{oed}$ are related by the following relationship

$$\frac{K}{E_{oed}} = \frac{1 + \nu}{3(1 - \nu)}, \quad (4.5)$$

where $\nu$ is Poisson’s ratio.
Alternatively, the coefficient of (isotropic) consolidation can be estimated from experimentally determined time-settlement curves using different approaches (e.g. Bishop and Henkel 1962, Robinson and Allam 1996, Head 1998, Germaine and Germaine 2009). The time factor $T_v$ in equation (4.3) depends on the condition of the consolidation problem with respect to the load and the initial excess pore pressure distribution (Terzaghi 1943). Several curves which relate the time factor $T_v$ to the degree of consolidation $U\%$ for different conditions have been published (e.g. Terzaghi and Fröhlich 1936, Terzaghi 1943). For an isotropic consolidation in a triaxial cell, where the stress is applied from all three directions equally, the specimen consolidates in three dimensions. Therefore, strictly speaking, the three-dimensional theory of consolidation applies. If only vertical drainage via the porous stones at top and bottom of the specimen is permitted, Terzaghi’s one-dimensional consolidation can be applied as an adequate approximation for the time required for full consolidation (Scott 1963). Thus, aiming for a consolidation degree of 95%, the following equation can be used to estimate the required time $t_{95\%}$ to complete consolidation (according to Terzaghi 1943)

$$t_{95\%} = \frac{1.129}{c_v} \frac{h^2}{v},$$

(4.6)

where $h$ is half the height of the specimen.

Filter strips used at the side of the specimen might decrease the time required for full consolidation by allowing radial drainage (Bishop and Henkel 1962, Leroueil et al. 1988, Mitachi et al. 1988). If both vertical and radial drainage is allowed and the specimen height is assumed to be twice its diameter, a combination of the solution for radial drainage and vertical drainage can be used (Gibson and Lumb 1953, Scott 1963)

$$t_{95\%} = 0.37 \frac{R^2}{c_v},$$

(4.7)

where $R$ is the radius of the specimen.

In practice the consolidation degree is usually controlled by examining time-dependent variations in volumetric strain. Furthermore, the change in water content due to excess pore pressure dissipation can be analyzed. The consolidation of the specimen is considered sufficient when the strain approaches a constant value and the water content remains constant.

4.2.3 Shearing stage

Challenges in undrained and drained shearing of consolidated, saturated clay shale specimens mostly arise from the strong hydro-mechanical coupling. During drained shearing, the shearing rate has to be slow enough to avoid significant excess pore pressure build-up within the specimen, especially at peak strength. The choice of an appropriate loading/strain rate is therefore crucial to avoid heterogeneous pore pressure distributions and to measure reliable drained properties. During undrained shearing, pore pressures need to be measured to determine the effective strength. In principle, the load can be applied
faster than for drained tests. However, to measure the excess pore pressure close to the end faces of the specimens, a sufficiently slow loading/strain rate is required that allows for a redistribution of pore pressure within the specimen in order to measure a pore pressure representative for the bulk specimen. The time required to reach the peak strength can be estimated based on the theory of consolidation and can further be used to determine an appropriate loading/strain rate when the load or strain at failure is known (Bishop and Henkel 1962). These are usually not known in advance, which adds an uncertainty to the estimation of the appropriate loading/strain rate, in addition to the uncertainties in estimating the time that is required to dissipate any excess pore pressure or equalize nonuniform pore pressure within the specimen.

4.2.3.1 Theoretical considerations on the loading/strain rate

The loading/strain rate for a CU test is chosen in such a way as to allow a reliable measurement of pore pressure throughout the test. Clay shales tend to compact or dilate during shearing causing the pore pressure to change. Pore pressure transducers are normally connected via filter material and drainage lines close to the specimens top and/or bottom end faces. Due to their compliance, minor volume changes are possible causing a small fluid flow in or out of the specimen (Bishop and Henkel 1962, Wissa 1969, Monfared et al. 2011, Hu et al. 2014). This is necessary to achieve pore pressure equalization between the specimen and the drainage system. The time needed for this equalization depends on the compliance of the drainage system. The higher the compressibility of a system, the longer it takes to redistribute pore pressure changes induced by shearing and hence the loading/strain rate has to be reduced to measure pore pressures representative for the bulk behavior of the specimen (Whitman et al. 1961, Wissa 1969, Monfared et al. 2011). Furthermore, in a standard triaxial compression test, the pore pressure at the ends of the specimen will be slightly higher than at its center due to unequal loading caused by boundary effects (Blight 1963, Peng 1971). The equalization of the pore pressure within the specimen depends on its permeability, its dimensions, and loading/strain rate (Bishop and Henkel 1962). From experimental test result from different shales and clays, Blight (1963) concluded that for an overconsolidation ratio up to 20, a degree of pore pressure equalization (P%) of 95% is sufficient to avoid an error in effective confining stress bigger than 5%. Gibson (published in Bishop and Henkel 1962) gives theoretical relationships between P% and the time factor T (as defined in equation (4.3)). The underlying assumptions are that the specimen is subject to a constant loading/strain rate, the pore pressure distribution over the height of the specimen is parabolic, the difference in pore pressure is proportionally related to the magnitude of the stress increment, and that there is no difference between the coefficient of consolidation and the coefficient of swelling. From these theoretical relationships and for the assumption that 95% pore pressure equalization are sufficient, the test duration for CU tests (i.e. time to failure t_f) on specimens without lateral drainage can be estimated by
\[ t_f = 1.67 \frac{h^2}{c_v}. \] (4.8)

For a CU test on a specimen with additional lateral drains (e.g. filter strips) and a height which is twice its diameter, the following equation can be used

\[ t_f = 0.07 \frac{h^2}{c_v}. \] (4.9)

In theory, the loading/strain rate for CD tests has to be infinitely small to avoid any pore pressure build-up. In practice, this is not possible and a small excess pore pressure will always be created. To avoid any influence of this pore pressure build-up on the measured strength, it is important that the excess pore pressure at peak strength is negligible. Based on the theory of consolidation, Gibson and Henkel (1954) developed a theoretical expression that allows an estimate of the time required to reach peak strength and thus the loading/strain rate for CD tests. The average degree of pore pressure dissipation at failure \( U_f \) (which is equal to the degree of consolidation) can be related to the time to failure as follows:

\[ U_f = 1 - \frac{h^2}{\eta c_v t_f}, \] (4.10)

where \( \eta \) is a factor depending on the drainage conditions (values given in Gibson and Henkel 1954). Based on experiments on different clays, Gibson and Henkel (1954) stated that 95% of pore pressure dissipation is sufficient to consider a test drained. For a CD test on a specimen where drainage from both ends are permitted, the time to failure can therefore be estimated by

\[ t_f = 6.67 \frac{h^2}{c_v}. \] (4.11)

The estimation of time to failure for a specimen with drainage from both ends and at the radial boundary (e.g. via filter strips) can be approximated by

\[ t_f = 0.57 \frac{h^2}{c_v}, \] (4.12)

assuming that the height of the specimen is twice its diameter.

In case the pore pressure is to be monitored during the CD tests, only drainage at one side of the specimen can be provided. The correspondent relationship for the time to failure for a specimen under these test conditions is given by

\[ t_f = 26.7 \frac{h^2}{c_v}. \] (4.13)

### 4.2.3.2 Practical considerations on the loading/strain rate

The above considerations are solely based on theoretical considerations and uncertainties remain in the choice of an adequate loading/strain rate. A more robust, but very time-consuming way to derive an appropriate loading/strain rate for CU test is a series of triaxial tests which utilizes rates that vary one to
two orders of magnitude (e.g. \(10^{-6} \text{s}^{-1}, 10^{-7} \text{s}^{-1}, 10^{-8} \text{s}^{-1}\)). Such a test series requires full saturation of the specimens. Furthermore, the material characteristics of the different specimens used for evaluation should be comparable. The loading/strain rate is adequate for CU tests on similar test material and with similar specimen dimensions if the pore pressure response does not change between two tests that utilize different loading rates.

To check if the loading/strain rate for a CU test is appropriate, pore pressure magnitudes that evolve during elastic shearing can be examined using Skempton’s pore pressure coefficients \(A\) and \(B\). The change in pore pressure \(\Delta u\) can be expressed as follows (Skempton 1954)

\[
\Delta u = B \left( \Delta \sigma_1 + A \left( \Delta \sigma_1 - \Delta \sigma_3 \right) \right),
\]

where \(\Delta \sigma_1\) and \(\Delta \sigma_3\) are changes in the principal stresses. For a standard triaxial test, where only changes in axial stress \(\sigma_{ax}\) (i.e. \(\Delta \sigma_1 = \Delta \sigma_{ax}\)) and no changes in confinement \(\sigma_{conf}\) (i.e. \(\Delta \sigma_3 = \Delta \sigma_{conf} = 0\)) occur, equation (4.14) reduces to

\[
\Delta u = AB \Delta \sigma_{ax}.
\]

The value of \(A\) for overconsolidated clays typically lies between 0.25 and 0.5 (Skempton and Bjerrum 1957). For an isotropic, perfectly elastic material, \(A\) equals 1/3 (Skempton 1954). Note that \(A\) depends on the magnitude of the applied stress and is often reported as \(A\) at failure. This is not the case in Skempton and Bjerrum (1957) who report \(A\)-values representative for the elastic response. \(B\) can be taken at the end of the saturation phase (i.e. on a saturated specimen). However, depending on the confinement used during triaxial shearing, the dependency of \(B\) on the effective confinement needs to be considered. Alternatively, \(B\) can also be estimated from equation (4.16), neglecting the compressibility of the solid constituents

\[
B = \frac{1}{1 + n \frac{c_f}{c_d}},
\]

where \(n\) is the porosity, \(c_f\) the compressibility of the pore fluid, and \(c_d\) the compressibility of the rock skeleton (i.e. drained compressibility).

A theoretical value \(AB\) can be calculated and used as an indicator for a correct loading/strain rate. If \(AB\) is significantly lower than the theoretical value, the loading/strain rate is too fast to capture the actual pore pressure response of the specimen. Note that this criterion can only be used if the saturation phase and the consolidation phase are complete. Incomplete saturation or nonuniform distribution of pore pressure during shearing would reduce the \(AB\)-value substantially.

During CD tests and especially at peak strength, the pore pressure build-up should be insignificant. If pore pressure is measured during the test, this can be used as a data based criteria for the adequacy of the loading/strain rate. The ratio between pore pressure at peak \(u_p\) and the initial pore pressure \(u_0\) should be close or equal to one. If it significantly differs from unity, excess pore pressure developed and the
peak strength value is not representative for drained condition. Additionally, the pore pressure evolution throughout the test needs to be minimized. This is of particular relevance for dilatant materials such as clay shales. In such materials, positive excess pore pressure that develop during the early phase of differential loading under undrained conditions might be reduced as a consequence of dilation and the ratio \( u_p/u_0 \) might be unity at peak stress. In such a case, a difference between CU and CD tests can therefore not be identified.

4.3 Example data set

4.3.1 Material description

Examples of CU and CD tests conducted on Opalinus Clay are taken to illustrate the challenges arising when testing low permeable clay shales. Opalinus Clay was deposited in a shallow marine environment about 180 million years ago. The samples used for this study have been cored in the shaly facies at the Mont Terri Underground Rock Laboratory (URL) in Switzerland. The main mineralogical constituent of the shaly facies at the Mont Terri URL are clay minerals (30-80%), quartz (10-30%), carbonates (5-20%), and feldspar (0-5%) (Thury and Bossart 1999, Klinkenberg et al. 2003, Bossart 2005). The recent overburden at the Mont Terri URL is about 250 m but it is estimated to have reached about 1350 m in the past (Mazurek et al. 2006). The Opalinus Clay can therefore be considered as overconsolidated. Due to its complex history of sedimentation, burial, physical compaction, development of diagenetic bonding, tectonic faulting, uplift, and erosion, Opalinus Clay shows a pronounced bedding (Marshall et al. 2005, Van Loon et al. 2004). Its physical behavior is therefore often considered as transversely isotropic. The hydraulic conductivity varies between the order of \( 10^{-12} \) m/s and \( 10^{-14} \) m/s depending on the orientation with respect to bedding and the confining stress (Marschall et al. 2004). The water loss porosity (calculated from weight loss at 105°C and grain density) varies between 12 and 18% (Thury and Bossart 1999).

4.3.2 Sample extraction, specimen preparation and setup

4.3.2.1 Core sampling

The samples were taken from two boreholes (BHM-1, BHM-2) with a core diameter of 67.5 mm. The 25 m long boreholes were oriented parallel (BHM-1) and normal (BHM-2) to bedding. Triple tube core barrel technique with pressurized air cooling was used to obtain high quality cores. Small core pieces of BHM-1 at different depths were used to determine the water content of the core samples after drilling and core extraction. The water content was determined according to ISRM suggested methods (ISRM 1979). The individual water contents are listed in Table 4.1. They are, except for one sample, consistent with the water content reported in literature (6.0-8.6%, e.g. Pearson et al. 2003).
Table 4.1: Water content of core pieces tested directly after drilling.

<table>
<thead>
<tr>
<th>specimen</th>
<th>depth (m)</th>
<th>water content (%)</th>
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</thead>
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<td>7.5</td>
</tr>
<tr>
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<td>12.35</td>
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<tr>
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<td>~ 17.70</td>
<td>5.3</td>
</tr>
<tr>
<td>7</td>
<td>~ 20.40</td>
<td>8.1</td>
</tr>
</tbody>
</table>

4.3.2.2 Sample storage and preparation

The cores were covered by a plastic tube, sealed immediately after core extraction in vacuum-evacuated aluminum foil, and stored in wooden boxes. After arrival in the laboratory, the cores were stored at constant humidity and temperature. To avoid undesired desaturation, which is often accompanied by desiccation cracks (Wild et al. 2015a), the specimen preparation procedure was optimized and reduced to 20-30 minutes. Smooth and precise cutting of the sub-sample into specimens with a length of about 135 mm was obtained by using a diamond band saw (Proxxon, Model MBS 240/E) that operates with pressurized air instead of water cooling. The feed rate was manually controlled. A two trails system was designed and manufactured to allow for cutting the edges of the specimen along a planar surface, perpendicular to the core axis.

After preparation the specimen was measured, weighted, photographed, and placed in the triaxial cell. Eight strips and two circles of filter paper were placed on the specimen’s side and top/bottom faces, respectively, as shown in Figure 4.1a before a rubber membrane (2 mm thick) was put over the specimen. O-rings were put in place for appropriate sealing at the top cap and at the pedestal (Figure 4.1b) while the in- and outlet at the top and bottom were closed to avoid air entering the specimen. At this point the specimen was sealed and isolated from the laboratory environment and the following steps (mounting local displacement transducers, connecting pore pressure circuit, closing the pressure vessel, etc.) could be performed.

Two different orientations of specimens were distinguished: P-specimens, where the axial load was applied parallel to bedding, and S-specimens, where the axial load was applied normal to bedding.
4.3.2.3 Water content and saturation degree

The water content and saturation degree determined after sample extraction (Table 4.1) was compared to the values determined after specimen preparation (Table 4.2) to quantify any severe desaturation that may have taken place during specimen preparation. The water content given in Table 4.2 was determined with respect to the weight of the specimens after 24h drying at 105°C. Specimens ETH10_2 and ETH20_2 were dried to constant weight. The comparison between the water content calculated after 24h drying and after drying to constant weight (reached after about 2 days) for these specimens revealed that the water content after 24h probably underestimates the water content by about 0.4 %. Similar results have been observed for specimens tested by Amann et al. (2011) and Amann et al. (2012). It can be seen that most specimens show a water content that lies slightly below the values measured after drilling (Table 4.1) but within the variability reported in the literature (6.0-8.6%, e.g. Pearson et al. 2003). The water content changed even though the preparation procedure was optimized and required only 20 to 30 min. This decrease in water content might be significant in terms of strength/stiffness (Wild et al. 2015a).

The water content after saturation and consolidation given in Table 4.2 was estimated using the weight of the specimen after saturation/consolidation (calculated by adding and subtracting the measured amount of water flown into or out of the specimen during saturation phase and consolidation phase, respectively, to the initial weight before testing). The dry density, porosity, and saturation were determined according to the ISRM suggested methods (ISRM 1979). Grain density for Opalinus Clay ranges between 2.69 and 2.78 g/cm³ (Pearson et al. 2003, Bossart 2005, own data). A mean value of 2.73 g/cm³ was considered in this study to calculate the porosity. Note that the estimated values of saturation strongly depend on this parameter.
Table 4.2: Properties of specimens.

<table>
<thead>
<tr>
<th>Test specimen code</th>
<th>Specimen orientation</th>
<th>Diameter before test (mm)</th>
<th>Height before test (mm)</th>
<th>Water content before test (%)</th>
<th>Dry density (g/cm³)</th>
<th>Porosity (%)</th>
<th>Saturation before test (%)</th>
<th>Water content after saturation (%)</th>
<th>Water content after consolidation (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ETH08</td>
<td>S</td>
<td>67.68</td>
<td>123.74</td>
<td>7.5</td>
<td>2.26</td>
<td>17.2</td>
<td>98.6</td>
<td>8.3</td>
<td>8.2</td>
</tr>
<tr>
<td>ETH09</td>
<td>S</td>
<td>67.74</td>
<td>135.88</td>
<td>7.5</td>
<td>2.27</td>
<td>17.1</td>
<td>98.9</td>
<td>7.9</td>
<td>7.7</td>
</tr>
<tr>
<td>ETH10_2</td>
<td>S</td>
<td>67.59</td>
<td>133.89</td>
<td>7.4</td>
<td>2.27</td>
<td>17.0</td>
<td>98.9</td>
<td>7.9</td>
<td>7.8</td>
</tr>
<tr>
<td>ETH11</td>
<td>P</td>
<td>67.45</td>
<td>135.31</td>
<td>6.2</td>
<td>2.29</td>
<td>16.2</td>
<td>87.7</td>
<td>6.7</td>
<td>6.6</td>
</tr>
<tr>
<td>ETH12</td>
<td>P</td>
<td>67.1</td>
<td>134.49</td>
<td>6.3</td>
<td>2.29</td>
<td>16.2</td>
<td>89.1</td>
<td>6.9</td>
<td>6.7</td>
</tr>
<tr>
<td>ETH13</td>
<td>P</td>
<td>67.4</td>
<td>133.09</td>
<td>6.4</td>
<td>2.28</td>
<td>16.5</td>
<td>88.1</td>
<td>7.0</td>
<td>6.6</td>
</tr>
<tr>
<td>ETH14</td>
<td>P</td>
<td>67.47</td>
<td>134.15</td>
<td>7.0</td>
<td>2.26</td>
<td>17.1</td>
<td>92.6</td>
<td>8.0</td>
<td>7.3</td>
</tr>
<tr>
<td>ETH16</td>
<td>P</td>
<td>67.47</td>
<td>133.00</td>
<td>7.0</td>
<td>2.28</td>
<td>16.6</td>
<td>96.2</td>
<td>8.1</td>
<td>7.8</td>
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<td>ETH17</td>
<td>P</td>
<td>67.47</td>
<td>135.35</td>
<td>6.4</td>
<td>2.27</td>
<td>16.8</td>
<td>86.6</td>
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<td>ETH19</td>
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<td>67.50</td>
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<td>16.6</td>
<td>83.4</td>
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<td>7.1</td>
</tr>
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<td>ETH20_2</td>
<td>P</td>
<td>67.65</td>
<td>133.3</td>
<td>7.2</td>
<td>2.26</td>
<td>17.5</td>
<td>92.7</td>
<td>7.7</td>
<td>6.5</td>
</tr>
</tbody>
</table>

4.3.2.4 Triaxial apparatus

The triaxial tests in this study have been conducted at the DIPLAB Geomeccanica laboratory of the Politecnico di Torino (Italy) using two triaxial apparatus (HPTA, high pressure triaxial apparatus and MPTA, medium pressure triaxial apparatus) which were manufactured by GDS Instruments Ltd. and modified in the laboratory (Figure 4.2) (Barla et al. 2010).

Figure 4.2: The two triaxial apparatus used for testing: a) medium pressure triaxial apparatus (MPTA) and b) high pressure triaxial apparatus (HTPA).

Axial load/displacement, radial pressure/displacement and back pressure/volume applied to the specimen can be controlled individually. The two machines differ in the maximum possible confining pressure/load/back pressure that can be applied. For the MPTA machine, confining pressures up to

83
20 MPa, back pressures up to 16 MPa, and axial loads to maximum of 64 kN can be applied. For the HPTA machine, confining pressures up to 64 MPa, back pressure up to 20 MPa and deviatoric axial loads up to 250 kN are possible. The axial displacement is determined externally with an accuracy of 1 μm by measuring the displacement of the sliding plate of the loading frame. Additionally, radial and axial displacement were measured locally by a set of LVDTs (i.e. two axial transducers which are diametrically opposed and a radial transducer which is mounted on a belt at half the specimen’s height) that were glued on the membrane (Figure 4.3). The accuracy of the vertical measurements is 1 μm on a full scale of 10 mm, the accuracy of the radial measurement is 0.5 μm on a full scale of 5 mm. The load can be applied and measured with a load cell placed between the upper horizontal beam of the loading frame and the top cap (accuracy of 60 N). Pore pressure changes can be measured by a transducer located in the back pressure controller, which is connected to the bottom end of the pore pressure circuit, and by an external transducer which is directly connected to the top end of the pore pressure circuit, immediately after its exit from the triaxial cell (accuracy of 8 kPa).

4.4.1 Saturation stage
De-aired water with composition similar to the in-situ pore water at Mont Terri URL (according to the recipe by Pearson 2002) was used. At the inlet the back pressure was increased in several steps (Figure 4.4). Back pressures between 0.11 and 0.35 MPa were utilized. The outlet was at 1 atm. The confining stress was always higher than the back pressure and large enough to avoid specimen swelling.
and ensure tightness. The stresses were increased in several steps. On average, the duration of the flushing phase was between 1.1 and 7.0 days.

Figure 4.4: Example of pressure increase during the flushing phase for test ETH11.

Subsequently to the flushing phase the specimen was further saturated by increasing the back pressure at both specimen’s faces. Figure 4.5 shows the typical evolution of back pressure and confining pressure during the saturation phase. Back pressure was increase in several steps. Before each back pressure increase, the valves were closed and saturation was checked by performing $B$-checks. Back pressure phases lasted hours to days whereas equilibration during $B$-checks was reached within about 1-2 hours. The whole saturation phase took several days to weeks. The resulting curves for the individual $B$-checks are shown in Figure 4.6. The specimen was assumed to be saturated if the $B$-value was sufficiently high and didn’t change significantly by more than ± 0.03 for two subsequent $B$-checks. This criterion is valid for all specimens shown in Figure 4.6 except for specimens ETH08 ($\Delta B=0.08$), ETH10_2 ($\Delta B=0.05$), and ETH16 ($\Delta B=0.06$). Although these latter specimens show a change in $B$-value greater than 0.03, the $B$-values itself are sufficiently high (i.e. higher than 0.9). Therefore, specimens ETH08, ETH10_2, and ETH16 are considered to be saturated.
Figure 4.5: Example of saturation phase with its back pressure stages (indicated by numbers) and performed B-checks in between (highlighted in grey) for test ETH14.

Figure 4.6: Values of Skempton’s pore pressure coefficient B obtained for the individual B-checks during the saturation phase.

The assessment of the saturation phase based on the test results is in agreement with theoretical considerations. Figure 4.7 shows the theoretical curve for the minimum back pressure needed to saturate a specimen depending on its initial degree of saturation. The theoretical curve is based on equation (4.2) and related assumptions. Also plotted are the relationships between the initial saturation degrees (see Table 4.2) and the maximum changes in back pressure applied to the specimens. The error bars show the saturation range due to the uncertainties in grain density (a value of 2.73 ± 0.03 g/cm³ was used for
calculation here). It can be seen that all specimens were subject to a back pressure higher than the theoretical pressure necessary to reach saturation.

![Figure 4.7: Back pressure change applied to the individual specimens (symbols) compared to the theoretical pressures required to saturate the specimen to 100% (line) calculated using equation (2). The error bars show the saturation range due to uncertainties in grain density.](image)

### 4.4.2 Consolidation stage

For consolidation, the confinement and back pressure on both end faces of the specimen were increased within 24h to establish a target effective stress of 0.5, 0.75, 1.0, 2.0 or 4.0 MPa. The consolidation stage ranged from 48 to 211 hours in case of P-specimens and from 75 to 453 hours in case of S-specimens. At the end of consolidation, either the strain or the change in back volume were constant, indicating complete consolidation. A typical curve of a completely consolidated specimen is shown in Figure 4.8.

Ferrari and Laloui (2012) reported values of $c_v$ from an oedometer test for an S-specimen of Opalinus Clay from the Mont Terri URL ranging between 0.06 and 0.2 mm$^2$/s for an effective vertical stress between 1 and 4 MPa. Assuming a Poisson’s ratio of 0.2 for Opalinus Clay in equation (4.5), the corresponding $c_v$ for an isotropic consolidation ranges between 0.03 and 0.1 mm$^2$/s. Using equation (4.6), the resulting time theoretically required to consolidate a S-specimen is 14-48 hours. However, the load that is applied at the beginning of the consolidation stage is ramped up over a time of 24h. According to Olson (1977), the required time for consolidation due to ramp loading is estimated to range between 30 and 68h. This is less than the actual observed time required for consolidation. The value of $c_v$ for a P-specimen is unknown and therefore no theoretical time can be estimated.
4.4.3 Shearing stage

CU tests were carried out using constant axial strain rates between 0.88*10^{-6} s^{-1} and 1.25*10^{-6} s^{-1} for P-specimens and between 1.23*10^{-7} s^{-1} and 1.35*10^{-7} s^{-1} for S-specimen. To check the adequacy of the strain rates, the $A\bar{B}$-values measured in the elastic region (i.e. at low differential stress) are given in Table 3. $A$ was expected to range between 0.25 and 0.50. For $B$, the actual measured values (0.80-0.97) were utilized. The resulting $A\bar{B}$-values range between 0.20 and 0.49. The actual measured values range between 0.15 and 0.69 (Table 4.3) and are in agreement with $A\bar{B}$-values expected for fully saturated specimens. The chosen axial strain rate can therefore be assumed to be adequate for a CU test.

Table 4.3: Coefficient $A\bar{B}$ measured for the different CU tests.

<table>
<thead>
<tr>
<th>specimen</th>
<th>$A\bar{B}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>ETH08</td>
<td>0.49</td>
</tr>
<tr>
<td>ETH09</td>
<td>0.58</td>
</tr>
<tr>
<td>ETH10_2</td>
<td>0.69</td>
</tr>
<tr>
<td>ETH16</td>
<td>0.33</td>
</tr>
<tr>
<td>ETH17</td>
<td>0.15</td>
</tr>
<tr>
<td>ETH19</td>
<td>0.27</td>
</tr>
<tr>
<td>ETH20_2</td>
<td>0.26</td>
</tr>
</tbody>
</table>

For the CD tests in this study axial strain rates of 1.25*10^{-8} s^{-1} and 1.28*10^{-8} s^{-1} for tests ETH11 and ETH12 and an axial strain rate of 2.53*10^{-8} s^{-1} for tests ETH13 and ETH14 were used. Pore pressure
was measured by closing the upper valve. The valve close to the bottom of the specimen was opened to allow the specimen to drain. Back pressure at the bottom of the specimen was kept at the initial value of about 2.1 MPa. Figure 4.9 shows the pore pressure evolution for all drained tests. The maximum pore pressure developed during shearing before peak strength does not exceed 10% of the initial pore pressure. In Figure 4.10, the pore pressure at peak \( u_p \) normalized by the initial pore pressure \( u_0 \) is plotted for the different tests. Also presented are the values for CU tests conducted on P-specimens. Whereas the ratio \( u_p/u_0 \) for the CU tests increases with increasing confinement, it stays within ± 5% of \( u_0 \) for CD tests (except for ETH14 which exceeds \( u_0 \) by 7%).

![Figure 4.9: Pore pressure evolution during CD tests on P-specimens.](image1)

![Figure 4.10: Pore pressure at peak \( u_p \) compared to initial pore pressure \( u_0 \) as a function of the confining stress \( \sigma_{\text{conf}} \). The grey area represents the 5% interval around \( u_p/u_0 =1 \).](image2)
The assessment of the strain rates based on the test results is in agreement with theoretical considerations. Assuming a $c_v$ of 0.03-0.1 mm$^2$/s for S-specimens, an axial strain at peak strength of 0.9-1.3% (Amann et al. 2012), and using equation (4.8), an axial strain rate between $0.36 \times 10^{-7}$ s$^{-1}$ and $1.71 \times 10^{-7}$ s$^{-1}$ is theoretically required for CU-tests on S-specimens. This is in the same range as the axial strain rate for S-specimen (i.e. $1.23 \times 10^{-7}$-$1.35 \times 10^{-7}$ s$^{-1}$) that has been used in this study and which was proven to be adequate by assessing the $AB$-value. For P-specimens, the $c_v$ remains unknown and thus no conclusion above the theoretical time required for CU and CD tests on P-specimens can be given.

4.5 Conclusions

To acquire reliable poroelastic and effective geomechanical parameters for low permeable geomaterials which are representative for in-situ conditions, a testing procedure is required that allows the establishment of full saturation, completion of consolidation, and that utilizes a loading/strain rate that is slow enough to capture pore pressure changes during undrained loading or maintaining the pore pressure during drained loading within valid bounds. Such a laboratory testing procedure was applied to a series of triaxial tests on Opalinus Clay.

The main conclusions are:

(1) The water and saturation loss during sampling (including drilling operation, core extraction) can hardly be avoided. It is related to pore pressure decrease due to unloading which depends on the in-situ stresses and the material behavior. Saturation loss may occur due to gas escaping from solution, air-entry and/or cavitation. This unavoidable desaturation is the main reason for the necessity of a laboratory testing procedure that allows to back-saturate the specimen. Desaturation due to storage and specimen preparation can be minimized (but not avoided) by limiting the time the specimen are exposed to the laboratory environment. Vacuum evacuated aluminum foil or sealing with paraffin wax was shown to meet this requirements. Measuring water content of samples right after drilling and comparing it to the water content of the specimen after preparation can be used as an indicator of the saturation state of the test specimen.

(2) A flushing phase should be conducted to saturate the testing equipment and to remove large air bubbles from the specimen. A small back pressure of maximum 0.5 MPa is applied at the bottom of the specimen while the top is connected to the atmosphere. A confining stress exceeding this pore pressure and which is large enough to avoid swelling deformations and flow in the interface between membrane and specimen was used. In this study, 1.1-7.0 days were allotted for this phase.

(3) Saturation within the specimen is established by increasing the back pressure and allowing gas to solve within the pore water. Subsequent tests for Skempton’s pore pressure coefficient $B$ should be used to confirm full saturation. For rocks with $B < 1.0$ (i.e. the compressibility of the pore fluid is comparable to the compressibility of the rock matrix), the specimen can be considered saturated when the $B$-value is sufficiently high and two subsequent $B$-values do not differ more than
approximately 0.03. Up to two weeks were required in this study to complete the saturation stage. The back pressure required to saturate the specimen depends on its initial saturation degree and can be estimated prior to testing and used as an indicator for the state of saturation during the saturation phase. However, the estimation of the initial saturation is strongly dependent on the grain density and therefore prone to uncertainties. In combination with the criteria \( \Delta B = \pm 0.03 \) for two subsequent \( B \)-checks, the saturation of the specimen can be confirmed.

(4) The consolidation aims at establishing a uniform effective stress state within the specimen prior to testing. The time needed to reach a sufficient degree of consolidation (suggested is 95%) can be estimated using Terzaghi’s theory of one-dimensional consolidation (if specimen is allowed to drain at the end facies) or by the equation suggested by Gibson and Lumb (1953). Factors like loading time, drainage conditions at the boundaries of the specimen (including the effectiveness of filter paper), and the consolidation coefficient significantly influence the estimation of the required consolidation time. Due to uncertainties in estimating the required consolidation time, an assessment of the time dependent evolution of strains and volume of water flowing out of the specimen provides a more reliable criteria for the completeness of the consolidation phase. The specimen is considered to be sufficiently consolidated if strains and water volume are constant. 48-211 h were necessary for P-specimens and 75-453 h for S-specimens to assure complete consolidation in this study.

(5) The loading/strain rate for CU tests should be sufficiently slow to allow the measurement of pore pressure changes at the end faces of the specimen which are representative for the bulk pore pressure evolution in the specimen during shearing. Estimation of the loading/strain rate prior to testing can be based on the theory of one-dimensional consolidation. Uncertainties in the coefficient of consolidation, however, affect a proper estimation of the loading/strain rate. The measurement of the product of Skempton’s pore pressure coefficients \( AB \) was used in this study as a data based assessment criteria for the loading/strain rate. Both theoretical considerations and the quantities of \( AB \) suggest that the utilized axial strain rates in the order of \( 10^{-6} \) to \( 10^{-7} \) s\(^{-1} \) were adequate for a reliable CU test on Opalinus Clay.

(6) For CD tests, the loading/strain rate can also be estimated prior to testing based on the theory of one-dimensional consolidation. 95% dissipation of pore pressure at peak is sufficient to consider the test drained. Again, the coefficient of consolidation is a key factor that influences the estimation of the loading/strain rate. As a data based assessment criteria, the pore pressure should be monitored during testing and excess pore pressure developing during shearing should be kept minimal. The peak strength value can be considered representative for a drained test when no significant excess pore pressure developed (i.e. the ratio \( u_p/u_0 \) is 1±5%). Examination of the pore pressure response of the tests conducted in this study showed that pore pressure build-up during shearing could be minimized and no significant excess pore pressure was measured at peak strength. The axial strain rates in the order of \( 10^{-8} \) s\(^{-1} \) were adequate for reliable CD tests on P-specimens of Opalinus Clay.
Acknowledgement

This study was funded by the Swiss Federal Nuclear Safety Inspectorate ENSI. We thank Dr. Martin Vogelhuber (Dr. von Moos AG, Zurich) for the thorough review and very useful suggestion that greatly improved the manuscript. We would also like to show our gratitude to Dr. Richardus R. Bakker and Dr. Alba Zappone (both ETH Zurich) for their help with the helium pycnometer. We are also grateful to Dr. Matthew Perras (ETH Zurich) for the fruitful discussions.

4.6 References


5. Confinement and anisotropy controlled pore pressure response and effective geomechanical properties of Opalinus Clay

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Abstract: Consolidated drained and undrained tests with pore pressure measurements were conducted on back-saturated specimens of a clay shale to characterize the influence of confinement and anisotropy on the pore pressure response and effective geomechanical properties (i.e. Young’s modulus, stress at onset of dilation, peak and ultimate strength). Opalinus Clay, a clay shale chosen as host rock for high level nuclear waste in Switzerland was utilized. The result showed that the Skempton’s pore pressure parameters $A$ and $B$, the stress at the onset of dilation, and the Young’s modulus depend on the confinement. Additionally, a change in behavior of the material was observed at effective consolidation stresses between 5 and 8 MPa. The specimens at lower effective consolidation stresses (i.e. heavily overconsolidated specimens) showed a dilatant behavior in the pre-peak region and a significant post-failure stress drop. Specimens consolidated at higher effective consolidation stresses (i.e. slightly overconsolidated to normally consolidated specimens) showed compaction from initial loading until post-peak and a brittle-ductile post-failure behavior. These observations were manifested in pore pressure response curves, effective stress paths and stress-strain curves. Furthermore, they could be correlated to a non-linear appearance of the peak strength failure envelope. An explanation for the non-linear failure envelope related to the dilatant structure of the material is suggested.
5.1 Introduction

Clay shales are encountered in many engineering applications, including nuclear waste repository design, conventional and unconventional oil and gas extraction, and sequestration of CO₂. Commonly, safety and stability issues of underground excavations such as boreholes, tunnels and caverns have to be assessed. In the context of underground repositories for nuclear waste, clay shales represent a natural geological barrier and are considered as potential host rocks due to their extremely low permeability, the high sorption capacity and the potential for self-sealing. The relatively low strength and strength anisotropy, however, may influence the constructability of underground disposal tunnels at depth since damage (inelastic deformation) can be induced, which can negatively affect the natural barrier function of the host rock (Blümling et al. 2007). Due to their often low hydraulic conductivity and complete saturation in the natural state, the behavior of clay shales is characterized by a strong hydro-mechanical coupling. Due to the high advance rate (i.e. > 0.1-1 m/h) and the low hydraulic conductivity of the rock mass (i.e. < 10⁻⁷-10⁻⁶ m/s), an excavation in clay shales is essentially undrained (i.e. the mass of the pore fluid within the porous medium remains constant when a stress increment is applied) (Anagnostou and Kovári 1996). During the excavation, a so-called excavation damage zone (EDZ) may develop including shear and extensional fractures enhancing the permeability of the rock mass (Blümling et al. 2007, Tsang et al. 2008). The failure processes and extent of the EDZ is influenced by the orientation of the tunnel with respect to the orientation of the bedding planes and by the in-situ state of stress, the excavation size, the strength and stiffness of the material, the existence of natural fractures, and the excavation method (Bossart et al. 2002, Martin et al. 2004, Marschall et al. 2006, Marschall et al. 2008, Nussbaum et al. 2011, Yong et al. 2013, Thoeny 2014, Kupferschmied et al. 2015). Due to the hydro-mechanical coupling, induced stress changes and the associated mechanical behavior lead to a change in pore pressure within the rock mass during excavation and vice versa. Dilation leads to a reduction in pore pressure, which favors the stability of the rock mass around the underground construction. The dissipation of excess pore pressure in the longer term is associated with time-dependent deformation such as consolidation or mechanical swelling.

To understand the processes occurring during and after excavation and to predict the behavior of the rock mass around underground structures, it is important to determine the poroelastic and effective strength properties of the material which are representative for the in-situ conditions. Of particular interest are the relationship between stress-strain behavior and pore water pressure evolution under undrained conditions as well as the effective strength and stiffness. In this study, laboratory tests are considered to estimate the drained and undrained poroelastic and geomechanical properties of Opalinus Clay, a Mesozoic clay shale chosen as host rock for the disposal of high-level nuclear waste in Switzerland. Consolidated undrained (CU) and drained (CD) tests using a standard triaxial stress path are conducted. Particular interest lies in the dilatant behavior and its influence on pore pressure evolution during undrained tests, and in the effective strength properties. Opalinus Clay can be classified as a clay
shale, i.e. an over-consolidated clay with well-developed diagenetic bonds. Such materials have to be considered transitional between soil and rock (Bjerrum 1967, Botts 1986, Pei 2003). This requires the application of concepts from both soil and rock mechanics to examine the behavior of the material adequately.

5.2 Material description
Specimens used for this study were obtained from the shaly facies of Opalinus Clay from the Mont Terri Underground Rock Laboratory (URL). The main mineralogical composition of Opalinus Clay consists of clay minerals (30-80%), quartz (10-30%), carbonates (5-20%), and feldspar (0-5%). The clay minerals can be subdivided into illite-smectite mixed layers (5-20%), illite (15-30%), chlorite (5-20%), and kaolinite (15-35%) (Mazurek 1998, Thury and Bossart 1999, Nagra 2002, Klinkenberg et al. 2003, Bossart 2005). Opalinus Clay shows a pronounced bedding and its hydraulic and mechanical behavior can therefore be described as transversely isotropic. The hydraulic conductivity is $10^{-12}$ m/s parallel to bedding and $10^{-14}$ m/s normal to bedding (Marschall et al. 2004). The water loss porosity (calculated from weight loss at 105 °C and grain density) lies between 12 and 18% (Thury and Bossart 1999, Amann et al. 2011, Amann et al. 2012, Wild et al. 2015, Wild et al. (in preparation, see Chapter 4)).

5.3 Sampling and testing methods

5.3.1 Sampling, specimen characterization and preparation
The specimens were taken from two 67.5 mm diameter cores obtained from two 25 m long boreholes (BHM-1, BHM-2) drilled in the shaly facies at the Mont Terri URL. The boreholes were oriented parallel (BHM-1) and normal (BHM-2) to bedding. Triple-tube core drilling with compressed air cooling was utilized and the samples were recovered in plastic tubes and immediately wrapped in vacuum evacuated aluminum foil. Cylindrical specimens with a height-to-diameter ratio of 2:1 were prepared by dry cutting using a diamond band saw (Proxxon, Model MBS 240/E). Parallelism of the end faces was achieved by using a two trails system. The exposure time to the laboratory environment was minimized through a rigorous preparation procedure and by protecting the specimens with plastic foil between subsequent preparation steps (Wild et al. (in preparation, see Chapter 4)).

5.3.2 Triaxial testing procedure
CU and CD tests were performed at the DIPLAB Geomeccanica laboratory of the Politecnico di Torino, Italy, using different initial effective confinements. A high and medium pressure triaxial apparatus (HPTA, MPTA) were utilized (for details see Barla et al. 2010, Wild et al. (in preparation, see Chapter 4)).

In a first step, a flushing phase was used to remove air from the apparatus system and pore space by letting water permeating the specimen from bottom to top. The confining stress was large enough to
avoid specimen swelling. Deaired artificial pore water with a composition close to the formation water at the Mont Terri URL was utilized (Pearson 2002). The subsequent saturation phase is based on the principle that an increase in back pressure reduces the volume of remaining gas bubbles according to Boyle’s law and at the same time increases the amount of gas which is soluble in the water according to Henry’s law (Lowe and Johnson 1960). Back pressure was increased in several steps at both sides of the specimen. Before each increase, Skempton’s pore pressure coefficient $B$ (Skempton 1954) was determined to assure saturation. The specimens were considered saturated when $B$ was sufficiently high (i.e. higher than 0.9) and/or did not change significantly for two subsequent $B$-checks (i.e. $\Delta B$ in the order of $\pm$ 0.03).

After saturation, a consolidation phase was performed to ensure that the effective stress state within the specimen is uniform. The confinement ($\sigma_3$) and pore pressure ($u$) were increased over 24 hours to the desired value of isotropic effective consolidation stress (i.e. $\sigma'_c = (\sigma_3 - u) = 0.5, 0.75, 1.0, 2.0, 4.0, 6.0, 8.0, 12.0$, and $16.0$ MPa) and maintained until the specimens were consolidated (i.e. until the strain and change in water volume was constant). Differential loading was achieved by increasing the axial stress ($\sigma_1$) while the confining stress ($\sigma_3$) was kept constant. The axial stress was either applied parallel (P-specimens) or perpendicular to bedding (S-specimens) (Figure 5.1). The tests were axial strain controlled. Constant axial strain rates of $10^{-6}$ s$^{-1}$ for undrained tests on P-specimens and $10^{-7}$ s$^{-1}$ for S-specimens were utilized. For the drained tests on P-specimens, an axial strain rate in the order of $10^{-8}$ s$^{-1}$ was applied. Pore pressure was measured with the help of an external transducer close to the specimens’ top face (accuracy 8 kPa, Barla et al. 2010). The axial displacement (or strain) was determined externally by measuring the displacement of the sliding plate of the loading frame (accuracy 1 $\mu$m, Barla et al. 2010). Additionally, axial and radial strain were measured locally by a set of LVDTs which were mounted on the membrane at half of the specimen’s height (Barla et al. 2010, Wild et al. (in preparation, see Chapter 4)).

![Figure 5.1](image.png)

Figure 5.1: Different loading geometries used within this study: S-specimens for which the axial stress ($\sigma_1$) is applied normal to bedding (on the left) and P-specimens for which the axial stress is applied parallel to bedding (on the right).
5.3.3 Data evaluation methods

5.3.3.1 Onset of dilation
The methods used in this study to determine the onset of dilation are based on the approach by Brace et al. (1966). They stated that the onset of dilation can be identified as the point where the axial stress-volumetric strain curve deviates from linearity at low axial stresses. A non-linear stress-strain response suggests pre-failure yielding associated with the volumetric strain. Since a change in pore pressure is directly related to a change in volumetric strain (Detournay and Cheng 1993), the pore pressure-axial stress curve can be used to examine the onset of dilation (method 1). Additionally, the differential stress-effective confinement curve was considered (method 2). Also here, the onset of dilation is determined as the point where the curve deviates from linearity. As a third method (method 3), the strain curves (axial and radial) have been examined. Due to the anisotropy of the Opalinus Clay, the axial strain might be affected by shearing that occurs at the onset of dilation. This effect is manifested as a deviation from linearity.

5.3.3.2 Skempton’s pore pressure parameter $A$ and $B$
Skempton (1954) postulated that under undrained triaxial conditions the pore pressure change ($\Delta u$) can be related to changes in the principal stresses $\Delta \sigma_i$ and $\Delta \sigma_j$ by the following equation

$$\Delta u = B \left( \frac{1}{3} (\Delta \sigma_1 + 2 \Delta \sigma_3) + \frac{3A - 1}{3} (\Delta \sigma_1 - \Delta \sigma_3) \right),$$

where $A$ and $B$ are the Skempton’s parameters that can be determined experimentally. The induced pore pressure is, according to equation (5.1), related to a combination of a change in octahedral normal stress (associated with $B$) and a change in differential stress (associated with $A$). The $B$-value of a saturated specimen was taken from the last $B$-check of the saturation stage. For determining $A$ (or rather the product $AB$), the specimen is subjected to a standard triaxial stress path (i.e. $\Delta \sigma_1 > 0$, $\Delta \sigma_3 = 0$). Therefore, equation (5.1) reduces to

$$\Delta u = AB \Delta \sigma_1.$$  \hfill (5.2)

Measuring the change in pore pressure related to a change in axial stress during the early stage of shearing (i.e. at low differential stress, before yielding) reveals the product $AB$.

5.3.3.3 Young’s modulus and strength
The Young’s modulus was obtained from the linear portion of the axial stress-axial strain curve at low axial stresses (example shown in Figure 5.2). The results of the undrained tests (expressed in total stress) revealed the undrained modulus whereas the drained Young’s modulus was determined from drained tests (also expressed in total stress).
5.4 Results

5.4.1 Specimen characterization

The initial water content of the specimens lies between 6.19% and 7.51% and is therefore comparable with the water contents measured on cores right after core extraction (Pearson et al. 2003, Amann et al. 2011, Wild et al. (in preparation, see Chapter 4)). The geometry, test type, dimensions, water content, and porosity of the specimens are summarized in Table 5.1.

Table 5.1: Properties and test configurations of specimens. The initial water content of the specimens was determined with respect to the weight of the specimens after 24h drying at 105°C except for specimens ETH10_2 and ETH20_2-ETH24 which were dried to constant weight.

<table>
<thead>
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<th>height (mm)</th>
<th>initial water content (%)</th>
<th>porosity (%)</th>
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5.4.2 The influence of anisotropy, dilatancy, and confinement on pore pressure response

5.4.2.1 Poroelastic response

Skempton’s pore pressure parameters $B$ and $AB$ for P- and S-specimens are shown in Figure 5.3. The individual values are given in Table 5.2. No significant difference between P- and S-specimens with respect to $B$ can be identified (Figure 5.3a). However, there is a dependency of the $B$-value on the effective confinement ($\sigma_3'$). The $B$-value decreases from values of 0.90-0.97 to 0.76-0.80 for an increase in effective confinement of 0.6 MPa.

The poroelastic response to a change in differential stress also depends on the effective confinement but differs for P- and S-specimens (Figure 5.3b). For P-specimens, the product $AB$ is equal or smaller than $1/3B$ and decreases slightly with increasing confinement. For S-specimens, $AB$ is significantly higher than $1/3B$ and tends to increase with increasing effective confinement.

Figure 5.3: Values for the Skempton’s pore pressure parameters $B$ and $AB$ in dependency of the effective confinement: a) Skempton’s pore pressure parameter $B$ of P- and S-specimens obtained from the last $B$-check during the saturation phase (3 S-specimens and 4 P-specimens values from Wild and Amann (submitted) were added), b) Skempton’s pore pressure parameter $AB$ for P- and S-specimens tested during the elastic phase of shearing (i.e. before the onset of dilation).
Table 5.2: Skempton’s pore pressure parameters $B$ and $AB$ and the corresponding effective confinements for S- and P-specimens. No $AB$-values were measured for CD tests (*) or for specimens from Wild and Amann (submitted) that were sheared using a different non-standard triaxial stress path (**).

<table>
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<th>specimen no.</th>
<th>$\sigma_{3}'$ (MPa)</th>
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<th>$\sigma_{1}'$ (MPa)</th>
<th>$AB$</th>
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5.4.2.2 Elasto-plastic pore pressure response

For both geometries, the pore pressure increase during shearing is approximately linear before it deviates from linearity (Figure 5.4). For specimens consolidated at lower effective confinements (< 6 MPa), the pore pressure increase becomes smaller with each incremental differential stress increase, indicating pre-failure dilatant yielding. For those specimens, the maximum pore pressure is measured before the peak strength is reached. This observation is pronounced for S-specimens (Figure 5.4b). For tests conducted on P-specimens at higher effective consolidation stress (> 6 MPa), the pore pressure continues to increase linearly or even shows an augmented increase with each incremental differential stress increase, indicating compaction. For those specimens, the maximum pore pressure is reached either at peak strength or even slightly afterwards (Figure 5.4a).

The continuously suppressed dilation with increasing effective confinement is reflected by the increasing ratio of the pore pressure at peak strength ($u_p$) compared to the initial pore pressure ($u_0$) with increasing effective consolidation stress ($\sigma_{e}'$) (Figure 5.5, values in Table 5.3). A ratio $u_p/u_0$ close to 1 as observed for specimens consolidated at low effective stresses indicates that the pore pressure at peak is almost equal to the initial pore pressure. Hence, pore pressure builds up during shearing but as the
specimen dilates it reduces again in a way that no excess pore pressure is observed at peak strength. For specimens consolidated at higher effective consolidation stresses, the pore pressure increases until peak strength is reached and thus the ratio $u_p/u_0$ significantly exceeds unity. No significant difference between S- and P-specimens can be identified.

Figure 5.4: Pore pressure response for a) P-specimens and b) S-specimens. The light blue triangles indicate the highest pore pressure, the red points the peak strength, respectively.

Figure 5.5: Ratio of pore pressure at peak ($u_p$) and initial pore pressure ($u_0$) for P- and S-specimens.

The effect of the effective consolidation stress on the behavior of the specimens is also visible on the effective stress paths (Figure 5.6). The stress paths are plotted in terms of differential stress $q = (\sigma_1 - \sigma_3)$ and mean effective stress $p' = (\sigma_1' + 2\sigma_3')/3$. The effective stress paths for specimens consolidated at low effective stresses (i.e. overconsolidated specimens) turn to the right when the differential stress is
increased, indicating dilation and thus a reduction of pore pressure, which leads to an increase in mean effective stress. Those specimens continue to dilate after peak strength is reached. At effective consolidation stresses of 8 MPa and higher (i.e. for specimens that are closer to normal consolidation), the effective stress paths start to turn to the left. Thus, the specimens consolidated at higher effective stresses contract when approaching the peak strength and even continue to contract after peak strength is reached before they start to dilate in the post-peak region.

![Figure 5.6: Effective stress paths during the CU tests of P- and S-specimens.](image)

**Table 5.3:** Effective geomechanical properties for CU and CD tests on P- and S-specimens. \( \sigma_c' \): effective consolidation stress, \( \sigma_3 \): confinement, \( u_0 \): initial pore pressure, \( E \): Young’s modulus, \( \sigma_{1p} \): axial stress at peak strength, \( u_p \): pore pressure at peak strength, \( p' \): effective mean stress at peak strength, \( q \): differential stress at peak strength.

<table>
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<th>( \sigma_3 ) (MPa)</th>
<th>( u_0 ) (MPa)</th>
<th>( E ) (GPa)</th>
<th>( \sigma_{1p} ) (MPa)</th>
<th>( u_p ) (MPa)</th>
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5.4.3 The influence of anisotropy and effective consolidation stress on the stress-strain response

A comparison between the effective stress-strain curves for P- and S-specimens is shown in Figure 5.7. The curves have been normalized by dividing the effective axial stress \(\sigma_1'\) by the initial effective axial stress \(\sigma_1'_{0}\). For both geometries, the normalized effective stress at peak strength tends to decrease with higher effective consolidation stress. The decrease is higher for S-specimens than for P-specimens. The axial strain at peak increases with effective consolidation stress and is about two times higher for S-specimens than for P-specimens. The stress-strain curves for P-specimens consolidated at lower effective stress (< 6 MPa) are approximately linear up to peak strength (Figure 5.7a). A more distinct non-linear behavior towards the peak strength can be observed for specimens consolidated at higher effective stress (> 6 MPa). All specimens show post-peak strain-softening and approach an ultimate post-failure strength. Specimens consolidated at lower effective stresses show a larger post-peak stress drop compared to the specimens consolidated at higher effective stresses. Compared to P-specimens, the stress-strain curve for S-specimens deviates from linearity significantly before peak strength even at lower consolidation stresses (Figure 5.7b). Furthermore, the stress-strain curve rather reaches a plateau when it approaches the peak strength.

5.4.4 The influence of anisotropy, dilatancy and confinement on the geomechanical properties

5.4.4.1 Young’s moduli, onset of dilatancy

Figure 5.8 shows the undrained and drained Young’s moduli \(E\) from CU and CD tests respectively in dependency of the effective consolidation stress. The values are given in Table 5.3. \(E\)-values from CU...
tests on P-specimens increase with increasing effective consolidation stress. This is consistent with findings reported e.g. by Corkum and Martin (2007) for Opalinus Clay and Islam and Skalle (2013) for Pierre-I shale. The trend is less significant for the CU tests on S-specimens and no clear dependency on the confinement can be identified for the $E$-values from the CD tests on P-specimens. The difference in $E$ between P- and S-specimens tested under undrained conditions is about a factor of 2. Drained $E$-values of P-specimens lie within the range of S-specimens for the tested consolidation stresses.

![Figure 5.8: Dependency of drained and undrained Young’s moduli on the effective consolidation stress.](image)

The stress at the onset of dilation ($\sigma_{1,\text{dil}}$) is shown in Figure 9 in dependency of the effective consolidation stress. Plotted are the points that were determined with the different methods described in section 5.3.3.1. The individual values are given in Table 5.4. A linear dependency on the effective confinement can be observed. The stress at the onset of dilatancy increases with increasing effective consolidation stress. No significant difference between P- and S-specimens can be identified. For specimens with an effective consolidation stress exceeding 4 MPa, the specimens did not dilate but contract with increasing axial stress. Thus, $\sigma_{1,\text{dil}}$ cannot be determined.
Table 5.4: Individual values of the axial stress at the onset of dilation determined with the three different methods defined in section 5.3.3.1. \( \sigma_c' \): effective consolidation stress, \( \sigma_{1,dil} \): axial stress at the onset of dilation.

<table>
<thead>
<tr>
<th>specimens no.</th>
<th>( \sigma_c' ) (MPa)</th>
<th>( \sigma_{1,dil} ) (MPa)</th>
<th>( \sigma_{1,dil} ) (MPa)</th>
<th>( \sigma_{1,dil} ) (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ETH08</td>
<td>1.01</td>
<td>4.07</td>
<td>4.11</td>
<td>4.21</td>
</tr>
<tr>
<td>ETH09</td>
<td>1.99</td>
<td>5.46</td>
<td>5.50</td>
<td>5.84</td>
</tr>
<tr>
<td>ETH10_2</td>
<td>4.04</td>
<td>7.59</td>
<td>7.58</td>
<td>8.49</td>
</tr>
<tr>
<td>ETH16</td>
<td>0.50</td>
<td>1.93</td>
<td>1.92</td>
<td>2.24</td>
</tr>
<tr>
<td>ETH17</td>
<td>0.90</td>
<td>2.83</td>
<td>2.73</td>
<td>2.78</td>
</tr>
<tr>
<td>ETH19</td>
<td>2.02</td>
<td>5.30</td>
<td>5.20</td>
<td>-</td>
</tr>
</tbody>
</table>

5.4.4.2 Effective strength properties

The effective peak strength values of the CU and CD tests are plotted in Figure 5.10 in a \( q-p' \) plot. The individual peak strength values are given in Table 5.3. No significant difference between drained and undrained strength as well as between the different specimen geometries can be identified. The effective peak strength is clearly dependent on the effective mean stress and hence on the effective consolidation stress. Also shown in Figure 5.10 are the ultimate strength properties (i.e. the effective stress values at end of the tests). Also here, no significant difference in between the geometries and test conditions can be identified and an increase in ultimate differential stress with increasing effective mean stress is found.
5.5 Discussion

5.5.1 Poroelastic response

An inversely proportional dependency between Skempton’s pore pressure parameter $B$ and the effective confinement was observed. Such a decrease in $B$ with increasing effective confinement has been reported for different rock types such as sandstone, limestone, marble, granite (e.g. Mesri et al. 1976, Green and Wang 1986, Hart and Wang 1999, Lockner and Stanchits 2002), and shales (Hart and Wang 1999, Mesri et al. 1976, Cook 1999). Mesri et al. (1976) found a decrease in $B$-value from 0.9 at 3 MPa effective confinement to 0.7-0.8 at 5.5 MPa for different shales. A similar finding was reported by Cook (1999) for a Jurassic shale where the $B$-value decreased from 0.99 to 0.8 for an effective confinement increase from 3 to 7 MPa. The observed decrease in $B$-value with increasing effective confinement can be related to a decrease in drained compressibility. The $B$-value can be related to the following equation (Bishop 1966)

$$B = \frac{c_d - c_s}{c_d - c_s + n(c_f - c_s)}$$

where $c_d$ is the drained compressibility, $c_s$ the compressibility of the solid material, $c_f$ the compressibility of the fluid, and $n$ the porosity. An increase in effective confinement leads to the closure of naturally existing pores within the material. Hence the material becomes stiffer and the matrix (or drained)
compressibility increases. The compressibility of the fluid and the solid material remain essentially constant with increasing effective confinement (Bishop 1966, Lee et al. 1969, Mesri et al. 1976, Hart and Wang 1986, Zimmerman 1991). Assuming that the porosity does not decrease more than the drained compressibility, the $B$-value will decrease with decreasing drained compressibility according to equation (5.3).

The poroelastic response of the specimens to differential stresses was expressed by Skempton’s pore pressure parameter $A$. For an isotropic specimen, $A$ is 1/3 and pore pressure changes are solely related to changes in mean stress according to equation (5.1). A value of $A < 1/3$ indicates that the pore pressure built-up is smaller than for an isotropic specimen, which can be explained by differential stress induced dilation of the specimen. A value of $A > 1/3$ indicates that a higher pore pressure has been induced, which can be related to compaction of the specimen as a response to differential stresses (Wang 1997). By comparing $3AB$ to the $B$-value obtained for the corresponding effective confinement, the value of $A$ and the corresponding hydromechanical, elastic behavior of the specimens can be evaluated (i.e. if $3AB > B$, $A > 1/3$, if $3AB < B$, $A < 1/3$). It was seen that for P-specimens of Opalinus Clay, the $AB$-value is equal or smaller than 1/3$B$ and thus $A$ is smaller than 1/3. This indicates that shearing of P-specimens induces a smaller excess pore pressure than one would expect for an ideal isotropic elastic media. Furthermore, the $AB$ value decreases with increasing effective confinement. For S-specimens, $3AB$ is significantly higher than the $B$-value (i.e. $A > 1/3$), which represents a higher excess pore pressure build-up than expected for an ideal isotropic elastic media. The $AB$-value increases with increasing effective confinement.

The findings are consistent with results reported by Bellwald (1990), Aristorenas (1992), and Islam and Skalle (2013). Bellwald (1990) and Aristorenas (1992) tested Opalinus Clay specimens under undrained pure shear compression and extension. They showed that excess pore pressure develops even under these test configurations (i.e. when the mean octahedral normal stress remains constant). Positive excess pore pressures were observed for S-specimens tested under pure shear compression, negative excess pore pressures for S-specimens tested under pure shear extension. Islam and Skalle (2013) investigated the hydro-mechanical behavior of Pierre-I shale in standard triaxial compression tests with confinements between 10 and 30 MPa. Specimens loaded parallel to bedding showed a smaller pore pressure build-up than specimens loaded perpendicular to bedding.

The difference in pore pressure evolution for P- and S-specimens at low differential stresses can be related to the transverse isotropy of the material (Aristorenas 1992, Bobet et al. 1999, Einstein 2000, Islam and Skalle 2013). For a transversely isotropic material, the stiffness parallel to bedding is higher than the stiffness normal to bedding, which results in a higher change in volumetric strain for S-specimens than for P-specimens under standard triaxial compression. As the pore pressure built-up is directly related to volumetric strain, this will result in a higher excess pore pressure for S- than for P-specimens.
5.5.2 Poroplastic response

5.5.2.1 Undrained shear strength

For clays, the SHANSEP procedure (Stress History and Normalized Soil Engineering Properties) developed by Ladd and Foot (1974) is often used to characterize the undrained shear strength. According to this empirical relationship, the undrained shear strength \( s_u = q/2 \) obtained from an isotropically consolidated undrained triaxial compression test, normalized with respect to the effective consolidation stress is related to the overconsolidation ratio by a power law (Ladd et al. 1977)

\[
\frac{s_u}{\sigma_r^{0}} = a(OCR)^{b},
\]

where \( a \) and \( b \) are parameters, \( \sigma_r^{0} \) is the isotropic consolidation stress, and \( OCR \) is the overconsolidation ratio defined as \( OCR = \frac{\sigma_{cm}^{0}}{\sigma_r^{0}} \), where \( \sigma_{cm}^{0} \) is the maximum past effective isotropic stress. Gutierrez et al. (2008) demonstrated the applicability of SHANSEP for clay shales and therefore the applicability of the correlation even for materials showing apparent preconsolidation due to diagenetic bonding and cementation.

For overconsolidated specimens, an increase in normalized undrained shear strength with \( OCR \) is related to the augmented dilative behavior of the material. Dilation causes a reduction in excess pore pressure and leads to failure at effective stresses higher than the consolidation stress. For normally consolidated clays, the normalized shear strength is unique. The contractive behavior of normally consolidated specimens increases the excess pore pressure, which causes failure to occur at effective stresses lower than the consolidation stress. Such an increase in excess pore pressure at peak strength with increasing effective consolidation stress was observed in this study (Figure 5.5).

For Opalinus Clay at the Mont Terri URL, the maximum past isotropic effective stress is estimated to be 19 MPa, assuming that the overburden in the past was at least 1350 m (Mazurek et al. 2006) and a bulk density for the shaly facies of 2450 kg/m³ (Bossart 2005). The normalized shear strength is plotted against the \( OCR \) in a log-log diagram in Figure 5.11. The data are fitted with equation (5.4) and the resulting best fit is indicated. It can be seen that the trend observed for Opalinus Clay is consistent with the empirical relationship proposed by Ladd et al. (1977).
5.5.2.2 Effective strength properties

The peak strength values determined in this study were fitted with a linear Mohr-Coulomb failure envelope (Figure 5.12, black solid line) resulting in an effective cohesion of 1.0 MPa and an effective friction angle of 35.3° (Table 5.5). Both drained and undrained P- and S-specimens were used to obtain the best fit based on least squares regression analyses. Alternatively, a bi-linear Mohr-Coulomb failure envelope (blue dashed lines in Figure 5.12) revealed a higher friction angle of 47.1° for specimens consolidated at lower effective confinements (< 5-8 MPa, i.e. overconsolidated specimens) and a slightly lower friction angle of 31.9° for specimens consolidated at higher effective stresses (> 5-8 MPa). A Mohr-Coulomb fit of the ultimate strength values revealed a friction angle of 22.5° and a cohesion of 0.49 MPa. All linear regressions show a coefficient of determination (R²) of 0.93 or higher.

Table 5.5: Peak and ultimate strength Mohr-Coulomb failure envelopes for all tests combined. The fits were made considering all data points (linear) or by subdividing them into “low confinement” and “high confinement” (bi-linear).

<table>
<thead>
<tr>
<th></th>
<th>effective friction angle</th>
<th>effective cohesion</th>
</tr>
</thead>
<tbody>
<tr>
<td>peak strength, linear</td>
<td>35.3°</td>
<td>1.00 MPa</td>
</tr>
<tr>
<td>peak strength, bi-linear</td>
<td></td>
<td></td>
</tr>
<tr>
<td>low confinement (p' &lt; 5-8 MPa)</td>
<td>47.1°</td>
<td>0.20 MPa</td>
</tr>
<tr>
<td>high confinement (p' &gt; 5-8 MPa)</td>
<td>35.3°</td>
<td>2.01 MPa</td>
</tr>
<tr>
<td>ultimate strength, linear</td>
<td>22.5°</td>
<td>0.49 MPa</td>
</tr>
</tbody>
</table>
The observed decrease in friction angle with increasing effective consolidation stress can be correlated to the observed change in behavior of Opalinus Clay from brittle and dilative towards more ductile and contractive behavior for increasing consolidation stresses (i.e. towards normal consolidation), which is reflected in the stress-strain behavior, the pore pressure evolution, the effective stress paths, and the undrained shear strength. Considering the bi-linear failure envelope in Figure 5.12, a change in friction angle is observed at 5-8 MPa, i.e. at an effective consolidation stress where also the behavior changes.

Non-linear failure envelopes for clays and clay shales have been reported by many authors (e.g. Bishop et al. 1965, Graham and Li 1985, Nakken et al. 1989, Burland 1990, Wu 1991, Petley 1999, Amorosi and Rampello 2007, Deng et al. 2011). Bishop et al. (1965) conducted CU and CD tests on specimens of undisturbed London Clay and found a decrease of the slope of the failure envelope when passing from low to high effective confinements (i.e. between 0 and 10 MPa). In addition, they found a progressive change from brittle to ductile behavior with increasing effective confinement. Graham and Li (1985) reported a non-linear failure envelope for natural Winnipeg clay and related it to the micro- and macrostructure of the material. Petley (1999) reviewed undrained shear test results from different clays and clay shales and found a non-linear peak strength envelope with a steadily reducing slope (i.e. friction) with increasing mean effective consolidation stress. He proposed that the non-linearity of the envelope represents the initiation of the brittle-ductile transition. Nygård et al. (2006) demonstrated the transition from brittle to ductile behavior by analyzing the undrained shear behavior of shales and mudrocks from the North Sea and adjacent area. Normally consolidated or slightly overconsolidated specimens showed a ductile behavior while overconsolidated specimens with high overconsolidation...
ratios showed brittle behavior. A transition from brittle to ductile behavior with increasing confinement has also been reported by Hu et al. (2014) for a Callovo-Oxfordian clay shale from Meuse-Haute/Marne. However, both Nygård et al. (2006) and Hu et al. (2014) do not report non-linear failure envelopes. Due to its complex geological history including deposition, burial, and unloading, clay shales like Opalinus Clay show a distinct structure, i.e. a combination of particle arrangement (fabric) and interparticle forces (bonding) (Bjerrum 1967, Lambe and Whitman 1969). This structure significantly influences the hydro-mechanical behavior of the natural material (e.g. Bishop et al. 1965, Graham and Li 1985, Peters 1988, Leroueil and Vaughan 1990, Burland 1990, Rampello et al. 1993, Cotecchia and Chandler 1997, Petley 1999, Callisto and Rampello 2004, Amorosi and Rampello 2007, Favero et al. 2016). No diagenetic cements such as pyrite, siderite or calcite cement are found that fill the pore space and bonding is therefore mostly related to recrystallization of clay particles and adhesion from molecular bonds (Nagra 2002). At laboratory scale (mm-dm), the fabric of Opalinus Clay is determined by the preferred alignment of the clay platelets (Wenk et al. 2008, Favero et al. 2016).

The bi-linear nature of the failure envelope is similar to the failure envelope proposed by Patton (1966) for rock joints which accounts for the influence of undulating fracture surfaces (or asperities) on the shear resistance. Comparable to a joint, the undulating arrangement of clay platelets in Opalinus Clay form distinct bedding planes which act as zones of weakness. Due to this undulating fabric of the clay shale, shearing at low confinements is associated with dilatancy, which adds an additional frictional resistance. With increasing confinement, however, this dilation is suppressed, as demonstrated in this study. Shearing will lead to the destruction of possible undulations along the failure surfaces. Eventually, the failure plane goes entirely through the intact material and thus will result in a friction angle that is a material property.

5.6 Conclusions
A series of CU and CD tests on back-saturated specimens of Opalinus Clay was conducted to study the influence of confinement and anisotropy on the pore pressure response and effective geomechanical properties. The specimens were loaded parallel (P-specimens) and normal (S-specimens) to bedding. No significant difference in $B$-value with respect to the specimen’s geometry was found. However, a decrease in $B$-value from 0.90-0.97 to 0.76-0.80 with an increase in effective confinement from 0.03 to 0.63 MPa was observed, which could be related to a decrease in drained compressibility. At low differential stresses, P- and S-specimens behave differently: P-specimens build-up less pore pressure compared to a theoretical isotopic material whereas S-specimen show a higher pore pressure increase. This is reflected in an $A$-value smaller or higher than 1/3, respectively.

The determination of the undrained Young’s moduli for P-specimens revealed a dependency on the effective consolidation stress. $E$ increased from 0.3 to 3.5 GPa between 0.5 and 16 MPa effective consolidation stress. The undrained Young’s modulus for P-specimens was about twice the undrained...
Young’s modulus of S-specimens. Drained Young’s moduli from P-specimens were in the range of undrained S-specimens. An increase with increasing effective consolidation stress was also measured for the axial stress at the onset of dilatancy. For effective consolidation stresses exceeding 4 MPa, however, no dilation occurred anymore.

For the effective strength properties, no difference between P- and S-specimens and the test type (i.e. CU or CD tests) was recognized. A fit of a linear Mohr Coulomb failure envelope through all peak strength values revealed an effective friction angle of 35.3° and an effective cohesion of 1.0 MPa. However, a bi-linear or non-linear failure envelope is possible and can be related to a change in behavior of the Opalinus Clay when passing from an overconsolidated state towards a normally consolidated state. This change was reflected by a change from dilation to compaction and a change from brittle to more brittle-ductile behavior between 5 and 8 MPa effective consolidation stress, which can be related to the fabric of the material.

Acknowledgement
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5.7 References


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6. The influence of the stress path on the pore pressure evolution in a clay shale

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Abstract: Triaxial tests on back-saturated specimens using different controlled stress paths were conducted to study the influence of the stress path on the hydro-mechanical behavior of a clay shale. Opalinus Clay, a clay shale chosen for the disposal of high-level nuclear waste in Switzerland, was utilized. Three different stress paths approximating a tunnel excavation were applied: a two-dimensional stress path with isotropic initial stress conditions (i.e. pure shear compression), a two-dimensional stress path with anisotropic initial stress conditions, and a three-dimensional stress path. The influence of the stress path was investigated with respect to the specimens’ geometry. Two end members were tested: specimens where the axial load was applied parallel to bedding (P-specimens) and specimens where the load was applied normal to bedding (S-specimens). Differences in the hydro-mechanical response were identified with respect to the specimens’ geometry but also to the stress paths. When subjected to the two-dimensional stress paths, positive excess pore pressures were measured for S-specimens at low differential stresses whereas P-specimens showed negative excess pore pressure. The magnitudes of excess pore pressure were higher for the anisotropic two-dimensional stress path. Dilation associated with yielding was observed for all specimens. The application of the three-dimensional stress path revealed pore pressure responses that were conceptually comparable to the pore pressure response observed in situ. The often observed increase in pore pressure in front of the tunnel face can be related to poroelastic response of the rock mass to a change in differential and mean stress. Again, differences between P- and S-specimens were identified. Decrease in pore pressure before the tunnel face can be related to the dilation associated with yielding. No difference in peak strength between P- and S-specimens and the different stress paths was found. Nevertheless, it was shown that the state of failure is affected by the transversal isotropy as S-specimens fail at lower effective stresses compared to P-specimens.
Notation

\( p (p') \): mean stress, \( p = \frac{\sigma_{\text{ax}} + 2\sigma_{\text{conf}}}{3} \) (effective)

\( q \): differential stress, \( q = \sigma_{\text{ax}} - \sigma_{\text{conf}} \)

\( u (u_0) \): pore pressure (initial)

\( \theta \): angle

\( \nu \): Poisson’s ratio

\( \sigma_1 \): maximum principal stress

\( \sigma_2 \): intermediate principal stress

\( \sigma_3 \): minimum principal stress

\( \sigma_{\text{ax}} (\sigma_{\text{ax}}, \sigma_{\text{ax ini}}, \sigma_{\text{ax peak}}) \): axial stress (effective, initial, at peak strength)

\( \sigma_c' \): effective consolidation stress

\( \sigma_{\text{conf}} (\sigma_{\text{conf}}, \sigma_{\text{conf ini}}, \sigma_{\text{conf peak}}) \): confinement (effective, initial, at peak strength)

\( \sigma_l \): longitudinal stress (stress parallel to circular excavation)

\( \sigma_m \): octahedral mean stress

\( \sigma_r \): radial stress

\( \sigma_x \): horizontal in plane stress

\( \sigma_y \): horizontal out of plane stress

\( \sigma_z \): vertical stress

\( \sigma_\theta \): tangential stress

\( \tau \): octahedral shear stress
6.1 Introduction

Clay shales are considered as favorable host rocks for nuclear waste repositories due to their extremely low permeability, the high sorption capacity, and the potential for self-sealing. The geomechanical behavior of clay shales is strongly affected by a pronounced hydro-mechanical coupling. During tunnel excavation, undrained behavior dominates as a result of the high advance rate (> 0.1-1 m/h) and the low hydraulic conductivity (< $10^{-7}$-$10^{-6}$ m/s) (Anagnostou and Kovári 1996). The stress redistribution occurring around an excavation in the short term causes a change in pore pressure and vice versa.

Pore pressure changes measured during excavation of underground laboratory drifts in clay shales have been reported by different authors (e.g. Neerdael et al. 1991, Verstricht et al. 2003, Corkum and Martin 2007, Yong 2007, Wileveau and Bernier 2008, Martin et al. 2011, Armand et al. 2012, Morel et al. 2013, Nagra 2014, Giger et al. 2015). Although the measurements were conducted around excavations in different types of clay shales (Opalinus Clay, Callovo-Oxfordian clay stone, Boom Clay), under different initial stress conditions, and excavated with different excavation methods (drill and blast, road header, pneumatic hammer), similarities in the pore pressure evolution can be identified (Wild et al. 2015). The pore pressure evolution around an excavation measured at a point close and a point farther away from the tunnel circumference is conceptually shown in Figure 6.1. A gradual increase in pore pressure is often observed at a distance of up to about 7 tunnel diameter between the sensor location and the approaching tunnel face (Armand et al. 2012, Corkum and Martin 2007, Nagra 2014). The increase augments as the tunnel face comes closer to the monitoring location and a peak value is reached at about 0.4-2 diameter ahead of the advancing face (Nagra 2014). Depending on the location of the monitoring point, a pore pressure peak of up to about 1.6 times the initial pore pressure is reached (Wild et al. 2015). The peak value is smaller for sensors located farther away from the tunnel. As the face passes the sensor locations, pore pressure decreases. Some sensors close to the tunnel (within a range of around 1.3 times the tunnel radius) show a pore pressure drop to atmospheric pressure, indicating a connection to the tunnel atmosphere through open fractures (Wild et al. 2015). With a few exceptions where the pore pressure remains elevated after excavation (e.g. Masset 2006), most sensors located farther away from the tunnel (within 6 radii distance from the tunnel center, Nagra 2014) still show a drop in pore pressure but level off at values that lie below the initial pore pressure.

Stress redistribution that occurs during tunnel excavation causes deformation and thus pore pressure changes which can be considered as stress controlled. Hence, the stress path experienced by a point around the excavation influences the deformations and consequently also the pore pressure evolution and the hydro-mechanical behavior of the material surrounding the excavation (Lambe 1967). By applying an approximation of the stress conditions before, during, and after excavation in a laboratory test, the behavior of the material can be examined and conclusions about the processes occurring during tunnel excavation can be made. Such tests were done for example by Aristorenas (1992) who used pure

Figure 6.1: Conceptual pore pressure evolution around a tunnel measured at a point close to the tunnel and a point further away. The distance between the sensor and the tunnel face are given in diameter (d). The pore pressure ($u$) is given relative to the initial pore pressure before excavation ($u_0$).

This study examines the influence of different stress paths on the hydro-mechanical behavior of Opalinus Clay, a Mesozoic clay formation that was chosen as host rock for the disposal of high-level nuclear waste in Switzerland (BFE 2011). Cylindrical test specimens were back-saturated, consolidated and sheared under undrained conditions to failure in a triaxial cell using controlled stress paths. Stress paths that approximately reflect the stress evolution during tunnel excavation at a point in the crown and sidewall of a tunnel were used. Stress conditions representative for an excavation at the Mont Terri Underground Rock Laboratory (URL) were considered. Pore pressure was measured throughout all tests. The study focuses on the influence of the stress path on pore pressure evolution and strength. Furthermore, the results are discussed with respect to the pore pressure evolution monitored during tunnel excavation.

6.2 Approximation of stress paths for laboratory tests

6.2.1 General approach

The general problem that was chosen to be simulated in triaxial tests represents a horizontal tunnel excavated in a transversely isotropic ground with a horizontal bedding plane orientation (Figure 6.2a). The tunnel is aligned with the y-axis of the chosen right handed coordinate system. The x- and z-axis lie within the cross section of the tunnel. The in-situ principal stresses (i.e. $\sigma_1$, $\sigma_2$, and $\sigma_3$) are in line with the axis of the coordinate system whereby the maximum principal stress ($\sigma_1$) is the vertical stress, the intermediate principal stress ($\sigma_2$) is the horizontal stress perpendicular to the tunnel axis and the minimum principal stress ($\sigma_3$) is the out of plane stress (i.e. acts in direction of the excavation). A point at the tunnel roof and a point at the tunnel sidewall were considered for laboratory testing. Since the radial stress ($\sigma_r$) at the tunnel circumference trends towards zero during tunnel excavation, a point at the roof can be represented in a triaxial compression test, where the confinement is smaller than the axial...
load, by using a specimen where the bedding planes are oriented parallel to the core axis and the axial load is applied parallel to the bedding planes (P-specimen) (Figure 6.2b). A point at the sidewall can be simulated by utilizing a specimen for which the axial load is applied perpendicular to the bedding planes that are oriented normal to the core axis (S-specimen) (Figure 6.2b).

![Figure 6.2: a) Situation of the problem chosen for simulation. b) Cross section of the tunnel parallel to the x-z plane. A P-specimen (blue), for which the tangential stress ($\sigma_{\theta}$) > radial stress ($\sigma_r$) is applied parallel to bedding, is representing a point in the roof. A S-specimen (red), for which $\sigma_{\theta} > \sigma_r$ is applied normal to bedding, is representing a point at the sidewall.](image)

For the simulation of the stress evolution during excavation experienced by a rock sample around the tunnel, different approximations were applied. All of them consider an elastic behavior of the rock mass. Two stress paths (hereafter referred to as 2D-stress paths) that linearly interpolate the initial stress state with the stress state at the end of excavation (i.e. at around three diameter behind the face) and that consider an isotropic and an anisotropic in-situ stress state, respectively, are used. Furthermore, a stress path (hereafter referred to as 3D-stress path) which approximates the complete stress evolution during excavation is utilized. Note that, independent of the stress path used, all three principal stresses are considered. This creates a difficulty for the implementation of the stress paths into the laboratory test program because in the triaxial cell only the axial stress $\sigma_{ax}$ and the confinement $\sigma_{conf}$ can be controlled. Therefore, the actual three-dimensional situation at the tunnel circumference cannot be represented by 3 different principal stresses (i.e. $\sigma_1$, $\sigma_2$, $\sigma_3$) but has to be approximated using two stresses only (i.e. $\sigma_{ax}$ and $\sigma_{conf}$). The approach used in this study to convert the three-dimensional stress state into a two-dimensional stress condition is based on the octahedral mean and shear stress ($\sigma_m$ and $\tau$). For the three-dimensional stress state, $\sigma_m$ and $\tau$ are given by

$$
\sigma_m = \frac{\sigma_1 + \sigma_2 + \sigma_3}{3}, \quad (6.1)
$$

$$
\tau = \frac{1}{3} \left( (\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2 \right)^{1/2}. \quad (6.2)
$$

In the triaxial compression testing apparatus, $\sigma_2 = \sigma_3 = \sigma_{conf}$ and $\sigma_1 = \sigma_{ax}$. The octahedral mean and shear stress for the two-dimensional stress state are therefore given by
\[ \sigma_m = \frac{\sigma_{ax} + 2\sigma_{conf}}{3}, \]

\[ \tau = \frac{\sqrt{2}}{3}(\sigma_{ax} - \sigma_{conf}). \]

Combining equations (6.1) to (6.4), the confinement and axial stress for the triaxial test can be expressed by the octahedral mean and shear stress as follows

\[ \sigma_{ax} = \frac{2}{\sqrt{2}} \tau + \sigma_m, \]

\[ \sigma_{conf} = \sigma_m - \frac{1}{\sqrt{2}} \tau. \]

For each approximation used within this study, first the octahedral mean and shear stresses are calculated for the three-dimensional stress state according to equations (6.1) and (6.2). The related axial stress and the confinement that have to be applied in the laboratory are then calculated using equations (6.5) and (6.6). The stress paths chosen for triaxial testing are based on theoretical and numerical calculations (described below) and are considered to approximate a tunnel excavation in an in-situ state of stress representative for the Mont Terri URL.

### 6.2.2 Stress state at the Mont Terri URL

Martin and Lanyon (2003) compared results from three-dimensional numerical models, stress-induced borehole breakouts, under-coring, borehole slotter, and hydraulic fracturing tests and established an estimate of the in-situ stress tensor for the Mont Terri URL. The magnitude of the maximum principal stress (which is approaching the overburden stress) has been found to be consistent throughout the different methods (Martin and Lanyon 2003). However, stress measurements in a transversely isotropic material are challenging and especially the magnitude of the minimum principal stress is difficult to determine. Martin and Lanyon (2003), for example, proposed a minimum principal stress value which is smaller than the pore pressure (\(u\)) of about 2 MPa measured outside the excavation damage zone. A maximum value of 2.9 MPa for the minimum principal stress has been suggested by Evans et al. (1999) based on hydraulic fracturing tests and Corkum (2006) suggested a value between 2 and 3 MPa. Despite these uncertainties, it is assumed here that the magnitudes are reasonably well constrained. The principal stress state representing the in-situ condition at the Mont Terri URL used to derive the different stress paths for the laboratory tests is given in Table 6.1.

<table>
<thead>
<tr>
<th>in-situ stress state</th>
<th>magnitude (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>(\sigma_1)</td>
<td>6.5</td>
</tr>
<tr>
<td>(\sigma_2)</td>
<td>4.5</td>
</tr>
<tr>
<td>(\sigma_3)</td>
<td>2.5</td>
</tr>
<tr>
<td>(u)</td>
<td>2.0</td>
</tr>
</tbody>
</table>

Table 6.1: Stress magnitudes used within this study to represent the in-situ stress state at the level of the Mont Terri URL.
6.2.3 2D stress paths

Although the problem analyzed in this study considers a tunnel excavated in a transversely isotropic rock, isotropic elastic ground is assumed for simplicity reasons to derive the 2D-stress paths. For an infinite homogeneous, isotropic elastic medium, the stress distribution around a circular tunnel can be calculated using analytical solutions given by Hiramatsu and Oka (1962)\(^2\). Thus, for the problem considered in this study (see Figure 2a), the tangential stress \(\sigma_\theta\), radial stress \(\sigma_r\), and longitudinal stress \(\sigma_l\) (i.e. the stress along the tunnel axis) at the tunnel circumference (i.e. distance of point to tunnel center = radius of tunnel) after excavation result in

\[
\sigma_\theta = \sigma_1 + \sigma_2 - 2(\sigma_2 - \sigma_1) \cdot \cos 2\theta ,
\]

(6.7)

\[
\sigma_r = 0 ,
\]

(6.8)

\[
\sigma_l = \sigma_3 + 2\nu(\sigma_2 - \sigma_1) \cdot \cos 2\theta ,
\]

(6.9)

where \(\nu\) is the Poisson’s ratio (assumed to be 0.25) and \(\theta\) is the angle between a point at the tunnel circumference and the x-axis, measured counter clockwise (i.e. \(\theta = 0^\circ\) for a point at the sidewall and \(\theta = 90^\circ\) for a point at the tunnel roof).

Two cases are considered here. The stress path “2D-iso” linearly interpolates the initial three-dimensional stress state and the three-dimensional stress state after excavation assuming an isotropic in-situ stress state of 4.5 MPa which is equal to the mean in-situ stress at the Mont Terri URL (see Table 6.1). Since an isotropic in-situ stress state is considered, the tangential stress after excavation will take the same value all around the tunnel circumference and the resulting stress path is a pure shear compression stress path. The derived values for the axial stress and the confinement representing the state before and after excavation are given in Table 6.2 and the resulting stress path is illustrated in Figure 6.3a.

For the stress path “2D-aniso”, an anisotropic in-situ stress state (i.e. \(\sigma_1 = 6.5\) MPa, \(\sigma_2 = 4.5\) MPa, and \(\sigma_3 = 2.5\) MPa) that represents the stress state at Mont Terri URL more accurately was considered as starting point. Again, the initial three-dimensional stress state and the three-dimensional stress state after excavation are linearly interpolated. In this case, the stress path at the roof and the sidewall of the tunnel are different. The stress values before and after excavation for this case are also given in Table 6.2 and the two stress paths are plotted in Figure 6.3b.

\(^2\) These equations are often referred to as the Kirsch equations (Fjaer et al. 2008). Kirsch (1898), however, only gives the formulation for the stress distribution around a circular hole in an infinite plate in one-dimensional tension/compression. Nonetheless, these equations can easily be generalized to problems with anisotropic far-field stresses and arbitrary tunnel/borehole orientations (Fjaer et al. 2008). Such equations were published by e.g. Hiramatsu and Oka (1962, 1968) or Bradley (1979).
Table 6.2: Stress states considered for the two stress paths “2D-iso” and “2D-aniso”. Given are the principal stresses, the corresponding octahedral mean and shear stresses, and the derived axial and confining stresses applied in the laboratory tests (bold values) for the stress states before and after excavation.

<table>
<thead>
<tr>
<th>Stress path “2D-iso”</th>
<th>Stress path “2D-aniso”</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>roof and sidewall</td>
</tr>
<tr>
<td>Initial stress state</td>
<td></td>
</tr>
<tr>
<td>(\sigma_1) (MPa)</td>
<td>4.5</td>
</tr>
<tr>
<td>(\sigma_2) (MPa)</td>
<td>4.5</td>
</tr>
<tr>
<td>(\sigma_3) (MPa)</td>
<td>4.5</td>
</tr>
<tr>
<td>(\sigma_m) (MPa)</td>
<td>4.5</td>
</tr>
<tr>
<td>(\tau) (MPa)</td>
<td>0.0</td>
</tr>
<tr>
<td>(\sigma_{ax}) (MPa)</td>
<td>4.5</td>
</tr>
<tr>
<td>(\sigma_{conf}) (MPa)</td>
<td>4.5</td>
</tr>
<tr>
<td>Stress state after excavation</td>
<td></td>
</tr>
<tr>
<td>(\sigma_0 = \sigma_1) (MPa)</td>
<td>9.0</td>
</tr>
<tr>
<td>(\sigma_1 = \sigma_2) (MPa)</td>
<td>4.5</td>
</tr>
<tr>
<td>(\sigma_3 = \sigma_3) (MPa)</td>
<td>0.0</td>
</tr>
<tr>
<td>(\sigma_m) (MPa)</td>
<td>4.5</td>
</tr>
<tr>
<td>(\tau) (MPa)</td>
<td>3.7</td>
</tr>
<tr>
<td>(\sigma_{ax}) (MPa)</td>
<td>9.7</td>
</tr>
<tr>
<td>(\sigma_{conf}) (MPa)</td>
<td>1.9</td>
</tr>
</tbody>
</table>

Figure 6.3: 2D stress paths used in this study: a) stress path “2D-iso” resulting from an isotropic initial stress state (applied for P- and S-specimen), b) stress paths “2D-aniso” representing the stress paths at a point at the roof (applied to P-specimen) or the sidewall (applied to S-specimen) for an anisotropic in-situ stress state.

6.2.4 3D stress path

A three-dimensional hydro-mechanically coupled numerical model was conducted using the commercially available three-dimensional continuum code FLAC3D (Fast Lagrangian Analysis of Continua in 3 Dimensions, Itasca 2009) to derive the elastic stress path “3D-iso” occurring during the excavation phase. The model geometry consists of a circular tunnel with a diameter of 4 m (Figure 6.4). The model has a length of 26 m in y-direction and extends to 24 m in both x- and z-direction. A full-
face excavation sequence with a round length of 0.2 m was simulated. No support measures were used. An isotropic initial stress state of $\sigma_1 = \sigma_2 = \sigma_3 = 4.5 \text{ MPa}$ was applied. The initial pore pressure was set to 2 MPa. The principal total stresses were monitored after each excavation round at a point in the roof and sidewall (0.25 m away from the tunnel circumference), in a plane normal to the tunnel axis in the center of the tunnel (i.e. at $y = 10$ m). A linear elastic model was used to characterize the rock mass response. The bulk and shear modulus were 0.4 GPa and 1.2 GPa, respectively, and a density of 2.45 g/cm$^3$ was utilized. A fully-coupled approach was used representing undrained behavior. A fluid bulk modulus of 2 GPa and a porosity of 18% were applied.

The resulting total stress paths at the roof and sidewall points are shown in Figure 6.5. For the actual stress path “3D-iso” used in the laboratory tests, the stress path was simplified. The approximation is also shown in Figure 6.5. The individual stress values at the vertices of the stress path (A-F, labeled in Figure 6.5) are given in Table 3 for both the $\sigma_m-\tau$- and the $\sigma_{conf}-\sigma_{ax}$-space. Furthermore, Figure 6.5 shows points along the stress path which indicate the position of the face with respect to the monitoring point. Negative values indicate that the face ahead of the monitoring location, for positive values, the face passed the monitoring point. The distance between the face and the monitoring point is expressed with respect to the tunnel diameter (d). The approximate position of the vertices of the stress path (A-F) with respect to the face is given in Table 6.3.
Figure 6.5: 3D-stress path for a point in the roof and sidewall (each 0.25m away from the tunnel circumference) derived from the conceptual 3D-model. Also shown is the approximation of those stress paths used in the laboratory tests. The points indicate the distance of the monitoring point to the advancing face (expressed with respect to the tunnel diameter (d), negative = in front of the face, positive = behind the face).

Table 6.3: Individual stress values used along the stress path “3D-iso” and approximate distance of corner points to face given with respect to the tunnel diameter (negative = in front of the face, positive = behind the face).

<table>
<thead>
<tr>
<th>vertex</th>
<th>$\sigma_m$ (MPa)</th>
<th>$\tau$ (MPa)</th>
<th>$\sigma_{\text{conf}}$ (MPa)</th>
<th>$\sigma_{\text{ax}}$ (MPa)</th>
<th>approx. distance to face (diameter)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>4.5</td>
<td>0.0</td>
<td>4.5</td>
<td>4.5</td>
<td>-2.5</td>
</tr>
<tr>
<td>B</td>
<td>4.5</td>
<td>0.9</td>
<td>3.9</td>
<td>5.8</td>
<td>-0.5 to -0.25</td>
</tr>
<tr>
<td>C</td>
<td>6.0</td>
<td>0.9</td>
<td>5.4</td>
<td>7.3</td>
<td>-0.1</td>
</tr>
<tr>
<td>D</td>
<td>6.0</td>
<td>2.5</td>
<td>4.2</td>
<td>9.5</td>
<td>0.05</td>
</tr>
<tr>
<td>E</td>
<td>4.0</td>
<td>2.5</td>
<td>2.2</td>
<td>7.5</td>
<td>0.5</td>
</tr>
<tr>
<td>F</td>
<td>4.5</td>
<td>3.0</td>
<td>2.4</td>
<td>8.7</td>
<td>2.5</td>
</tr>
</tbody>
</table>

6.3 Material description, sampling and testing methods

Specimens from the shaly facies of Opalinus Clay at the Mont Terri URL were used in this study. Opalinus Clay is an overconsolidated clay shale with an age of about 180 million years. It mainly consists of clay minerals (50-80%), quartz (10-20%), carbonates (5-20%), and feldspar (0-5%). The clay minerals can be divided into illite-smectite mixed layers (10-15%), illite (15-25%), chlorite (5-15%), and kaolinite (20-30%) (Mazurek 1998, Nagra 2002, Klinkenberg et al. 2003, Bossart 2005). A pronounced bedding related to the alignment of the clay platelets can be identified causing a transversely isotropic hydraulic and mechanical behavior. The hydraulic conductivity is $10^{-12}$ m/s parallel to bedding and $10^{-14}$ m/s normal to bedding (Marschall et al. 2004). The water loss porosity (calculated from weight loss at 105°C and grain density) lies between 15 and 19% (Bossart 2005, Amann et al. 2011, Amann et al. 2012, Wild et al. 2015, Wild and Amann submitted, Wild et al. (in preparation, see Chapter 4)).
The specimens were obtained from two 25 m long cores (BHM-1, BHM-2). A triple-tube core barrel with compressed air cooling was used. The samples were recovered in plastic tubes and sealed in vacuum evacuated aluminum foil. Three cylindrically P-specimens (from BHM-1) and three S-specimens (from BHM-2) with parallel end faces perpendicular to the core axis were prepared by dry cutting using a diamond band saw (Proxxon, Model MBS 240/E). A height-to-diameter ratio of 2:1 was used.

The tests were performed at the DIPLAB Geomeccanica laboratory of the Politecnico di Torino, Italy. A high and medium pressure triaxial apparatus (HPTA, MPTA) were used (for details see Barla et al. 2010, Wild et al. (in preparation, see Chapter 4)). The specimens were saturated, consolidated and subsequently sheared under undrained conditions using the different stress paths described in the previous chapters. For each stress path (i.e. “2D-iso”, “2D-aniso”, and “3D-iso”) a P- and a S-specimen was tested, respectively. The specimens were sheared under stress controlled conditions until failure occurred. Due to the difference in hydraulic conductivity between P- and S-specimens, the required time to failure for undrained triaxial loading conditions is substantially smaller for P-specimens than for S-specimens (Wild et al. (in preparation, see Chapter 4)). The individual stress rates were selected so that the failure was expected to occur within 3 hours for P-specimens and 30 hours for S-specimens. During the tests, pore pressure was measured by an external transducer close to the specimens’ top (accuracy 8 kPa, Barla et al. 2010).

6.4 Results

6.4.1 Specimens saturation and consolidation

Figure 6.6a shows the individual values of Skempton’s pore pressure parameter $B$ (Skempton 1954) obtained for the specimens during each step of the saturation phase. A specimen is considered to be saturated if $B$ is substantially high and/or does not change more than $\pm$ 0.03 for two subsequent $B$-checks (Wild et al. (in preparation, see Chapter 4).

Except for specimens ETH01 and ETH02, all specimens show a change in $B$-value for the last two $B$-checks which is equal or smaller than 0.03. The change in $B$ for specimen ETH02 is 0.04. Furthermore, its $B$-value is about the same as for ETH15 and ETH04 which were tested under the same effective confinement (about 0.2 MPa). ETH01 shows a slightly different behavior compared to the others as its $B$-value decreases by a value of 0.02 from $B$-check 6 to 7 before it increases again by a value of 0.1. The final $B$-value is in the range of the other S-specimens (ETH03, ETH05).

Figure 6.6b shows the theoretical change in back pressure needed to saturate a specimen depending on its initial state of saturation according to Lowe and Johnson (1960). The line for 100% saturation is plotted together with the back pressures actually used to saturate the specimens with respect to their initial saturation. The initial saturation was estimated according to the ISRM suggested methods (ISRM 1979) from the initial water content of the specimens. The values are given in Table 6.4. A grain density of $2.73 \pm 0.03$ g/cm$^3$ was used to calculate the porosity (Pearson et al. 2003, Bossart 2005, own data).
The uncertainty in the grain density results in a range of saturation indicated by the error bars in Figure 6.6b. It can be seen that all specimens lie significantly above the theoretical pressure needed to reach full saturation. Together with the conclusions above, it is therefore concluded that all specimens were saturated.

Table 6.4: Properties of the specimens before testing, after saturation, and after consolidation. The initial water content of the specimens was determined with respect to the weight of the specimens after 24h drying at 105°C. The water content after saturation/consolidation was estimated from the weight of the specimen after saturation/consolidation phase determined by adding/subtracting the measured amount of water flowing into/out of the specimen during these phases.

<table>
<thead>
<tr>
<th>test specimen code</th>
<th>specimen orientation</th>
<th>diameter (mm)</th>
<th>height (mm)</th>
<th>water content before test (%)</th>
<th>dry density (g/cm³)</th>
<th>porosity (%)</th>
<th>saturation before test (%)</th>
<th>water content after saturation (%)</th>
<th>water content after consolidation (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ETH01</td>
<td>S</td>
<td>67.89</td>
<td>136.15</td>
<td>6.44</td>
<td>2.27</td>
<td>16.8</td>
<td>87.0</td>
<td>7.38</td>
<td>6.95</td>
</tr>
<tr>
<td>ETH02</td>
<td>P</td>
<td>67.67</td>
<td>134.77</td>
<td>7.39</td>
<td>2.26</td>
<td>17.4</td>
<td>96.1</td>
<td>7.66</td>
<td>7.29</td>
</tr>
<tr>
<td>ETH03</td>
<td>S</td>
<td>67.67</td>
<td>134.41</td>
<td>6.21</td>
<td>2.27</td>
<td>16.7</td>
<td>84.4</td>
<td>6.71</td>
<td>6.68</td>
</tr>
<tr>
<td>ETH04</td>
<td>P</td>
<td>67.26</td>
<td>134.52</td>
<td>6.07</td>
<td>2.28</td>
<td>16.4</td>
<td>84.7</td>
<td>6.79</td>
<td>5.81</td>
</tr>
<tr>
<td>ETH05</td>
<td>S</td>
<td>67.71</td>
<td>134.39</td>
<td>7.46</td>
<td>2.26</td>
<td>17.4</td>
<td>96.5</td>
<td>8.28</td>
<td>7.88</td>
</tr>
<tr>
<td>ETH15</td>
<td>P</td>
<td>67.51</td>
<td>135.30</td>
<td>7.09</td>
<td>2.27</td>
<td>17.0</td>
<td>94.3</td>
<td>7.79</td>
<td>6.87</td>
</tr>
</tbody>
</table>

Figure 6.7 shows the change in back volume flowing out of the specimen during the consolidation stage. All specimens trend to a constant value after a certain time, indicating that they are sufficiently consolidated to establish a uniform effective stress state.
6.4.2 Stress paths and pore pressure evolution

6.4.2.1 2D stress paths

Figure 6.8 shows the total stress paths (TSP) and effective stress paths (ESP) for a P- and S-specimen subjected to the “2D-iso” stress path. All stress paths are shown in a $p'$-$q'$- or $p$-$q$- space, where $p = (\sigma_{ax} + 2\sigma_{conf})/3$, $p' = (\sigma_{ax'} + 2\sigma_{conf'})/3$, and $q = \sigma_{ax} - \sigma_{conf}$. The total stress path was corrected by the initial pore pressure $u_0$. For the P-specimen, a linear increase in pore pressure of 0.05 MPa was observed between 0 and 0.8 MPa differential stress (Figure 6.8). For differential stresses in excess of 0.8 MPa, the specimen starts to dilate and the pore pressure decreases up to failure. The pore pressure at failure (0.7 MPa) lies significantly below the initial pore pressure (2.1 MPa).

For the S-specimen, the effective stress path deviates from the total stress path and is linear at low differential stresses. At about 1.8 MPa differential stress, the stress path deviates from linearity, indicating less pore pressure build-up (i.e. the specimen dilates), even though pore pressure is built up until about 3.5 MPa differential stress. For differential stresses larger than 3.5 MPa, the pore pressure decreases. At failure, the excess pore pressure is approximately zero. Also shown in Figure 6.8 is the isotropic 2D total stress path (TSP tunnel) that represents a tunnel excavation considering plane strain conditions. It can be seen that the S-specimen fails before the end of the stress path is reached. For the P-specimen, however, failure occurs at a higher stress level.

Figure 6.7: Change in back volume during the consolidation stage.
Figure 6.8: Total and effective stress paths for P- and S-specimen subjected to the “2D-iso” stress path.

Figure 6.9 shows the theoretical total stress path and the actual tested total and effective stress paths that approximate a tunnel excavation for an anisotropic initial stress state. The P-specimen (Figure 6.9a) shows a deviation from the total stress path to the right hand side indicating a decrease in pore pressure. The pore pressure at peak is 0.6 MPa, i.e. 1.4 MPa below the initial pore pressure of 2.0 MPa. As for the “2D-iso” stress path, the effective stress path for the S-specimens deviates from the total stress path to the left hand side (i.e. pore pressure is build-up within the specimen) (Figure 6.9b). At 4.5 MPa differential stress, the pore pressure reaches its peak value before it decreases until the specimen fails. The pore pressure at peak lies 1.1 MPa below the initial pore pressure.

Figure 6.9: Total and effective stress paths for P-specimen (a) and S-specimen (b) subjected to the “2D-aniso” stress path.
6.4.2.2 3D stress path

The response to the „3D-iso“ stress path for the P- and S-specimen is shown in Figure 6.10. Figure 6.10a shows the pore pressure ($u$) and the applied differential stress ($q$) vs. time for the P-specimen. The curves are subdivided into 6 segments according to the vertices of the total stress path shown in Figure 6.10b and Figure 6.5. Additionally, the point where the tunnel face is assumed to pass the pore pressure monitoring point in the tunnel cross-section (derived from the three-dimensional numerical model) is indicated by a black point on each curve. The curves for the S-specimen are shown in Figure 6.10c and Figure 6.10d, respectively. For both specimen geometries, a similar pore pressure response is observed. There is an increase in pore pressure up to segment C-D (i.e. ahead of the tunnel face) where the pore pressure reaches a peak value before it decreases.

Differences between P- and S-specimens can be observed in the first segment (A-B) where no change in mean stress occurred (i.e. pure shear compression). The pore pressure for the P-specimen remains approximately constant whereas the pore pressure within the S-specimen increases by 0.6 MPa. In segment B-C, the differential stress was kept constant and the mean stress was increased causing a pore pressure increase of 0.8 MPa for the P-specimen and 1.2 MPa for the S-specimen. From C to D, a pure shear compression stress path (i.e. differential stress increase with constant mean stress) was applied. Pore pressure for a P-specimen slightly increased at the beginning of the C-D segment before it decreased. A peak pore pressure of 3.1 MPa was reached (i.e. 1.5 times the initial pore pressure). In the same stress path segment (i.e. C-D), the pore pressure increase for the S-specimen exceeded the pore pressure increase for the P-specimen significantly. For the S-specimen, the peak value in pore pressure was 4.4 MPa (which is 2.1 times the initial value). The stress path between D and E is an unloading stress path where the differential stress is kept constant and the mean stress is decreased. For both specimens, the pore pressure decreased in this section (by 1.5 MPa and 2.1 MPa for the P- and S-specimen, respectively). Afterwards (E-F) the specimen is sheared by increasing both the mean and differential stress. The pore pressure for the P-specimen increased by 0.2 MPa in this section until failure was reached. The pore pressure for the S-specimen stays constant until it decreases abruptly by 0.7 MPa associated with failure.
Figure 6.10: Stress and pore pressure response for P- and S-specimen subjected to stress path “3D-iso”: a) differential stress applied over time and related pore pressure response for the P-specimen, b) total stress path for the P-specimen, c) differential stress applied over time and related pore pressure response for the S-specimen, d) total stress path for the S-specimen. The black dots approximately represent the point where the tunnel face would pass the monitoring point.

6.4.3 Strength

The peak strength values for the individual specimens are given in Table 6.5. The differential stress at peak is plotted with respect to the mean effective stress in Figure 6.11. In addition to the three different stress paths used in this study, the results from Wild and Amann (submitted) for P- and S-specimen that were consolidated to different effective confinements and subsequently subjected to a standard triaxial stress path under undrained conditions (CU-STSP) are plotted. Furthermore, test results from consolidated undrained pure shear compression (CU-PSC) or extension tests (CU-PSE) from Aristorenas (1992) are shown.
No significant differences in strength between the specimen’s geometry and the different stress paths can be identified. This is consistent with findings reported by Law (1981) who subjected specimens of stiff Champlain Sea clay to different triaxial and plane strain stress paths and found that their failure envelope is independent on the path. Similar observations have been made by Ng and Lo (1985).

Table 6.5: Initial stress conditions and peak strength values for the tested specimens.

<table>
<thead>
<tr>
<th>test specimen code</th>
<th>specimen orientation</th>
<th>stress path</th>
<th>$\sigma_{\text{conf,ini}}$ (MPa)</th>
<th>$\sigma_{\text{ax,ini}}$ (MPa)</th>
<th>$u_0$ (MPa)</th>
<th>$\sigma_{\text{conf,peak}}$ (MPa)</th>
<th>$\sigma_{\text{ax,peak}}$ (MPa)</th>
<th>$u_{\text{peak}}$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ETH15</td>
<td>P</td>
<td>2D-iso</td>
<td>4.59</td>
<td>4.60</td>
<td>2.86</td>
<td>1.67</td>
<td>10.62</td>
<td>0.74</td>
</tr>
<tr>
<td>ETH01</td>
<td>S</td>
<td>2D-iso</td>
<td>4.58</td>
<td>4.64</td>
<td>2.07</td>
<td>3.00</td>
<td>7.94</td>
<td>2.14</td>
</tr>
<tr>
<td>ETH02</td>
<td>P</td>
<td>2D-aniso</td>
<td>3.44</td>
<td>7.31</td>
<td>2.06</td>
<td>1.43</td>
<td>7.78</td>
<td>0.63</td>
</tr>
<tr>
<td>ETH03</td>
<td>S</td>
<td>2D-aniso</td>
<td>3.44</td>
<td>6.95</td>
<td>2.07</td>
<td>3.07</td>
<td>8.61</td>
<td>0.99</td>
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<td>ETH04</td>
<td>P</td>
<td>3D-iso</td>
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<td>4.82</td>
<td>2.12</td>
<td>2.80</td>
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<td>1.66</td>
</tr>
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<td>3D-iso</td>
<td>4.56</td>
<td>4.70</td>
<td>2.10</td>
<td>2.50</td>
<td>9.23</td>
<td>1.28</td>
</tr>
</tbody>
</table>

Figure 6.11: Differential stress at peak vs. effective mean stress for the different stress paths. Also plotted are results from Wild and Amann (submitted) for P- and S-specimens tested under consolidated undrained conditions with a standard triaxial stress path (CU-STSP) and from Aristorenas (1992) for S-specimens subjected to consolidated undrained pure shear compression (CU-PSC) and extension (CU-PSE) stress paths.

6.5 Discussion

The P- and S-specimen subjected to the “2D-iso” stress path show a different hydro-mechanical response. Whereas the P-specimen only builds up a small positive excess pore pressure at low
differential stresses and starts to dilate relatively early, which is reflected by a decrease in pore pressure, the S-specimen contracts significantly before it starts to dilate. These findings are consistent with the findings made by Bellwald (1990) and Aristorenas (1992) who subjected Opalinus Clay specimens to pure shear compression and extension stress paths under undrained conditions.

The pore pressure is related to the volumetric strain that can, for a linear-elastic material in a triaxial cell, be expressed by: \( \Delta \varepsilon_{\text{vol}} = \Delta \varepsilon_{\text{ax}} + 2\Delta \varepsilon_{\text{rad}} \). An isotropic specimen subjected to a pure shear compression stress path (i.e. \( \Delta \sigma_{\text{ax}} = -2\Delta \sigma_{\text{conf}} \)) would theoretically not show any change in pore pressure in the elastic regime (since \( \Delta \varepsilon_{\text{vol}} = 1/E\Delta \sigma_{\text{ax}} + 2*1/E\Delta \sigma_{\text{conf}} = 1/E\Delta \sigma_{\text{ax}} - 1/E\Delta \sigma_{\text{ax}} = 0 \)). For a transversely isotropic specimen, however, the stiffness parallel to bedding exceeds the stiffness perpendicular to bedding, and the change in volumetric strain is not zero. For a S-specimen, this will result in a positive change in volumetric strain and thus the pore pressure will increase under pure shear compression as the specimen contracts. For a P-specimen the theoretical volumetric change under pure shear compression is negative, leading to a decrease in pore pressure. 3

For both specimen geometries subjected to the 2D-stress paths, the pore pressure increase at low shear stress is approximately linear. At differential stresses between 1-2 MPa, the response deviates from linearity indicating plastic yielding of the specimen. Aristorenas (1992) related this non-linearity to the growth of micro cracks/fissures. As these cracks evolve progressively, this mechanics results in an overall non-linear expansion. For a S-specimen, the dilation associated with yielding has to counteract the compressive behavior resulting from the transversal isotropy. Therefore, the behavior changes gradually from compaction to expansion. For a P-specimen, the dilation enhances the already dilatant behavior.

The “3D-iso” stress path applied to P- and S-specimens in this study revealed a pore pressure response that is conceptually comparable to the pore pressure response observed in situ. The stress path applied can be subdivided into parts that can be represented by the “2D-iso” stress path, an increase/decrease in mean effective stress, and a standard triaxial stress path. Since the pore pressure increase observed in the laboratory for the “3D-iso” stress path is (at least in the beginning) linear, the in-situ increase in pore pressure in front of the face can be most probably related to a poroelastic response of the rock mass to a change in differential and mean stress. A difference in the amount of pore pressure increase is related to the transverse isotropy as discussed for the “2D-iso” stress path (S-specimens show a higher pore pressure increase than P-specimens). This is consistent with findings of Vaunat et al. (2013). They modeled the pore pressure evolution during an excavation using isotropic and anisotropic (with respect to hydraulic parameters, stiffness, or stress state) numerical models and compared the results to pore pressure measurements in Boom Clay. They concluded that the peak in front of the face is controlled almost exclusively by the anisotropy in the elastic moduli.

3 For a more detailed derivation of the change in volumetric strain of a transversal isotropic media subjected to 2D-stress paths see Appendix D.
The initial phase of the “3D-iso” stress path (i.e. segment A-B) is followed by an increase in octahedral mean stress (segment B-C) causing a pore pressure increase in both P- and S-specimen. The rate of increase in segment B-C is higher than for the differential stress increase (segment A-B). This again fits with the field observations where the increase in pore pressure augments as the face approaches the sensor location.

In the laboratory tests in this study using the stress path “3D-iso”, a pore pressure peak was reached before the tunnel face is assumed to pass the sensor location. Also this is comparable with the in-situ observations and can be related to dilatancy accompanying yielding before peak strength is reached. The pore pressure continues to decrease as the mean stress is reduced after the face passed the sensor location (segment D-E).

Although there was no difference in peak strength between S- and P-specimens identified, there is a significant difference between the two geometries with respect to the occurrence of failure. For the “2D-iso” and “2D-aniso” stress paths, the S-specimen failed significantly before the end of the theoretical tunnel stress path was reached whereas the P-specimens failed after the end of the theoretical path. Also for the “3D-iso” stress path, failure of the S-specimen occurred before the P-specimen. Thus, considering a tunnel excavated in a transversely isotropic medium, where the bedding planes are oriented horizontally and which is characterized by an isotropic stress state, the stability is affected by the anisotropy of the medium although the strength is generally assumed to be independent of the principal direction (i.e. the effective failure envelope is equal for P-specimen and S-specimen). Despite the occurrence of dilation associated with yielding, which positively affects the stability, the built-up of excess pore pressure at low differential stresses differs for P- and S-specimens and thus leads to different behavior with respect to failure. At the roof (P-specimen), the effective stress path is more favorably influenced with regard to failure by the anisotropy whereas at the sidewall the anisotropy influences the effective stress path less favorably and failure occurs earlier. This is conceptually illustrated in Figure 6.12 for the example of the “2D-iso” stress path. Compared to the theoretical effective stress path expected for an isotropic specimen, the developed positive excess pore pressures in the S-specimens affect the effective stress path less favorable and failure is reached at a lower stress level than expected. For P-specimens, the negative excess pore pressure is positively affecting the effective stress path leading to a failure at higher stresses. Hence, the stability of the rock mass surrounding the tunnel is stress-path dependent as the total stress path influences the pore pressure evolution (i.e. dependent on the orientation of the applied load with respect to the plane of isotropy), which in turn affects the effective stress path.
Figure 6.12: Conceptual illustration of the effect of transversal isotropy on the stability of the rock mass surrounding a tunnel. Due to the difference in pore pressure evolution for P- and S-specimens, the latter fails at an effective stress level which is smaller than the failure strength expected for a theoretical tunnel stress path. A P-specimen, however, fails at higher effective stress.

6.6 Conclusion

Specimens of Opalinus Clay were subjected to different stress paths, theoretically describing the stress evolution occurring at a point in the crown and sidewall during an excavation of a circular tunnel. The influence of the stress paths on the hydro-mechanical behavior was investigated. For the 2D stress paths, positive excess pore pressure for S-specimens and negative excess pore pressure for P-specimens were found. After an elastic response, all specimens showed a non-linearity in pore pressure evolution related to the dilation of the specimens. The “3D-iso” stress path which is a combination of the “2D-iso” stress path, an increase/decrease of octahedral mean stress and a standard triaxial stress path, was able to reproduce the observe pore pressure in situ conceptually. An increase in pore pressure as the tunnel face approaches the monitoring sensor location can be related to a poroelastic response of the specimens. A pore pressure peak was reached before peak strength associated with the dilation of the specimen and a drop in pore pressure as observed in situ is therefore not solely related to failure. A drastic drop in pore pressure was not found in the segments of the stress path where the face is expected to pass the sensor location. Failure and the associated stress drop occurred later.

The experiments conducted in this study also demonstrated that the different pore pressure evolutions in P- and S-specimens related to their transversal isotropy affects the state of failure of the rock mass around an excavation.
Acknowledgement

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6.7 References


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7. Conclusion and recommendations

7.1 Conclusion

This study was conducted with the aim of gaining a better understanding of the hydro-mechanical processes that control the rock mass behavior around an excavation of an underground repository in Opalinus Clay. Consolidated undrained and drained triaxial tests with different stress paths were conducted and analyzed to determine the effective rock properties of the material, including elastic properties (e.g. Young’s moduli, pore pressure coefficients), effective peak strength, and stress-strain or pore pressure evolution. Characterizing the effective rock properties helps to understand the hydro-mechanical processes observed around underground excavations (e.g. formation of an EDZ, pore pressure evolution during excavation), which is necessary to assess the short-term and long-term performance of a nuclear waste repository.

Studies on the undrained strength of Opalinus Clay reported in the literature show a significant scatter within the peak strength values. A review of these studies revealed that the tests have been conducted using different inconsistent procedures and the unknown initial effective stress state of the specimens influences the results. To deepen the understanding of the influence of suction on the geomechanical properties of Opalinus Clay, a first study was conducted in the framework of this thesis. Specimens were equilibrated at different levels of relative humidity in desiccators before they were tested for their Brazilian tensile strength or unconfined compressive strength. Furthermore, relationships between total suction and elastic properties such as Young’s modulus and Poisson’s ratio as well as the ultrasonic p-wave velocity were established. The study showed that total suction has a major influence on both stiffness and strength of Opalinus Clay. The unconfined compression and Brazilian tensile strength as well as the Young’s modulus increased almost linearly with increasing suction. A dependency of the material on the anisotropy was evident as the increase in strength was more pronounced for the Brazilian tensile strength normal to bedding and the unconfined compression strength for S-specimens. Furthermore, the p-wave velocity parallel to bedding remained almost constant whereas the p-wave velocity normal to bedding dropped sharply with increasing suction up to the air-entry value. It was concluded that this observation reflects the generation of new desiccation cracks whose formation is mostly within the bedding planes and limited to the early stage of desaturation.

These observations highlight the significant influence of suction on the geomechanical properties of Opalinus Clay. The influence has to be considered when assessing the performance of a nuclear waste repository during the open-drift phase since the rock mass around an excavation has been unloaded by the excavation and therefore may experience desaturation due to negative pore pressures or cavitation. Furthermore, the rock mass at the tunnel wall is exposed to the tunnel atmosphere, which applies a suction boundary. On the other hand, the influence of suction on the geomechanical properties also
implies the need for tests on fully back-saturated specimens in order to determine the material properties representative for in-situ conditions before and during excavation.

Consolidated drained and undrained tests are often used to characterize the drained and undrained behavior of clays or clay shales in the context of conventional and unconventional oil/gas extraction, CO2-sequestration, and nuclear waste repository design. Testing of such materials is challenging due to the low permeability which asks for long test durations, the material’s sensitivity to fluids, or the desaturation during sampling and handling of the specimens. In literature, no uniform testing procedures are found. A review of the major key aspects that have to be considered during testing is provided in this thesis. For the aim of back-saturating the specimen and establishing equilibrated effective stress states, a multi-stage testing procedure is inevitable. Such a procedure requires a saturation phase where saturation is re-established and confirmed. This avoids effects of suction and decreases the risk of desaturation upon dilation during shearing. Subsequently to saturation, consolidation of the specimen has to be considered to establish a uniform effective stress within the specimen, which eliminates the measurement of artefacts during the shearing stage and at the same time allows for measurements which are representative for the in-situ conditions. Finally, the specimens are sheared in a way as to allow for capturing pore pressure changes during undrained loading or to maintain the pore pressure during drained loading within valid bounds. For each of these stages, theoretical and data based considerations, that allow proper planning of triaxial testing, were compiled and their applicability was illustrated with the help of examples from consolidated drained and undrained tests on Opalinus Clay.

The results from the consolidated undrained and drained tests were used to investigate the effective rock properties and hydro-mechanical behavior of Opalinus Clay, which was the main goal of this thesis. In a first test series, standard triaxial stress paths were applied to P-specimens and S-specimens. The observed pore pressure response and effective geomechanical properties were analyzed with respect to their dependency on the confinement and the material’s anisotropy. The anisotropy is manifested in the poroelastic behavior of the material. Although Skempton’s $B$-value is the same (0.76-0.97, depending on the confinement), Skempton’s $A$-value differs significantly between P- and S-specimens. For a P-specimen, the $A$-value is smaller than 1/3 and reflects a smaller pore pressure build-up during standard triaxial shearing compared to a perfectly isotropic specimen. For a S-specimen, $A$ exceeds 1/3 and thus, more pore pressure is generated by a differential stress change. The anisotropy was also observed in the undrained Young’s modulus: the undrained Young’s modulus for P-specimens is about twice the Young’s modulus for S-specimens. A non-linear increase in undrained Young’s modulus (0.3-3.5 GPa) with increasing effective confining stresses (0.5-16 MPa) was observed for P-specimens. With regard to the hydro-mechanical behavior of the specimens, a change from overconsolidated to normally consolidated behavior was observed. Specimens consolidated at low effective stresses showed dilation whereas specimens consolidated at higher effective stresses showed compaction. At the same time, a change from brittle to more ductile behavior was observed with increasing confining stresses. These changes occurred gradually, somewhere between 5 and 8 MPa, and are also reflected by the effective
stress failure envelope. Although a linear fit through the effective peak strength values revealed a good correlation (with 35.3° friction angle, and 1.0 MPa cohesion), a non-linear fit with an decreasing friction angle for increasing confinement correlates well with the behavioral change of the material and can be related to the fabric of the material. The approximation of the non-linear failure envelope with a bi-linear Mohr-Coulomb criterion revealed a friction angle of 47.1° and a cohesion of 0.2 MPa for effective confinements < 5-8 MPa, and a friction angle of 31.9° and a cohesion of 2.0 MPa for effective confinements > 5-8 MPa, respectively. The undulating arrangement of the clay platelets in Opalinus Clay for distinct bedding planes which act as zones of weakness. This undulating fabric provokes dilatancy when sheared at low confining stresses, which adds an additional frictional resistance. With increasing confinement, dilation gets suppressed and the undulations will be sheared off and thus the friction angle (i.e. the sum of dilatancy and material friction angle) is reduced.

The results from the consolidated undrained and drained test series using a standard triaxial stress path revealed important insights into the hydro-mechanical behavior of the material and allowed for the determination of relevant effective geomechanical properties (i.e. strength, Young’s modulus, poroelastic parameters). However, the rock mass around an excavation usually does not experience a standard triaxial stress path but more complex stress changes. To investigate the influence of the stress path on the hydro-mechanical behavior of Opalinus Clay, a second test series was conducted. Consolidated undrained tests on back-saturated specimens were run, utilizing different controlled stress paths approximating a tunnel excavation. A two-dimensional stress path with isotropic initial stress conditions (i.e. pure shear compression), a two-dimensional stress path with anisotropic initial stress conditions, and a three-dimensional stress path were applied. The influence of the stress path was investigated with respect to the specimen’s geometry. For each stress path, a S-specimen and a P-specimen were tested. An influence of the anisotropy was observed, consistent with the observations found for the standard triaxial tests. For the two-dimensional stress paths, positive excess pore pressure for S-specimens and negative excess pore pressure for P-specimens were found. After the elastic response, a non-linearity in the pore pressure response related to pre-failure dilatancy of the specimens was observed for both geometries. The three-dimensional stress path which is a combination of the applied two-dimensional stress paths, an increase/decrease in octahedral mean stress, and a standard triaxial stress path, was able to conceptually reproduce the pore pressure evolutions that are commonly observed around excavations in clay shales. Thus, some fundamental geomechanical processes underpinning these in-situ pore pressure responses were detected. An increase in pore pressure as the tunnel face approaches the monitoring sensor location can be related to a poroelastic response of the specimens. In the laboratory tests, a pore pressure peak was reached before peak strength associated with the dilation of the specimen and a drop in pore pressure as observed in situ is therefore not solely related to failure. However, a drastic drop in pore pressure was not found in the segments of the stress path where the face is expected to pass the sensor location. Failure and the associated stress drop occurred later. In terms of effective peak strength, the failure criterion found for the triaxial tests remains
valid. Nevertheless, there is a significant difference between the two geometries with respect to the occurrence of failure, which is related to the different hydro-mechanical response of the specimens and is reflected by different effective stress paths. Failure for a S-specimen occurs at a lower effective stress compared to a P-specimen. Thus, transferred to a tunnel excavation in a transversely isotropic medium, the stability is affected by the anisotropy of the medium, also in case of an isotropic stress state, although the strength is assumed to be independent of the principal direction (i.e. the effective failure envelope is equal for P-specimen and S-specimen).

7.2 Recommendations for future research

The outcome of this thesis revealed fundamental insights into the hydro-mechanical behavior of Opalinus Clay. Nevertheless, there are still some issues that could not be solved and ask for further exploration.

The results of the laboratory experiments conducted in this thesis showed a significant influence of the anisotropy on the pore pressure evolution and the elastic properties. As P- and S-specimens only represent two end members for the influence of anisotropy, further tests on specimens loaded in different angles with respect to the bedding planes are recommended in order to characterize the full effect of anisotropy on the effective geomechanical properties and the hydro-mechanical behavior of Opalinus Clay. Furthermore, shear tests for characterizing the bedding directly in terms of pore pressure evolution and effective strength would contribute to this task. Bedding planes have been shown to be of importance especially in the context of the EDZ formation. Around a borehole drilled parallel to bedding, for example, failure initiated as shear along bedding planes (Kupferschmied et al. 2015). Shearing along the bedding planes in combination with extensional brittle failure was also observed around an tunnel excavation at the Mont Terri URL in cases where the bedding plane shear was kinematically free and the rock mass was only sparsely faulted (Thoeny 2014).

Furthermore, it is important to implement the hydro-mechanical behavior into numerical models. Especially the yielding before peak strength, the hydro-mechanical response, and the anisotropy should be considered. The model should be capable to represent the response of the rock to a tunnel stress path including the dependency on the material’s anisotropy, which significantly affects the state of failure around an excavation (although the effective peak strength of the matrix can be described by a single failure envelope). The availability of such a constitutive model for numerical simulations is of particular relevance. When the short-term undrained behavior around an excavation can be represented adequately, such models can support the performance assessment of deep geological disposal sites for nuclear waste. Moreover, scenarios of the subsequent phases of an underground excavation, including the open-drift and the long-term phase where further (thermo)-hydro-mechanical-(chemical)-processes become relevant, can be simulated, starting from the short-term state after excavation.
To support the calculation for the behavior during the open-drift and long-term phases, permeability measurements of the material are needed. Permeability should be determined with respect to the influence of anisotropy and confining stresses. Possible change in permeability caused by damage related to the formation of an EDZ has probably to be accounted as well.

Further complexity will be added to the assessment of the longer-term behavior when the rock mass is not saturated anymore. In the case of saturation, the geomechanical properties found in this study are still valid and especially the failure criteria can still be applied. However, drained behavior has to be considered as pore pressures will dissipate in the longer term. In the case of a partially saturated rock mass, the effective stress law would not be valid anymore and the material would have to be characterized with respect to its degree of saturation. Therefore, the work on the influence of suction on the geomechanical properties might be further expanded to include also triaxial conditions and hydro-mechanical behavior.

7.3 References


Appendix

A Some fundamental hydro-mechanical processes relevant for understanding the pore pressure response around excavations in low permeable clay rocks

Wild, K.M., Amann, F., Martin, C.D.


Abstract: Argillaceous rock formations, in particular clay shales, are notorious for creating challenges in rock engineering. Due to their generally low hydraulic conductivity and their complete saturation in their natural state at depth, the behavior of clay shales is usually characterized by a strong hydro-mechanical coupling.

This paper presents typical pore pressure responses monitored in different underground rock laboratories and explores alternative ways for their interpretation. The underpinning processes and their relevance are identified based on analytical and conceptual numerical models. Pore pressure monitoring data from Opalinus clay at the Mont Terri Underground laboratory, the Boom Clay at the Mol facility and the Callovo-Oxfordian clay in the Meuse/Haute–Marne URL show a similar pore-pressure response associated with stress changes accompanying tunnel construction although the tunnels were built in different rock types, at different locations, and under different in-situ conditions.

The numerical analyses in this study shows that the pore pressure response around an excavation in low permeable clay shales may have different explanations. Pore pressure drops observed in the tunnel near field can theoretically be explained by both, a pure elastic response in an isotropic or anisotropic rock mass with an anisotropic stress state, or an inelastic response and related dilatancy. However, a comparison between the numerical models and the in-situ measurements revealed that the latter is more likely. This conclusion is justified by the observation that pore pressure drops occur significantly ahead of the tunnel face which is not the case in elastic numerical simulations. Pore pressure measurements in the far field, where failure processes are unlikely to occur, can only partly be explained by a linear poroelastic models. The introduction of an anisotropic material plays a key role in reproducing the responses, conceptually.

A.1 Introduction

Among others, argillaceous rock formations are considered as favorable host rocks for nuclear waste repositories due to their extremely low permeability, the high sorption capacity and the potential for self-sealing. However, argillaceous rocks, especially clay shales, are notorious for creating challenges
in rock engineering. Experience from tunnels, slopes and dams constructed in and on these materials highlights the challenges in predicting their response to loading and/or unloading. In the context of underground repositories for nuclear waste, the relatively low strength and strength anisotropy of clay shales may affect the constructability of underground disposal tunnels at depths of 300-900 m, and the advantageous properties may be altered due to the damage (inelastic deformations) induced by the construction of an underground excavation. Consequently, the natural barrier function of a host rock can be affected negatively (Blümling et al. 2007). It is therefore necessary to identify the characteristics and properties that dominate the behavior of such materials when assessing their short-term and long-term performance.

Due to the generally low hydraulic conductivity and the complete saturation in the natural state at depth, the behavior of clay shales is characterized by a strong hydro-mechanical coupling. Analyzing the excavation phase (hours) of underground excavations in clay shales, these processes have to be taken into account, i.e., one has to consider how mechanical changes influence the hydraulic behavior of the rock mass and vice versa. Undrained behavior (i.e., the mass of the pore fluid within the porous medium remains constant when a stress increment is applied) dominates the short-term pore pressure response around underground excavation when the advance rate is high (> 0.1m/hr) and the hydraulic conductivity of the rock mass is low (< $10^{-13}$ m/s). Seepage flow is therefore insignificant (Anagnostou and Kovári 1996). The excavation phase is accompanied by excavation induced stress redistribution around the tunnel and the development of a so-called excavation damage zone (EDZ). Within this fractured zone, shear and extensional failure are typically observed depending on the orientation of the tunnel with respect to the bedding plane orientation and in-situ state of stress (Yong et al. 2013, Thoeny 2014). Outside the heavily fractured EDZ a so called excavation disturbed zone (EdZ) exists where the rock mass undergoes elastic, mechanical processes (Tsang et al. 2005).

Due to induced stress changes and the associated mechanical behavior, a change in pore water pressure occurs around the tunnel, which in turn influences the mechanical processes. Since the state after the excavation phase sets the conditions for long-term processes (e.g., consolidation and swelling behavior), it is relevant for future nuclear waste repositories to predict the initial state of pore pressure perturbation. However, interpretation of pore pressure data monitored around excavation in clay shale is challenging and depends on various factors. These factors include the poroelastic response, the inelastic, dilatant yielding behavior, the stiffness, strength and in-situ stress anisotropy, and the stress path. This paper discusses typical pore pressure responses monitored during excavations in different underground rock laboratories (URLs) in clay shales. Based on analytical solution and conceptual numerical models, fundamental processes, which influence the pore pressure evolution are presented, and different ways of interpretation for the observed pore pressure responses are explored.
A.2 Field observations of pore pressure evolution during excavation

Pore pressure measurements during excavations of test drifts in different underground laboratories in clay shales have been reported by various authors (e.g., Corkum and Martin 2007, Yong 2007, Martin et al. 2011, Neerdael et al. 1991, Verstricht et al. 2003, Wileveau and Bernier 2008, Armand et al. 2014). Even though these measurements were conducted in different rock types, at different locations, and under different in-situ conditions, similar patterns in the pore pressure response during tunnel excavation can be recognized (examples are given in Figure A.1).

Figure A.1: Examples for typical pore pressure responses: a) response in the vicinity of an excavation, i.e., within 1.2 tunnel radius (data from Martin et al. 2011), b) example 1 pore pressure response further away from the gallery, i.e., at 1.3-2.5 times the tunnel radius (data from Corkum and Martin 2007), c) example 2 pore pressure response further away from the excavation, i.e., at 2.3-3.0 times the tunnel radius (data from Masset 2006).

Generally, all sensors show a gradual increase in pore pressure even if the front of the excavation is far away from their location. When the tunnel face approaches, the pore pressure response in the vicinity of an excavation (out to about 1.3 times the tunnel radius, i.e. near field) shows the same behavior at all sites: ahead of the tunnel face pore pressure increases abruptly up to 1.6 times the initial value (Figure A.1a). The magnitude and timing of this “pore pressure spike” depends on the distance between sensor and excavation. Its magnitude decreases with increasing distance but pore pressure peaks appear earlier for sensors located further away (but within 1.3 times the tunnel radius). Either before or shortly after the excavation front passes the sensor location, the pore pressure drops to or below atmospheric pressure. A pore pressure drop to atmospheric conditions suggests that the sensor is connected to the tunnel atmosphere through EDZ fractures. A drop in pore pressure to values below the atmospheric pressure but larger than zero suggest no connection to the tunnel atmosphere. For some sensors (e.g., in Boom Clay (Bernier et al. 2003)), even absolute negative pressure have been recorded. The pore pressure stayed depleted for a long time after excavation.

At distances > 1.3 times the radius of excavation, (i.e., far field), two different pore-pressure responses can be distinguished. The first response is similar to the one described above but here the pore pressure does not drop to atmospheric pressure but to values smaller than the initial one when the tunnel face passes (Figure A.1b). The magnitude of the pore pressure drop increases with decreasing distance to the
tunnel. This response is observed for sensors whose shortest path to the excavation is oriented sub-perpendicular to the bedding plane orientation. The second type of pore pressure response in the far field is measured at sensors whose path to the tunnel is sub-parallel to the bedding planes. Such sensors show pore pressures that increase ahead of the tunnel face and remain elevated (Figure A.1c). A slight variation of this response has been monitored in Boom Clay. There, the pore pressure gradually increased prior to excavation, slightly decreased as the front of the excavation approached and then started to increase to values larger than the initial pore pressure as the face passed the sensor location. The values reached after the excavation passed the sensor location are again dependent on the distance between the sensor and the excavation.

The review of measured pore pressure responses around tunnel in low permeability rock types suggests that pore pressure changes around excavations in low permeable clay shales can be measured even at large distances from the tunnel wall (i.e., at distance/tunnel radius > 7) and two fundamentally different response types:

1) A pore pressure decrease can be observed in both the near- and far-field as defined above, i.e., in both the EDZ and Edz
2) A pore pressure increase can be observed in the Edz but also ahead of the tunnel face

The fundamental geomechanical processes associated with these pore pressure response types at different locations around underground excavations are not well understood. The following section addresses possible explanations. First the theory of poroelasticity will be explored. In a second step conceptual numerical models are conducted with various properties for the rock mass and in-situ state of stress ratio.

A.3 Theoretical aspects

A.3.1 Undrained poroelastic response

Considering an isotropic, linear elastic material behavior, pore pressure changes $\Delta u$ can, according to Skempton (1954), be related to changes in octahedral normal stress $(\Delta \sigma_m = (\sigma_1+\sigma_2+\sigma_3)/3)$: $\Delta u = B \Delta \sigma_m$ (1), where $B$ is the Skempton’s coefficient. As an alternative expression, the pore pressure evolution can be linked to the change in volumetric strain $\Delta \varepsilon_{vol}$ (Detournay and Cheng 1993): $\Delta u = -\alpha M \Delta \varepsilon_{vol}$, where $M$ is the Biot modulus and $\alpha$ is the Biot Coefficient.

For an infinite homogeneous, isotropic elastic medium, radial and tangential stresses around a horizontal circular tunnel under two dimensional plane strain conditions can theoretically be calculated using the analytical solutions of Kirsch (1898). For an isotropic stress distribution, the mean stress doesn’t change around the tunnel and hence no change in pore pressure occurs according to equation (1). For an anisotropic in-situ stress state, however, the mean stress changes depending on the location around the
tunnel. Pore pressures are therefore either increased or decreased locally, solely associated with an elastic mechanical response.

However, the above example represents a simplified model which does not consider the poroelastic behavior of an anisotropic porous media. Most clay shales show a distinct bedding and therefore cannot be considered as an isotropic material but rather as transversely isotropic. Bellwald (1990) and Aristorenas (1992) investigated the hydro-mechanical behavior of Opalinus Clay using undrained pure-shear compression and extension tests. The selected total stress paths do not cause a change in octahedral mean stress and are exactly equal to the plane strain stress path at the tunnel crown and invert for a circular tunnel in an isotropic in-situ stress and poroelastic medium. Assuming the theory of an isotropic poro-elastic medium is valid, these stress paths should not lead to any pore pressure change ($\Delta u$). Bellwald (1990) and Aristorenas (1992), however, demonstrated that excess pore pressures in Opalinus Clay develop even under these conditions. At low differential stress ($\sigma_{1}-\sigma_{3}$) (i.e., before the onset of dilatancy), pore pressure increases within the specimen under pure shear compression and pore pressure decreases under pure shear extension (for specimens loaded parallel to the bedding). The effective stress path therefore deviates from the total stress path. The observed excess pore pressures can be related to the transverse isotropy of the rock (Aristorenas 1992, Einstein 2000, Bobet et al. 1999). An isotropic porous material would not have shown excess pore pressures before the onset of dilatancy in pure shear compression and the effective stress path would have been identical with the total stress path for stress states below crack initiation. Similar findings have been reported by Barla (1999) for Caneva clay and Islam and Skalle (2013) for shales under undrained triaxial loading conditions.

For the examples shown above, plane strain conditions are assumed and the three-dimensional nature of stress perturbation around underground excavations is not considered. A plane strain assumption is only valid for a tunnel cross section at a considerable distance from the tunnel face, and the analytical solutions for the tangential and radial stress distribution around the tunnel based on plane strain only allow one to predict pore pressure magnitudes at the end of excavation. To capture the complete evolution of pore pressure during the excavation three-dimensional stress paths have to be analyzed. Barla (2000, 2007) investigated the elastic stress paths around a circular tunnel with a diameter of 10 m in two and three dimensions numerically. He demonstrated that the stress state after complete stress redistribution (i.e. when the distance to the tunnel face is much larger than the tunnel diameter) is the same for both, the two and three dimensional analyses and equals the stress state derived from the Kirsch solution (Kirsch 1898). However, the 3D stress path deviated substantially from the plane strain solution when the tunnel approaches and passes the monitoring cross section. Before the tunnel face arrives at the monitoring section the mean stress increases. When the tunnel face passes the mean stress decreases rapidly and reaches a value smaller than the in-situ mean stress. With increasing distance between the monitoring section and the tunnel face, the mean stress again increases and becomes eventually equal to the in-situ mean stress.
A.3.2 Undrained response considering failure

So far, only pore pressure changes in an elastic medium have been discussed. However, as mentioned above, the excavation phase is often accompanied by the development of fractures that form an EDZ, i.e. the ground does not behave elastically and may locally yield. As yielding occurs tangential stress drops (and hence also the mean stress) and dilation leads to volumetric strains, causing a decrease in pore pressure. Mair and Taylor (1993) used simple plasticity solutions to predict ground deformations and pore pressure changes caused by a tunnel constructed in clays. A linear elastic-perfectly plastic soil behavior in an isotropic in-situ stress field was assumed. Their model showed that the pore pressure decreases within the plastic zone. However, observed pore pressure changes in the elastic zone could not be reproduced by their model. Thus, non-linear elastic behavior was assumed that allowed to reproduce decreased pore pressure also in the elastic zone (Mair 1979, Neerdael and De Bruyn 1989). This assumption caused, however, a very big zone in which the pore pressure is decreased. Closed-form solutions for the short-term response of a poroelasto-plastic medium with compressible constituents (pore fluid and grains) due to instantaneous unloading of a cylindrical borehole have also been presented by Labiouse and Giraud (1998). The ground is assumed to be homogeneous and isotropic and to behave linearly-elastic, perfectly-plastic according to a Mohr-Coulomb failure criterion. Plane strain conditions and a hydrostatic initial stress field (total stress and pore pressure) were assumed. No pore pressure changes occur in the elastic zone due to the assumption that in an isotropic elastic material in an isotropic in-situ state of stress no mean stress changes and thus no volume change take place. The pore pressure within the plastic zone decreases (and can even reach negative values) as a consequence of volumetric straining. Labiouse and Giraud (1998) showed that the magnitude of pore pressure decrease and the extent of the plastic zone depends not only on the strength parameters but also on the compressibility characteristics of the solid matrix and the pore water. Anagnostou (2009) presented a closed-form solution similar to the one by Labiouse and Giraud (1998) but using the simplifying assumption of incompressible constituents (pore water and grains). Therefore, the pore pressure only depends on the strength parameters of the medium. However, the solutions by Labiouse and Giraud (1989) as well as the one by Anagnostou (2009) are only valid for an isotropic in-situ stress field. Analytical solutions for tunnel induced stress distribution for a non-uniform stress field in an elasto-plastic medium have rarely been discussed in literature. Detournay and Fairhurst (1987) presented a semi-analytical solution for the stress field and displacement in the elastic region but used a numerical solution for the plastic region.

A.3.3 Discussion

The considerations above show that pore pressure changes accompanying tunnel excavations in a low permeability saturated porous medium can have different explanations. Pore pressure change in the elastic domain only occurs for an anisotropic in-situ stress state or if the material is anisotropic. Around a tunnel excavated in an isotropic elastic medium within an isotropic initial stress field, no pore pressure changes are expected to be present at the end of the excavation phase. However, three-dimensional stress
paths have to be considered to capture the complete pore pressure evolution during the excavation. Taken this into account, also for isotropic materials in uniform in-situ stress fields, pore pressure changes may develop during the excavation, although there are not recognizable at a large distance to the tunnel face. In addition, dilatancy accompanying failure around an excavation in elasto-plastic materials will lead to a pore pressure decrease.

The geological environments where the excavations for nuclear waste repositories are supposed to be built are often characterized by an anisotropic stress state and an anisotropic mechanical behavior. Therefore, a complex combination of the factors influencing the pore pressure evolution around an excavation discussed above (i.e. stress-path, poroelastic behavior, in-situ stress state, and failure) has to be considered. In the following, some of these aspects are analyzed using three-dimensional conceptual numerical models.

### A.4 Conceptual numerical study – model description

Three-dimensional conceptual numerical models were conducted to analyze the pore pressure evolution during the excavation phase, using the commercially available three-dimensional continuum code FLAC3D (Fast Langrangian Analysis of Continua in 3 Dimensions, Itasca 2009). The model geometry is shown in Figure A.2a. The model consist of a circular tunnel, with the diameter of 4m. The model extent is 24 m in both x- and z-direction, and 26 m in y-direction (Figure A.2a). A full-face excavation sequence with a round length of 0.2 m was used, starting at the front face of the model. The tunnel excavation was stopped 6 m before the backside of the model. No support measures were installed after excavation. Both, an isotropic stress state (i.e. \( \sigma_1 = \sigma_2 = \sigma_3 = 6.5 \text{ MPa} \)) and a stress state with \( K_0 = 0.5 \) (i.e. \( \sigma_1 = 6.5 \text{ MPa}, \sigma_2 = \sigma_3 = 3.25 \text{ MPa} \)) were applied to analyze the influence of the in-situ stress on pore pressure evolution.

![Figure A.2: a) model geometry and dimensions of the FLAC3D model, b) location of monitoring plane, c) setup of monitoring points recording stress and pore pressure after each excavation round](image)

All in-situ principal stresses were assumed to align with the chosen coordinate system (i.e., \( \sigma_1 \) aligned with the z-, \( \sigma_2 \) and \( \sigma_3 \) with the y- and x-axis). Pore pressure was initiated at 2 MPa. Principal stresses \( \sigma_1, \sigma_2, \sigma_3 \), and pore pressure were monitored in a plane normal to the tunnel axis in the middle of the tunnel (i.e., at \( y = 10 \text{ m} \)) (Figure A.2b). A monitoring array with 147 monitoring points was used (Figure A.2c).
The points were arranged at different angles $\beta$ with respect to the z-axis ($\beta = 0^\circ-180^\circ$) and with different radial distances to the tunnel center (2.25, 2.5, 3.0, 4.0, 5.0, 6.0, and 7.0 m).

For all conceptual models, a fully-coupled approach was used assuming undrained behavior. All constituents were assumed to be incompressible. The rock mass was assumed to be completely saturated, with a porosity of 18%. A fluid bulk modulus of 2 GPa was utilized (i.e. assuming deaired water). Different constitutive models were used to characterize the rock mass: 1) Linear Elastic models which were either isotropic or transversely isotropic (with different orientations of bedding planes), and 2) isotropic linearly-elastic, brittle-plastic models based on a linear Mohr-Coulomb yield criterion. Pre-peak dilation was not considered for this conceptual models. A Mohr-Coulomb model with nonassociated shear and associated tension flow rules was assumed. Dilation is not taken into account.

A.5 Results of conceptual numerical models

A.5.1 Poroelastic response

Figure A.3 shows results from linear elastic models with different in-situ stress states and material behavior. An isotropic elastic model with $K_0 = 1.0$ shows an increase in pore pressure before the face arrives at the sensor location (Figure A.3a) as it has been observed for in-situ measurements both near the excavation and further away of it. However, the pore pressure at the end of the excavation ($p$) doesn’t change compared to its initial value ($p_0$) in the model, which is different to the observations made in the URLs. This changes when an anisotropic stress state (e.g., $K_0 = 0.5$) is taken into account instead of an isotropic stress state. Results of such models show that pore pressure changes irreversibly during excavation and stays either elevated or depleted compared to the initial value depending on the sensor location (Figure A.3b).

The analysis also shows that for the assumed in-situ pore pressure the pore pressure at the end of the excavation can drop to atmospheric or even absolute negative values (providing cavitation does not occur) in vicinity of the excavation. The pore pressure changes in these isotropic elastic models can
directly be related to changes in octahedral mean stress considering a 3D stress path in accordance to Skempton (1954).

Similar results are obtained when a transversely isotropic elastic material behavior is considered (Figure A.3c). Also here, pore pressure increases for all sensor locations as the excavation face approaches. However, the peak values which occur slightly before the tunnel face passes the monitoring section show a wider range and are generally higher than for the isotropic material behavior. The model also shows that the pore pressure increase tends to begin earlier at larger distances as compared to sensors in close vicinity to the face, which is consistent with field observations. The magnitude of pore pressure when the tunnel approaches the sensor array is dependent on the orientation of the plane of isotropy and the anisotropy ratio (i.e., $E_1/E_3$; for the models shown here a ratio $E_1/E_3 = 3.3$ was used). The values reached in the transversely isotropic elastic models with $K_0 = 1.0$ are 2-3 times the initial pore pressure. After the face passed the sensor location, the pore pressure drops again but stays either elevated or reaches lower values at the end of excavation compared to the initial value (Figure A.3c). The range of values covered at the end of excavation is again strongly dependent on the bedding orientation as well as the anisotropy ratio. For example, the range of pore pressure at the end of excavation around a tunnel in an elastic medium with horizontal bedding increases with increasing ratio $E_1/E_3$. Different models with various bedding orientations showed that the biggest drop or the highest value at the end of excavation are obtained when choosing the strike of the bedding planes to be parallel to the direction of advance (Figure A.4).

Finally, a model combining a transversely isotropic, elastic material behavior and an anisotropic stress state (e.g. $K_0 = 0.5$) leads to even more distinct results (Figure A.3d).
A.5.2 Poromechanical response

Although poroelastic models can explain the pore pressure evolution observed, there is clear evidence that stress redistribution and inelastic straining during excavation lead to the formation of an EDZ. Shear and extensional fractures are typically found within this zone (Bossart et al. 2002, Marschall et al. 2006, 2008, Nussbaum et al. 2011, Thoeny et al. 2014, Yong et al. 2013). Therefore, a Mohr-Coulomb brittle-plastic models was used to explore the pore pressure evolution in the vicinity of an excavation. Figure A.5 shows the pore pressure evolution in the roof at a distance of 0.25m to the tunnel of an isotropic, elastic, brittle-plastic model compared to different elastic models. It can be seen that the increase in pore pressure before the face starts significantly earlier in the elasto-plastic model than in the elastic models. Looking at the field measurements, it can be seen that in the vicinity of an excavation the peak is also reached before the tunnel face and the drop starts significantly before the face passes (Figure A.1a). Given the similarities it is reasonable to assume that pore pressure drops that occur ahead of the tunnel face are related to the formation of an EDZ. However, the peak value reached in the elasto-plastic model is not as high as the observed one and also smaller than the peak of the elastic models.

![Figure A.5](image)

Figure A.5: Comparison of pore pressure responses from different elastic models and an isotropic elasto-plastic model (plasticity indicators are shown in the right part of the figure). The monitoring point is located at the roof at a radial distance of 0.25m as shown in the right part of the figure. It can be seen, that yielding occurs ahead of the tunnel face.

A.6 Concluding remarks

In-situ pore pressure measurements around tunnels at underground laboratories in clay shales have demonstrated that significant pore pressure changes occur in the vicinity of the tunnel face during the advance. Three-dimensional models are needed to track the tunnel face advance since 2D models only provide the pore-pressures far from the tunnel face. The three dimensional conceptual numerical analyses carried out for this study shows that some key processes associated with the observed behavior around tunnel in clay shales can be reasonably explained using fundamental rock mechanical constitutive models that account for the material behavior.
The analyses in this study show that the pore pressure response around excavations in low permeable clay shales may have different explanation. Major pore pressure drops in the tunnel near field can, in theory, be explained by both a pure elastic response in an isotropic or anisotropic rock mass characterized by an anisotropic stress state, or by dilatancy which accompanies failure. A pore pressure drop to atmospheric pressure clearly suggests that fractures that formed during the excavation provide a direct link to the tunnel environment. Pore pressure drops to values below the nearest atmosphere (i.e., suction) do not suggest a direct connection to the tunnel and can be related to both, a poroelastic response or fracturing which does not lead to an interconnected fracture network. Conceptual numerical model results suggests that pore pressure drops in an elastic medium occur shortly before the tunnel face passes the monitoring point. This is not consistent with and actual pore pressure measurements which show that the pore pressure drop can occur significantly before the tunnel face. It is thus concluded here that near-field pore pressure drops are primarily related to yielding processes.

Pore pressure measurements in the far field, where failure processes are unlikely to occur, can only partly be explained by a linear poroelastic model. A pore pressure response as shown in Figure 1c, for example, cannot be explained by an isotropic poroelastic model even when an anisotropic state of stress is assumed. The introduction of a transversal isotropic elastic model enabled reproducing this response conceptually.

Acknowledgments

This study was funded by the Swiss Federal Nuclear Waste Inspectorate (ENSI).

A.7 References


B Core logs

The specimens for the laboratory tests conducted in the framework of this thesis were obtained from a borehole drilled from the FE-A-Niche (BFE-A3) and from two boreholes drilled from Gallery 08 of the Mont Terri URL at GM 59 (BHM-1, BHM-2). Borehole BFE-A3 was drilled from the tunnel invert parallel to bedding (azimuth: 250.95°, dip: 6.3°), borehole BHM-1 was drilled from the tunnel invert parallel to bedding (azimuth: 230°, dip: 0°), and borehole BHM-2 was drilled from the tunnel floor normal to bedding (azimuth: 320°, dip: 55°). The geological and structural mapping of the drillcores are presented in the following. Additionally, the location of the individual laboratory specimens used for the CD and CU tests are indicated. For the location of the individual specimens for the laboratory tests presented in Chapter 2, the reader is referred to Zimmer (2012) and Wymann (2013).
B.1 Core mapping of BFE-A3

Figure B.1: Drillcore map of BFE-A3 (Flach et al. 2013).
Borehole BFE-A03
From 5 - 10 m

Drillcore mapping: dol
Date: 01.12.11
Scale 1:7.5

Drill orientation: W

Figure B.2: Drillcore map of BFE-A3 (Flach et al. 2013).
Figure B.3: Drillcore map of BFE-A3 (Flach et al. 2013).
Figure B.4: Drillcore map of BFE-A3 (Flach et al. 2013).
Figure B.5: Drillcore map of BFE-A3 (Flach et al. 2013).
Figure B.6: Drillcore map of BFE-A3 (Flach et al. 2013).
Figure B.7: Drillcore map of BFE-A3 (Flach et al. 2013).
Figure B.8: Drillcore map of BFE-A3 (Flach et al. 2013).
B.2 Core mapping of BHM-1

Borehole BHM-1

Drillcore mapping: JAD

From 0 m - 3 m

Date: 05.11.2012

Scale: 1 : 7.5

Lithology / bedding | Structures | Samples
---|---|---
0 m | 1 m | 2 m | 3 m

Remarks:
- Borehole parallel to strike
- Bedding, azi: 230°
- Dip: 0°

Figure B.9: Drillcore map of BHM-1 (Häusler et al. 2014).
Borehole BHM-1

Drillcore mapping: gj

From 3 m - 6 m

Date: 06.11.2012

Scale: 1 : 7.5

Lithology / bedding | Structures | Samples
--- | --- | ---
3 m | ad | ETH02
4 m | ad | ETH03
5 m | ad | ETH04
6 m | ad | ETH05
7 m | ad | ETH06
8 m | ad | ETH07

Figure B.10 Drillcore map of BHM-1 (Häusler et al. 2014).
Borehole BHM-1

Drillcore mapping: gi

From 6 m - 9 m
Date: 07.11.2012
Scale: 1 : 7.5

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<th>Structures</th>
<th>Samples</th>
<th>Lithology / bedding</th>
<th>Structures</th>
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<tr>
<td>N trending faults</td>
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<td></td>
<td>bioclastic limestone</td>
<td></td>
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Figures B.11: Drillcore map of BHM-1 (Häusler et al. 2014).
Figure B.12: Drillcore map of BHM-1 (Häusler et al. 2014).
**Figure B.13: Drillcore map of BHM-1 (Häusler et al. 2014).**

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</tr>
<tr>
<td>12.00 - 12.35</td>
<td>ad (sawed)</td>
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<td>12.35 - 13.17</td>
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<td>13 m</td>
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<td>13.17 - 13.40</td>
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<td>14 m</td>
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<tr>
<td>14.03 - 14.84</td>
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<td></td>
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<tr>
<td>15 m</td>
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**Lithology:**
- sandy layers
- sandy nodules
- silty layers
- ammonites
- bioclastic limestone
- bedding
- N trending faults
- SE-dipping fault plane with striae (sst)
- SW-dipping fault plane with striae (sst)
- EDZ unloading joint (uj)
- SE- dipping fault plane with striae (sst)
- SW- dipping fault plane with striae (sst)
- artificial discontinuity (ad)

**Structures:**
- bedding
- N trending faults
- SE-dipping fault plane with striae (sst)
- SW-dipping fault plane with striae (sst)
- EDZ unloading joint (uj)
- SE- dipping fault plane with striae (sst)
- SW- dipping fault plane with striae (sst)
- artificial discontinuity (ad)

**Samples:**
- 12 m
- 13 m
- 14 m
- 15 m

Scale: 1 : 7.5

Date: 07.11.2012
Borehole BHM-1

Drillcore mapping: gj

From 15 m - 18 m

Date: 07.11.2012

Scale: 1 : 7.5

Figure B.14: Drillcore map of BHM-1 (Häusler et al. 2014).
Figure B.15: Drillcore map of BHM-1 (Häusler et al. 2014).
Figure B.16: Drillcore map of BHM-1 (Häusler et al. 2014).
B.3 Core mapping of BHM-2

Borehole BHM-2

Drillcore mapping: gji

From 0 m - 3 m

Date: 13.11.2012

Scale: 1 : 7.5

Figure B.17: Drillcore map of BHM-2 (Häusler et al. 2014).
Figure B.18: Drillcore map of BHM-2 (Häusler et al. 2014).
Figure B.19: Drillcore map of BHM-2 (Häusler et al. 2014).
### Borehole BHM-2

**Drillcore mapping: gij**

**From** 9 m - 12 m  
**Date:** 13.11.2012  
**Scale:** 1 : 7.5

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**Figure B.20: Drillcore map of BHM-2 (Häusler et al. 2014).**
Figure B.21: Drillcore map of BHM-2 (Häusler et al. 2014).
Borehole BHM-2

Drillcore mapping: gij

From 15 m - 18 m

Date: 14.11.2012

Scale: 1 : 7.5

Lithology / bedding | Structures | Samples
--- | --- | ---
Silty layer | 15 m | 53
Silty layer | 16 m | 14.95 - 15.90
Silty layer | 17 m | 16.60 - 17.30
Silty layer | 18 m | 17.30 - 18.15

Fractures:
- Artifical discontinuity (ad)
- SE-dipping fault plane with striae (sst)
- NW-dipping fault plane with striae (sst)
- EDZ unloading joint (uj)
- N-trending faults

Lithology:
- Sandy layers
- Silty layers
- Bivalves
- Ammonites
- Bioclastic limestone

Figure B.22: Drillcore map of BHM-2 (Häusler et al. 2014).
Figure B.23: Drillcore map of BHM-2 (Häusler et al. 2014).
Borehole  BHM-2

Drillcore mapping: gij

From 21 m - 22 m

Date: 14.11.2012

Scale: 1 : 7.5

Figure B.24: Drillcore map of BHM-2 (Häusler et al. 2014).
B.4 References


## C Triaxial test results

### C.1 General properties

Table C.1.1: General properties of the tested specimens. Specimens in grey were excluded for the analyses because complete consolidation could not have been guaranteed.

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<th>height (mm)</th>
<th>borecore</th>
<th>depth (m)</th>
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<td>S</td>
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<td>136.15</td>
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<td>134.77</td>
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<td>4.60-4.80</td>
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<td>67.47</td>
<td>134.15</td>
<td>BHM-1</td>
<td>17.00-17.20</td>
</tr>
<tr>
<td>ETH15</td>
<td>P</td>
<td>2D-iso</td>
<td>67.41</td>
<td>133.63</td>
<td>BHM-1</td>
<td>16.80-17.00</td>
</tr>
<tr>
<td>ETH15_2</td>
<td>P</td>
<td>2D-iso</td>
<td>67.51</td>
<td>135.30</td>
<td>BHM-2</td>
<td>5.15-5.35</td>
</tr>
<tr>
<td>ETH16</td>
<td>P</td>
<td>CU</td>
<td>67.47</td>
<td>133.00</td>
<td>BHM-1</td>
<td>18.70-18.93</td>
</tr>
<tr>
<td>ETH17</td>
<td>P</td>
<td>CU</td>
<td>67.47</td>
<td>135.34</td>
<td>BHM-1</td>
<td>17.90-18.10</td>
</tr>
<tr>
<td>ETH18</td>
<td>P</td>
<td>CU</td>
<td>67.54</td>
<td>134.74</td>
<td>BHM-1</td>
<td>17.70-17.90</td>
</tr>
<tr>
<td>ETH19</td>
<td>P</td>
<td>CU</td>
<td>67.50</td>
<td>134.40</td>
<td>BHM-1</td>
<td>17.20-17.40</td>
</tr>
<tr>
<td>ETH20</td>
<td>P</td>
<td>CU</td>
<td>67.30</td>
<td>134.10</td>
<td>BHM-1</td>
<td>17.40-17.70</td>
</tr>
<tr>
<td>ETH20_2</td>
<td>P</td>
<td>CU</td>
<td>67.65</td>
<td>133.30</td>
<td>BHM-1</td>
<td>5.50-5.70</td>
</tr>
<tr>
<td>ETH21</td>
<td>P</td>
<td>CU</td>
<td>67.56</td>
<td>133.99</td>
<td>BHM-1</td>
<td>20.70-20.90</td>
</tr>
<tr>
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<td>P</td>
<td>CU</td>
<td>67.45</td>
<td>134.31</td>
<td>BHM-1</td>
<td>20.30-20.50</td>
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<tr>
<td>ETH23</td>
<td>P</td>
<td>CU</td>
<td>67.45</td>
<td>133.81</td>
<td>BHM-1</td>
<td>20.70-20.90</td>
</tr>
<tr>
<td>ETH24</td>
<td>P</td>
<td>CU</td>
<td>67.43</td>
<td>134.11</td>
<td>BHM-1</td>
<td>21.40-21.60</td>
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</table>
C.2 Water content and saturation

Table C.2.1, Table C.2.3, and Table C.3.1 show the water contents, porosities, and saturations for the different specimens before the test, after the test and after the saturation/consolidation phase, respectively. The dry density, porosity, and saturation were determined according to the ISRM suggested methods (ISRM 1979). The water content was determined with respect to the weight of the specimens after 24h drying at 105°C. Specimens ETH10_2, ETH20_2, ETH21, ETH22, ETH23, and ETH24 were dried to constant weight. The comparison between the water content calculated after 24h drying and after drying to constant weight (reached after about 2 days) for these specimens revealed that the water content after 24h probably underestimates the water content by about 0.4 %. Similar results have been observed for specimens tested by Amann et al. (2011) and Amann et al. (2012).

The water content after saturation and consolidation were estimated using the weight of the specimen after saturation/consolidation (calculated by adding and subtracting the measured amount of water flown into or out of the specimen during saturation phase and consolidation phase, respectively, to the initial weight before testing). Grain density for Opalinus Clay ranges between 2.69 and 2.78 g/cm³ (Pearson et al. 2003, Bossart 2005, own data). A mean value of 2.73 g/cm³ was considered in this study to calculate the porosity. Note that the estimated values of saturation strongly depend on this parameter.
Table C.2.1: Water content, porosity, and saturation for the individual specimens during different phases of the CU and CD tests. Specimens in grey were excluded for the analyses because complete consolidation could not have been guaranteed.

<table>
<thead>
<tr>
<th></th>
<th>ETH01</th>
<th>ETH02</th>
<th>ETH03</th>
<th>ETH04</th>
<th>ETH05</th>
<th>ETH06</th>
<th>ETH07</th>
<th>ETH08</th>
<th>ETH09</th>
<th>ETH10</th>
</tr>
</thead>
<tbody>
<tr>
<td>weight before test</td>
<td>(g)</td>
<td>1192.1</td>
<td>1175.0</td>
<td>1167.8</td>
<td>1158.1</td>
<td>1172.9</td>
<td>1141.5</td>
<td>1140.0</td>
<td>1082.3</td>
<td>1192.1</td>
</tr>
<tr>
<td>weight after test</td>
<td>(g)</td>
<td>1211.1</td>
<td>1194.2</td>
<td>1192.0</td>
<td>1189.8</td>
<td>1197.4</td>
<td>1168.3</td>
<td>1171.4</td>
<td>1096.4</td>
<td>1209.4</td>
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<tr>
<td>dry weight after test</td>
<td>(g)</td>
<td>1120</td>
<td>1094.1</td>
<td>1099.5</td>
<td>1091.8</td>
<td>1091.5</td>
<td>1066.6</td>
<td>1070.2</td>
<td>1006.7</td>
<td>1109.4</td>
</tr>
<tr>
<td>water content before test</td>
<td>(%)</td>
<td>6.44</td>
<td>7.39</td>
<td>6.21</td>
<td>6.07</td>
<td>7.46</td>
<td>7.02</td>
<td>6.52</td>
<td>7.51</td>
<td>7.45</td>
</tr>
<tr>
<td>water content after test</td>
<td>(%)</td>
<td>8.13</td>
<td>9.15</td>
<td>8.41</td>
<td>8.98</td>
<td>9.70</td>
<td>9.53</td>
<td>9.46</td>
<td>8.91</td>
<td>9.01</td>
</tr>
<tr>
<td>dry density</td>
<td>(g/cm³)</td>
<td>2.27</td>
<td>2.26</td>
<td>2.27</td>
<td>2.28</td>
<td>2.26</td>
<td>2.26</td>
<td>2.26</td>
<td>2.26</td>
<td>2.27</td>
</tr>
<tr>
<td>mean porosity</td>
<td>(%)</td>
<td>16.81</td>
<td>17.37</td>
<td>16.74</td>
<td>16.38</td>
<td>17.43</td>
<td>17.45</td>
<td>17.90</td>
<td>17.22</td>
<td>17.07</td>
</tr>
<tr>
<td>mean saturation before test</td>
<td>(%)</td>
<td>87.0</td>
<td>96.1</td>
<td>84.4</td>
<td>84.7</td>
<td>96.5</td>
<td>90.8</td>
<td>81.7</td>
<td>98.6</td>
<td>98.9</td>
</tr>
<tr>
<td>back volume change during saturation</td>
<td>(ml)</td>
<td>10.58</td>
<td>2.86</td>
<td>5.47</td>
<td>7.86</td>
<td>8.96</td>
<td>11.30</td>
<td>6.97</td>
<td>8.02</td>
<td>5.17</td>
</tr>
<tr>
<td>back volume change during consolidation</td>
<td>(ml)</td>
<td>-4.87</td>
<td>-4.04</td>
<td>-0.32</td>
<td>-10.78</td>
<td>-4.39</td>
<td>4.05</td>
<td>-1.12</td>
<td>-1.23</td>
<td>-2.29</td>
</tr>
<tr>
<td>water content after saturation</td>
<td>(%)</td>
<td>7.38</td>
<td>7.66</td>
<td>6.71</td>
<td>6.79</td>
<td>8.28</td>
<td>8.08</td>
<td>7.17</td>
<td>8.31</td>
<td>7.92</td>
</tr>
<tr>
<td>water content after consolidation</td>
<td>(%)</td>
<td>6.95</td>
<td>7.29</td>
<td>6.68</td>
<td>5.81</td>
<td>7.88</td>
<td>8.46</td>
<td>7.07</td>
<td>8.18</td>
<td>7.71</td>
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</table>
Table C.2.2: Water content, porosity, and saturation for the individual specimens during different phases of the CU and CD tests (continued). Specimens in grey were excluded for the analyses because complete consolidation could not have been guaranteed. Specimens in brown revealed a very low water content after test, probably related to a reduced dry weight due to pieces that broke off during specimen’s handling after the test. Specimens indicated with * were dried to constant weight.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>weight before test (g)</th>
<th>weight after test (g)</th>
<th>dry weight after test (g)</th>
<th>water content before test (%)</th>
<th>water content after test (%)</th>
<th>dry density (g/cm³)</th>
<th>mean porosity (%)</th>
<th>mean saturation before test (%)</th>
<th>back volume change during saturation (ml)</th>
<th>back volume change during consolidation (ml)</th>
<th>water content after saturation (%)</th>
<th>water content after consolidation (%)</th>
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</thead>
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<tr>
<td>ETH10_2*</td>
<td>1170.4</td>
<td>1179.5</td>
<td>1089.8</td>
<td>7.40</td>
<td>8.23</td>
<td>2.27</td>
<td>98.9</td>
<td>98.9</td>
<td>4.94</td>
<td>-0.60</td>
<td>7.85</td>
<td>7.79</td>
</tr>
<tr>
<td>ETH11</td>
<td>1175.8</td>
<td>1201.9</td>
<td>1107.3</td>
<td>6.19</td>
<td>8.54</td>
<td>2.29</td>
<td>16.16</td>
<td>98.9</td>
<td>5.84</td>
<td>-1.12</td>
<td>6.71</td>
<td>6.61</td>
</tr>
<tr>
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<td>1112.5</td>
<td>1089.4</td>
<td>6.28</td>
<td>8.55</td>
<td>2.29</td>
<td>16.15</td>
<td>98.9</td>
<td>6.63</td>
<td>-2.33</td>
<td>6.89</td>
<td>6.67</td>
</tr>
<tr>
<td>ETH13</td>
<td>1152.0</td>
<td>1117.9</td>
<td>1082.9</td>
<td>6.38</td>
<td>8.77</td>
<td>2.28</td>
<td>16.52</td>
<td>98.9</td>
<td>6.60</td>
<td>-4.16</td>
<td>6.99</td>
<td>6.61</td>
</tr>
<tr>
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<td>1118.1</td>
<td>1085.7</td>
<td>7.01</td>
<td>8.80</td>
<td>2.26</td>
<td>17.14</td>
<td>98.9</td>
<td>10.32</td>
<td>-6.97</td>
<td>7.96</td>
<td>7.32</td>
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<tr>
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<td>1117.4</td>
<td>1084.1</td>
<td>6.91</td>
<td>8.33</td>
<td>2.27</td>
<td>17.03</td>
<td>98.9</td>
<td>7.57</td>
<td>-8.00</td>
<td>7.61</td>
<td>6.87</td>
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<tr>
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<td>1097.7</td>
<td>7.09</td>
<td>8.94</td>
<td>9.45</td>
<td>93.5</td>
<td>98.9</td>
<td>0.11</td>
<td>-9.66</td>
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<td>7.90</td>
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<td>1083.6</td>
<td>7.00</td>
<td>9.45</td>
<td>3.36</td>
<td>94.3</td>
<td>98.9</td>
<td>11.67</td>
<td>-3.03</td>
<td>9.08</td>
<td>6.85</td>
</tr>
<tr>
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<td>1099.8</td>
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<td>3.63</td>
<td>2.27</td>
<td>16.81</td>
<td>98.9</td>
<td>13.93</td>
<td>-9.05</td>
<td>8.69</td>
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<td>1092.5</td>
<td>6.88</td>
<td>8.08</td>
<td>2.26</td>
<td>17.18</td>
<td>98.9</td>
<td>3.19</td>
<td>1.59</td>
<td>90.7</td>
<td>90.7</td>
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</table>
Table C.2.3: Water content, porosity, and saturation for the individual specimens during different phases of the CU and CD tests (continued). Specimens in grey were excluded for the analyses because complete consolidation could not have been guaranteed. Specimens in brown revealed a very low water content after test, probably related to a reduced dry weight due to pieces that broke off during specimen’s handling after the test. Specimens indicated with * were dried to constant weight.

<table>
<thead>
<tr>
<th></th>
<th>ETH19</th>
<th>ETH20</th>
<th>ETH20_2*</th>
<th>ETH21*</th>
<th>ETH22*</th>
<th>ETH23*</th>
<th>ETH24*</th>
</tr>
</thead>
<tbody>
<tr>
<td>weight before test (g)</td>
<td>1162.2</td>
<td>1160.1</td>
<td>1158.0</td>
<td>1162.8</td>
<td>1162.3</td>
<td>1166.98</td>
<td>1156.3</td>
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<tr>
<td>weight after test (g)</td>
<td>1121.0</td>
<td>1103.1</td>
<td>1176.4</td>
<td>1183.1</td>
<td>1177.3</td>
<td>1177.6</td>
<td>1161.9</td>
</tr>
<tr>
<td>dry weight after test</td>
<td></td>
<td></td>
<td>1095.6</td>
<td>1077.0</td>
<td>1080.5</td>
<td>1085.8</td>
<td>1086.5</td>
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<tr>
<td>water content before test (%)</td>
<td>6.08</td>
<td>7.72</td>
<td>7.17</td>
<td>7.09</td>
<td>6.98</td>
<td>6.78</td>
<td>7.04</td>
</tr>
<tr>
<td>water content after test (%)</td>
<td>2.32</td>
<td>2.42</td>
<td>8.88</td>
<td>8.96</td>
<td>8.36</td>
<td>7.76</td>
<td>7.55</td>
</tr>
<tr>
<td>dry density (g/cm³)</td>
<td>2.28</td>
<td>2.26</td>
<td>2.26</td>
<td>2.26</td>
<td>2.29</td>
<td>2.29</td>
<td>2.26</td>
</tr>
<tr>
<td>mean porosity (%)</td>
<td>16.61</td>
<td>17.42</td>
<td>17.45</td>
<td>17.25</td>
<td>17.13</td>
<td>16.33</td>
<td>17.43</td>
</tr>
<tr>
<td>mean saturation before test (%)</td>
<td>83.3</td>
<td>99.9</td>
<td>92.7</td>
<td>92.9</td>
<td>92.2</td>
<td>94.9</td>
<td>91.1</td>
</tr>
<tr>
<td>back volume change during saturation (ml)</td>
<td>14.95</td>
<td>5.77</td>
<td>4.94</td>
<td>8.26</td>
<td>4.43</td>
<td>7.02</td>
<td>9.66</td>
</tr>
<tr>
<td>back volume change during consolidation (ml)</td>
<td>-3.90</td>
<td>-5.69</td>
<td>-0.60</td>
<td>-0.62</td>
<td>-13.13</td>
<td>-15.87</td>
<td>-11.63</td>
</tr>
<tr>
<td>water content after saturation (%)</td>
<td>7.44</td>
<td>8.25</td>
<td>7.63</td>
<td>7.85</td>
<td>7.38</td>
<td>7.42</td>
<td>7.93</td>
</tr>
<tr>
<td>water content after consolidation (%)</td>
<td>7.09</td>
<td>7.72</td>
<td>7.57</td>
<td>7.80</td>
<td>7.16</td>
<td>5.97</td>
<td>6.85</td>
</tr>
</tbody>
</table>
C.3 Flushing phase

The maximum back pressure and confinement that were applied during the flushing phase are given in Table C.3.1. Furthermore, the strains at the end of the flushing phase and the duration of the flushing phase are indicated. For further details the reader is referred to the summary reports Barla (2016a) and Barla (2016b).

Table C.3.1: Details of the flushing phase. Specimens in grey were excluded for the analyses because complete consolidation could not have been guaranteed.

<table>
<thead>
<tr>
<th>specimen no.</th>
<th>max back pressure (MPa)</th>
<th>max confinement (MPa)</th>
<th>local axial strain 1 (%)</th>
<th>local axial strain 2 (%)</th>
<th>radial strain (%)</th>
<th>duration (hours)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ETH01</td>
<td>0.132</td>
<td>0.372</td>
<td>0.000</td>
<td>-0.150</td>
<td>-0.271</td>
<td>72.5</td>
</tr>
<tr>
<td>ETH02</td>
<td>0.302</td>
<td>0.492</td>
<td>-0.462</td>
<td>-0.238</td>
<td>-1.398</td>
<td>71.6</td>
</tr>
<tr>
<td>ETH03</td>
<td>0.146</td>
<td>0.236</td>
<td>-0.402</td>
<td>-0.291</td>
<td>-0.138</td>
<td>120.3</td>
</tr>
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<td>0.420</td>
<td>0.045</td>
<td>-0.221</td>
<td>-1.872</td>
<td>72.3</td>
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<td>0.188</td>
<td>0.372</td>
<td>-0.353</td>
<td>-0.654</td>
<td>0.025</td>
<td>166.7</td>
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<td>ETH06</td>
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<td>0.191</td>
<td>-0.512</td>
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<td>-0.218</td>
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<td>0.236</td>
<td>-0.740</td>
<td>-0.867</td>
<td>-0.194</td>
<td>42.4</td>
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<td>0.172</td>
<td>-0.532</td>
<td>-0.560</td>
<td>-0.199</td>
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</tr>
<tr>
<td>ETH09</td>
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<td>0.172</td>
<td>-0.049</td>
<td>-0.320</td>
<td>-0.050</td>
<td>72.4</td>
</tr>
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<td>-0.381</td>
<td>-0.165</td>
<td>47.6</td>
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<td>0.237</td>
<td>-0.391</td>
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<td>-0.097</td>
<td>143.4</td>
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<td>0.062</td>
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<td>0.020</td>
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<td>-0.477</td>
<td>0.264</td>
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<td>-0.653</td>
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</tr>
<tr>
<td>ETH15_2</td>
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<td>-0.533</td>
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</tr>
<tr>
<td>ETH16</td>
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<td>0.220</td>
<td>-0.394</td>
<td>-0.415</td>
<td>-1.608</td>
<td>43.6</td>
</tr>
<tr>
<td>ETH17</td>
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<td>-0.333</td>
<td>-0.259</td>
<td>70.9</td>
</tr>
<tr>
<td>ETH18</td>
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<td>-0.220</td>
<td>0.013</td>
<td>-0.202</td>
<td>165.3</td>
</tr>
<tr>
<td>ETH19</td>
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<td>-0.396</td>
<td>-0.196</td>
<td>121.7</td>
</tr>
<tr>
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<td>-0.432</td>
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<td>-0.554</td>
<td>145.2</td>
</tr>
<tr>
<td>ETH20_2</td>
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<td>-0.772</td>
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<td>-1.037</td>
<td>99.1</td>
</tr>
<tr>
<td>ETH21</td>
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<td>-0.397</td>
<td>0.001</td>
<td>-0.090</td>
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</tr>
<tr>
<td>ETH22</td>
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<td>72.3</td>
</tr>
<tr>
<td>ETH23</td>
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<td>-0.473</td>
<td>-0.909</td>
<td>-0.492</td>
<td>70.5</td>
</tr>
<tr>
<td>ETH24</td>
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<td>0.228</td>
<td>-0.362</td>
<td>-0.327</td>
<td>-0.303</td>
<td>71.9</td>
</tr>
</tbody>
</table>
C.4 Saturation phase

The confinement, pore pressure, the resultant effective confinement, and the $B$-value for the last $B$-check are given in Table C.4.1. Furthermore, the axial and radial strains at the end of the saturation phase as well as the duration of the saturation phase are indicated. For further details the reader is referred to the summary reports Barla (2016a) and Barla (2016b).

Table C.4.1: Details of the saturation phase: confinement, pore pressure, and $B$-value refer to the values at the last $B$-check; for the axial and radial strains the values at the end of the saturation phase are given. Specimens in grey were excluded for the analyses because complete consolidation could not have been guaranteed.

<table>
<thead>
<tr>
<th>specimen no.</th>
<th>confinement (MPa)</th>
<th>pore pressure (MPa)</th>
<th>effective confinement (MPa)</th>
<th>$B$-value local axial strain 1 (%)</th>
<th>$B$-value local axial strain 2 (%)</th>
<th>$B$-value radial strain (%)</th>
<th>duration (hours)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ETH01</td>
<td>1.988</td>
<td>1.671</td>
<td>0.317</td>
<td>0.81</td>
<td>0.011</td>
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<td>0.225</td>
</tr>
<tr>
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<td>1.271</td>
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<td>-0.016</td>
<td>-0.013</td>
</tr>
<tr>
<td>ETH03</td>
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<td>1.978</td>
<td>0.267</td>
<td>0.77</td>
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<td>-0.095</td>
<td>-0.091</td>
</tr>
<tr>
<td>ETH04</td>
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<td>1.821</td>
<td>0.214</td>
<td>0.89</td>
<td>-0.084</td>
<td>-0.128</td>
<td>-0.383</td>
</tr>
<tr>
<td>ETH05</td>
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<td>1.929</td>
<td>0.251</td>
<td>0.76</td>
<td>-0.140</td>
<td>0.076</td>
<td>-0.003</td>
</tr>
<tr>
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<td>0.929</td>
<td>0.059</td>
<td>0.92</td>
<td>0.018</td>
<td>-0.043</td>
<td>-0.103</td>
</tr>
<tr>
<td>ETH07</td>
<td>1.644</td>
<td>1.528</td>
<td>0.116</td>
<td>0.93</td>
<td>-0.179</td>
<td>-0.152</td>
<td>-0.224</td>
</tr>
<tr>
<td>ETH08</td>
<td>1.196</td>
<td>1.162</td>
<td>0.034</td>
<td>0.90</td>
<td>-0.113</td>
<td>-0.171</td>
<td>-0.268</td>
</tr>
<tr>
<td>ETH09</td>
<td>0.965</td>
<td>0.922</td>
<td>0.043</td>
<td>0.94</td>
<td>-0.034</td>
<td>0.030</td>
<td>-0.092</td>
</tr>
<tr>
<td>ETH10</td>
<td>1.779</td>
<td>1.696</td>
<td>0.083</td>
<td>0.92</td>
<td>0.259</td>
<td>-0.078</td>
<td>-0.212</td>
</tr>
<tr>
<td>ETH10_2</td>
<td>1.445</td>
<td>1.367</td>
<td>0.077</td>
<td>0.95</td>
<td>-0.094</td>
<td>-0.550</td>
<td>-0.087</td>
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<tr>
<td>ETH11</td>
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<td>0.172</td>
<td>0.84</td>
<td>-0.092</td>
<td>-0.043</td>
<td>0.000</td>
</tr>
<tr>
<td>ETH12</td>
<td>1.997</td>
<td>1.772</td>
<td>0.225</td>
<td>0.93</td>
<td>-0.046</td>
<td>-0.091</td>
<td>-0.296</td>
</tr>
<tr>
<td>ETH13</td>
<td>1.693</td>
<td>1.553</td>
<td>0.140</td>
<td>0.91</td>
<td>-0.128</td>
<td>-0.047</td>
<td>-0.151</td>
</tr>
<tr>
<td>ETH14</td>
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<td>1.690</td>
<td>0.204</td>
<td>0.91</td>
<td>-0.020</td>
<td>-0.148</td>
<td>-0.211</td>
</tr>
<tr>
<td>ETH15</td>
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<td>1.733</td>
<td>0.272</td>
<td>0.94</td>
<td>-0.035</td>
<td>-0.093</td>
<td>-0.206</td>
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<td>ETH15_2</td>
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<td>1.575</td>
<td>0.213</td>
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<td>-0.217</td>
<td>-0.091</td>
<td>-0.551</td>
</tr>
<tr>
<td>ETH16</td>
<td>1.220</td>
<td>1.088</td>
<td>0.132</td>
<td>0.92</td>
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<td>0.049</td>
<td>-0.390</td>
</tr>
<tr>
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<td>0.913</td>
<td>0.027</td>
<td>0.97</td>
<td>-0.038</td>
<td>-0.017</td>
<td>-0.231</td>
</tr>
<tr>
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<td>2.091</td>
<td>1.402</td>
<td>0.96</td>
<td>-0.011</td>
<td>-0.003</td>
<td>-0.239</td>
</tr>
<tr>
<td>ETH19</td>
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<td>1.390</td>
<td>0.632</td>
<td>0.81</td>
<td>-0.061</td>
<td>-0.060</td>
<td>-0.191</td>
</tr>
<tr>
<td>ETH20</td>
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<td>1.173</td>
<td>0.072</td>
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<td>0.010</td>
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<td>0.066</td>
<td>0.91</td>
<td>-0.045</td>
<td>-0.081</td>
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</table>
## C.5 Consolidation phase

Table C.5.1 shows the confinement, pore pressure, resultant mean effective consolidation stress, and the axial and radial strains at the end of the consolidation phase. Furthermore, the duration of the consolidation phase is given. For further details the reader is referred to the summary reports Barla (2016a) and Barla (2016b).

Table C.5.1: Details of the consolidation phase. Specimens in grey were excluded for the analyses because complete consolidation could not have been guaranteed.

<table>
<thead>
<tr>
<th>specimen no.</th>
<th>confinement stress (MPa)</th>
<th>pore pressure (MPa)</th>
<th>mean effective consolidation stress (MPa)</th>
<th>local axial strain 1 (%)</th>
<th>local axial strain 2 (%)</th>
<th>radial strain (%)</th>
<th>duration (hours)</th>
</tr>
</thead>
<tbody>
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<td>ETH01</td>
<td>4.579</td>
<td>4.605</td>
<td>2.072</td>
<td>2.525</td>
<td>0.416</td>
<td>0.515</td>
<td>0.145</td>
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<tr>
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<td>7.309</td>
<td>2.060</td>
<td>3.961</td>
<td>0.533</td>
<td>0.337</td>
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<td>3.715</td>
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<td>0.011</td>
<td>0.048</td>
</tr>
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<td>4.661</td>
<td>2.102</td>
<td>2.532</td>
<td>0.163</td>
<td>0.207</td>
<td>1.604</td>
</tr>
<tr>
<td>ETH05</td>
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<td>2.086</td>
<td>2.541</td>
<td>0.658</td>
<td>0.576</td>
<td>0.091</td>
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<td>-0.532</td>
<td>-0.774</td>
<td>-0.253</td>
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<tr>
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<td>2.305</td>
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<td>0.754</td>
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<td>-0.127</td>
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<td>3.101</td>
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<td>1.012</td>
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<td>-0.165</td>
<td>0.074</td>
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<td>6.108</td>
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<td>4.001</td>
<td>1.173</td>
<td>0.417</td>
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<td>6.100</td>
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<td>-0.076</td>
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<td>0.093</td>
<td>0.077</td>
<td>0.106</td>
</tr>
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<td>2.024</td>
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<tr>
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<td>0.025</td>
<td>0.079</td>
<td>0.414</td>
</tr>
<tr>
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</table>
C.6 Shearing phase

The confinement, pore pressure, and axial stress at the beginning of the shearing phase and at peak strength, respectively, are given in Table C.6.1 and Table C.6.2. Furthermore, the maximum pore pressure, the applied strain rate, and the axial and radial strains at peak strength are indicated. For further details the reader is referred to the summary reports Barla (2016a) and Barla (2016b).

<table>
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<tr>
<th>specimen no.</th>
<th>initial confinement (MPa)</th>
<th>initial pore pressure (MPa)</th>
<th>initial axial stress (MPa)</th>
<th>axial stress at peak (MPa)</th>
<th>confinement at peak (MPa)</th>
<th>pore pressure at peak (MPa)</th>
<th>max pore pressure (MPa)</th>
<th>strain rate (s⁻¹)</th>
<th>external axial strain (%)</th>
<th>local axial strain 1 (%)</th>
<th>local axial strain 2 (%)</th>
<th>radial strain (%)</th>
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</thead>
<tbody>
<tr>
<td>ETH01</td>
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<td>4.642</td>
<td>7.938</td>
<td>2.996</td>
<td>2.134</td>
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<td>0.987</td>
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<td>2.912</td>
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<td>4.822</td>
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<td>3.101</td>
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Table C.6.2: Details of the shearing phase (continued). Specimens in grey were excluded for the analyses because complete consolidation could not have been guaranteed. Specimen ETH15 was not sheared until failure.

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<th>specimen no.</th>
<th>initial confinement (MPa)</th>
<th>initial pore pressure (MPa)</th>
<th>initial axial stress (MPa)</th>
<th>axial stress at peak (MPa)</th>
<th>confinement at peak (MPa)</th>
<th>pore pressure at peak (MPa)</th>
<th>max pore pressure (MPa)</th>
<th>strain rate (s⁻¹)</th>
<th>external axial strain (%)</th>
<th>local axial strain 1 (%)</th>
<th>local axial strain 2 (%)</th>
<th>radial strain (%)</th>
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C.7 References


D Change in volumetric strain of a transversal isotropic media subjected to 2D-stress paths

The linear stress-strain relationship for a transversely isotropic material as shown in Figure D.1 is given by the following relation (see e.g. Goodman 1980)

\[
\begin{pmatrix}
\varepsilon_{xx} \\
\varepsilon_{yy} \\
\varepsilon_{zz}
\end{pmatrix} = \begin{pmatrix}
\frac{1}{E} - \frac{\nu}{E} & \frac{\nu'}{E'} & \frac{\nu'}{E'} \\
\frac{\nu}{E} & \frac{1}{E} - \frac{\nu'}{E'} & \frac{\nu'}{E'} \\
\frac{\nu'}{E'} & \frac{\nu'}{E'} & \frac{1}{E'}
\end{pmatrix}
\begin{pmatrix}
\sigma_{xx} \\
\sigma_{yy} \\
\sigma_{zz}
\end{pmatrix}, \quad (6.1)
\]

where \(\varepsilon_{xx}, \varepsilon_{yy}, \varepsilon_{zz}\) and \(\sigma_{xx}, \sigma_{yy}, \sigma_{zz}\) are the strains and stresses in x-, y-, and z-direction, respectively, \(E\) and \(\nu\) are the Young’s modulus and Poisson’s ratio for a P-specimen, respectively, \(E’\) and \(\nu’\) are the Young’s modulus and Poisson’s ratio for a S-specimen, respectively. The geomechanical convention is applied.

![Figure D.1: Transversely isotropic material with plane of isotropy parallel to the x-y plane.](image)

The volumetric strain \(\varepsilon_{vol} = \varepsilon_{xx} + \varepsilon_{yy} + \varepsilon_{zz}\) is given by

\[
\varepsilon_{vol} = \sigma_{xx} \left( \frac{1}{E} - \frac{\nu}{E} \right) + \sigma_{yy} \left( \frac{1}{E} - \frac{\nu}{E} \right) + \sigma_{zz} \left( \frac{1}{E'} - \frac{2\nu'}{E'} \right). \quad (6.2)
\]

Now let \(\varepsilon_{vol,S}^{(\alpha)}\) be the volumetric strain for a S-specimen that is subjected to a stress \(\sigma_{zz} = \sigma_{i}\) and \(\sigma_{xx} = \sigma_{yy} = -\alpha \sigma_{i}\), where \(\alpha\) is a positive parameter. Using equation (6.2)

\[
\varepsilon_{vol,S}^{(\alpha)} = \sigma_{i} \left( \frac{1}{E'}(1 - 2(1 - \alpha)\nu) - \frac{2\alpha}{E}(1 - \nu) \right). \quad (6.3)
\]

Similarly, the volumetric strain for a P-specimen \(\varepsilon_{vol,P}^{(\alpha)}\) subjected to a stress \(\sigma_{xx} = \sigma_{i}\) and \(\sigma_{yy} = \sigma_{zz} = -\alpha \sigma_{i}\) is given by

\[
\varepsilon_{vol,P}^{(\alpha)} = \sigma_{i} \left( \frac{1}{E}((1 - \nu)(1 - \alpha)) - \frac{1}{E'}(\alpha(3\alpha - 1)\nu') \right). \quad (6.4)
\]

In particular, for a pure shear compression stress path \((\alpha = \frac{1}{2})\) we observe the following relation
For the anisotropic two dimensional stress paths considered in this study, we have

\[
\varepsilon^{(1)}_{\text{vol},S} = \sigma_1 \left( \frac{1}{E'} (1-\nu') - \frac{1}{E} (1-\nu) \right) = -2 \varepsilon^{(1)}_{\text{vol},P}. \tag{6.5}
\]

(6.5)

Assuming that \( E > E' > 0, 0 < \nu, \nu' < 1 \) and \( \sigma_1 > 0, \varepsilon^{(1)}_{\text{vol},S} > 0 \) is equivalent to \( \frac{E}{E'} > \frac{1-\nu}{1-\nu'} \). This is certainly true if \( \frac{1-\nu}{1-\nu'} < 1 \) or equivalently if \( \nu' < \nu \), which is given for Opalinus Clay (Amann et al. 2012, Wild et al. 2015). Therefore, the volumetric strain and thus the excess pore pressure for a S-specimen subjected to a pure shear compression stress path is positive (i.e. it compacts). Consequently, according to equation (6.5), the volumetric strain for a P-specimen is negative (i.e. it dilates). In absolute value, the volumetric strain of a P-specimen is half the volumetric strain of a S-specimen.

For a comparison between the volumetric strains of the isotropic and anisotropic 2D stress path, let us investigate the influence of \( \alpha \) on the volumetric strain. We notice that \( \varepsilon^{(a)}_{\text{vol},S} \) is linear in \( \alpha \) with

\[
\frac{\partial \varepsilon^{(a)}_{\text{vol},S}}{\partial \alpha} = 2 \sigma_1 \left( \frac{\nu'}{E'} - \frac{1-\nu}{E} \right). \tag{6.8}
\]

(6.8)

\( \frac{\partial \varepsilon^{(a)}_{\text{vol},S}}{\partial \alpha} \) is negative if \( \frac{E}{E'} < \frac{1-\nu}{\nu'} \). This is satisfied for Opalinus Clay since \( \frac{E}{E'} \) is roughly 2 (see Chapter 5), \( \nu=0.19 \), and \( \nu'=0.16 \) (Amann et al. 2012, Wild et al. 2015). Thus, \( \varepsilon^{(a)}_{\text{vol},S} \) is strictly decreasing in \( \alpha \) and the volumetric strain for a S-specimen subjected to the “2D-iso” stress path is smaller than for a S-specimen subjected to the anisotropic “2D-aniso” stress path in this study.

Similarly, for a P-specimen, \( \varepsilon^{(a)}_{\text{vol},P} \) is linear in \( \alpha \) with

\[
\frac{\partial \varepsilon^{(a)}_{\text{vol},P}}{\partial \alpha} = \sigma_1 \left( \frac{3\nu'-1}{E'} - \frac{1-\nu}{E} \right). \tag{6.9}
\]

(6.9)

\( \frac{\partial \varepsilon^{(a)}_{\text{vol},P}}{\partial \alpha} \) is negative if \( \frac{E}{E'} > \frac{\nu-1}{1-3\nu'} \) and \( 1-3\nu'>0 \), which is satisfied for Opalinus Clay. Thus, for Opalinus Clay, \( \varepsilon^{(a)}_{\text{vol},P} \) is strictly decreasing with \( \alpha \) and the volumetric strain for a P-specimen subjected to the “2D-iso” stress path is smaller than for a P-specimen subjected to the anisotropic “2D-aniso” stress path.
D.1 References

