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Assessment of Geomechanical Properties of Intact Opalinus Clay

Expertenbericht

im Rahmen der Beurteilung des Vorschlags von mindestens zwei geologischen Standortgebieten pro Lagertyp, Etappe 2, Sachplan geologische Tiefenlager

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Sachplan Geologische Tiefenlager, Etappe 2

Expert Report -Assessment of Geomechanical Properties of Intact Opalinus Clay

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Executive Summary

For analytical and numerical calculations for the engineering feasibility assessment of a deep geological repository for disposal of nuclear waste in the Opalinus Clay and its maximum depth below ground surface, NAGRA established geomechanical parameters that are based on a large number of laboratory test results. Results from uniaxial compression, triaxial compression and oedometer tests were used to quantify the effective strength properties, the undrained shear strength and both, drained and undrained elastic properties of intact Opalinus Clay. The authors of this report were commissioned by ENSI to review and judge these geomechanical properties in terms of completeness and reliability.

This review report addresses the conceptual constitutive framework for Opalinus Clay and the simplifications proposed by NAGRA, provides the geomechanical fundamentals that are needed to adequately judge the experiments on intact Opalinus Clay and their interpretation, and assesses in detail the various test series on intact Opalinus Clay utilized and interpreted by NAGRA.

Summary of NAGRA's approach

Based on laboratory experiments, borehole logging data and experience with other clay rocks NAGRA provides a description of fundamental constitutive aspects of the Opalinus Clay, which lead to a conceptual geomechanical framework that follows basic principles of critical state soil mechanics. This model shows how the elastic limits, expressed by the Hvorslev failure envelope, the tension cut-off, and the Roscoe yield surface, vary with changes in effective normal stress and void ratio. NAGRA states that the available analytical and numerical methods used for calculating the hydro-mechanical coupled response of Opalinus Clay do not offer constitutive relations that account for all behavioral aspects of Opalinus Clay. Therefore, simplifications such as to omit the Roscoe yield surface were introduced by NAGRA to derive a simplified constitutive framework.

A large series of uniaxial and triaxial compression tests were used by NAGRA to establish the effective strength properties of intact Opalinus Clay. The constitutive framework utilized by NAGRA suggests that geomechanical properties, such as the effective strength (i.e., the effective friction angle Φ ' and the effective cohesion c'), and elastic properties depend on the void ratio, which decreases with increasing depth and with increasing effective stress. This relation is not explicitly included in the simplified constitutive framework and thus two different parameter sets were established. One set is considered representative for a depth up to 400m below ground surface (called Opalinus Clay shallow). Another set is considered representative of a depth range between 400 and 900m (called Opalinus Clay deep). The effective matrix strength for the two depth ranges was derived from triaxial compression tests in which the bedding planes were either parallel (called P-sample) or normal (called S-sample) to the specimen's long axis. The effective bedding plane strength was derived from specimens where the bedding planes were either inclined 30° with respect to the specimen's long axis (called X-samples), or 45° (called Zsamples). The quality of these triaxial test results was assessed, classified and weighted by NAGRA based on the test protocols and the completeness of key parameters being monitored during testing. Except for one test series (Jahns 2013) an overall quality level and weighting factor was assigned to each test series. The weighted data points were further used to establish the effective friction and cohesion of Opalinus Clay (i.e. matrix and bedding planes) by linear-regression analysis through all data points in qp'-space.

Because of the uncertainties associated with consolidated undrained and consolidated drained tests an alternative interpretation based on total stress was performed by NAGRA assuming unconsolidated undrained testing conditions. Similar to the effective strength properties a large series of triaxial test results (including; artificially dried and wetted specimens, test results from Mont Terri, Schlattingen and Benken) were analyzed by NAGRA to establish the undrained shear strength, Su, for both matrix and

bedding planes. According to NAGRA's constitutive framework, a relationship that shows an increase in the undrained shear strength *Su* with decreasing water content was established by NAGRA and used to define a basis to estimate *Su*-values representative of the actual depth at the siting regions.

Drained and undrained elastic properties were determined by NAGRA, based on results of triaxial compression tests, oedometer tests and permeameter tests. Test results from Mont Terri, Benken and Schlattingen were compiled to constrain the elastic properties representative for Opalinus Clay shallow and deep.

Assessment of the NAGRA documentation by the reviewers

Constitutive framework

The constitutive framework described by NAGRA is in agreement with behavioral aspects that have also been reported in many other studies on clay shales (e.g., Aristorenas 1992). The model is well described and documented with literature and laboratory data. NAGRA introduces a series of simplification to the constitutive framework to account for limitations in the numerical codes used for the engineering feasibility studies. One major simplification is to omit the Roscoe yield surface and to assume a linear-elastic behavior before reaching the Hvorslev yield surface or tension cut-off, for which the elastic properties for loading and reloading are exactly the same. As a consequence, the non-linearity of the stress-strain behavior in the pre-failure region, observed on tested samples, is not explicitly included in the simplified model. Therefore, numerical and analytical calculations, which utilize elastic properties derived from unloading/reloading cycles, may lead to a relevant underestimation of pre-peak deformation. The simplification introduced by NAGRA is reasonable for engineering feasibility studies provided that the consequences of omitting the Roscoe yield surface are considered with adequate elastic properties. For quantitative engineering design calculations more advanced constitutive models are required.

Bedding plane strength

For determining the effective strength properties of the bedding planes NAGRA mainly utilized results from triaxial compression tests on Z-samples and X-samples. Assuming a Mohr-Coulomb failure criterion the triaxial strength is minimal for an angle of $45^\circ - \phi'/2$ between the axial loading direction and the bedding plane orientation (where ϕ' is the effective friction angle of the bedding). Specimens tested in Z-orientation provide a strength that is affected by bedding planes but overestimate the bedding plane strength. Triaxial tests using X-samples may also provide strength information that is affected by bedding planes (unless the effective friction angle is 30°) and may also overestimate the bedding plane strength.

Triaxial Compression Tests

Six assessment criteria were used by the reviewers to adequately judge the results of triaxial compression tests and their interpretation. These six criteria are related to three testing phases (a saturation phase, consolidation phase and a shearing phase) which are required to reliably constrain effective strength properties of low permeable rock types from consolidated undrained (CU) or consolidated drained (CD) tests. Reliable effective strength properties can only be established when the specimens are fully saturated, the consolidation phase is completed, and the loading rate is slow enough to capture the pore pressure change representative for the bulk specimen during undrained loading, or to avoid excess pore pressure during drained loading. For the assessment of the saturation phase the minimum backpressure required to saturate the specimen and the quantity and evolution of Skempton's *B* coefficient (i.e., the relation between pore pressure changes to isotropic stress changes under undrained conditions) in subsequent loading stages were utilized. The completeness of the consolidation phase was assessed using the theoretical time required to consolidate a specimen and the time-dependent development of the volumetric strain and changes in water content. The shearing phase was assessed based on the theoretical

time required to reach the peak strength for undrained or drained loading and, in cases of undrained tests, the quantity of Skempton's Ā coefficient that relates pore pressure changes to differential stress changes.

Two test series (Jahns 2013 and Rummel & Weber 1999) utilize cores obtained from the boreholes in Benken and Schlattingen. The reviewers consider these test series most relevant for characterizing the strength of Opalinus Clay deep at the actual siting regions. The reporting of the assessment of these tests is therefore done at a greater level of detail compared to test series that utilize samples taken from the Mont Terri Underground Research Laboratory. Even though, all above assessment criteria have been applied equally to all test series.

The CU tests reported in Jahns (2013) follow a consistent testing procedure, which is well described, documented and carefully applied. A clear separation between saturation and consolidation phase is missing, which makes the assessment of this two testing phases based on measured data (i.e., B-values measured during the saturation phase and the development of volumetric strain and changes in water content during consolidation) difficult. The application of the six criteria to assess the completeness and correctness of all testing phases shows that 2 tests satisfy all criteria. The remaining 22 tests do not satisfy all criteria. For 6 tests, full saturation was established, but the consolidation phase was incomplete or/and the loading rate was too fast. In this case, the measured pore pressure at the end-faces of the specimen is smaller than the actual pore pressure within the specimen and both the effective axial and radial stresses are overestimated. As a consequence, the strength is underestimated. For 7 tests full saturation could not be established. For unsaturated conditions capillary suction arises in the specimen and therefore the pore pressure within the specimen is smaller than the measured pore pressure at the specimen's end-faces. In this case, the effective axial and radial stress are underestimated. As a consequence the strength is overestimated. For 9 tests the saturation state of the specimens could not be assessed. In total 2 test results can be used for establishing the effective strength properties of Opalinus Clay deep and 8 test results for the undrained shear strength.

The CU tests reported in Rummel & Weber (1999) are incompletely documented, and the testing procedure does not satisfy state-of-the-art testing procedures for determining effective strength properties. Full saturation could most probably not be established (i.e., very low backpressures) and was not demonstrated by routinely determined *B*-values. As a consequence, no test result is suitable for determining the effective strength properties of Opalinus Clay. Since the specimens were most probably unsaturated, the derived effective strength properties overestimate the actual strength.

For a depth up to 400m (Opalinus Clay shallow), the assessment of all triaxial tests considered by NAGRA showed that no test result satisfied the criteria in this review report for a valid test, which is a prerequisite for establishing the effective strength properties for Opalinus Clay shallow. Two test series were performed without pore pressure control. For the remaining test series (CU and CD tests) saturation could not be established and demonstrated. The specimens for determining the effective strength properties of Opalinus Clay shallow were most probably partially saturated and the derived effective strength properties overestimate the actual strength.

For both depth ranges effective strength properties were derived by NAGRA through a weighted regression analysis. Four quality levels (A, B, C, D) with corresponding weighting factors (100%, 75%, 50%, 25%) were used by NAGRA. For the test series of Jahns (2013) quality levels were assigned on an individual basis following the suggestions given in Favero et al. (2013). The assessment of Favero et al. (2013) is basically in agreement with the quality assessment in this report. The primary difference is a different assessment of the loading rate, which was, according to this report, for most of the tests probably too fast to obtain reliable pore pressures at failure. For all other triaxial test series a global quality level and weighting factor was assigned by NAGRA.

The quality levels and weighting factors assigned by NAGRA largely contradict the assessment in this report and are inconsistently used. For tests series without any pore water control, for example, a weighting of 25% was assigned by NAGRA even though the pore pressure at failure is unknown. This procedure of quantifying uncertainties by introducing weighting factors is not reproducible. A large amount of inadequate test results overbalance the few adequate test results, which has a significant impact on the results of the regression analysis through the weighted data points. This can lead to wrong conclusions. In total, the effective strength properties established by NAGRA through weighted linear regression analysis tend to overestimate the actual strength for both Opalinus Clay deep and shallow. The magnitude of the overestimation cannot be quantified. For quantifying the effective strength of Opalinus Clay only two reliable triaxial tests exist. However, eight test results can be used for constraining the undrained shear strength.

Undrained shear strength

Based on fundamental geomechanical considerations the alternative interpretation of triaxial data assuming the concept of " Φ =0" with the undrained shear strength *Su* is only applicable when the specimens are fully saturated and Skempton's B coefficient is unity. The data set used by NAGRA contains data stemming from triaxial tests on samples, which were either dried/wetted before testing, conducted at a water content after sample dismantling from storage, or conducted at an elevated water content due to a partial or full saturation with backpressure. Using this data set for establishing the undrained shear strength is not appropriate because of the following reasons: 1) For determining the undrained shear strength the pore space needs to be saturated with pore water and results from dried specimens cannot be used for establishing a relation between the undrained shear strength and the water content representative of the in-situ conditions; 2) For the majority of triaxial specimens a saturated state could most probably not be re-established and/or demonstrated. Suction must be anticipated, which leads to an overestimation of the undrained shear strength. In total only 8 tests were identified by the reviewers in the data set used by NAGRA, which have a sufficient quality to constrain the undrained shear strength of intact Opalinus Clay.

Two consistency tests related to the undrained shear strength were performed by the reviewers. In a first test, the *Su*-values suggested by NAGRA for intact Opalinus Clay were compared to *Su* values calculated from the effective strength properties suggested by NAGRA (based on the assumption that Skempton's *B* is unity and therefore the volume of the rock remains constant under undrained shearing). The comparison was done for an effective stress state and water contents suggested by NAGRA representative for a depth of 500m and 900m. The comparison reveals major inconsistencies. Calculated *Su* values for the matrix and the bedding planes are considerably lower than *Su*-values derived by NAGRA from triaxial test results. In a second test, the *Su*-values suggested by NAGRA were compared to valid data points from the literature and the 8 valid data points identified in the data set used by NAGRA. This allow one to establish a relation between the undrained shear strength and the effective confining stress. It was assumed that the undrained shear strength increases linearly with increasing effective confining stress. The resulting *Su*-values deviate largely from the *Su*-values suggested by NAGRA.

Determination of the E-Modulus

For Opalinus Clay deep only 8 tests were conducted on specimens, which were most probably saturated. These 8 tests can be used to define reliable values for the undrained E-Modulus *Eu* for P-, S- and X-samples for Opalinus Clay deep. *Eu*-values suggested by NAGRA were derived from unloading/reloading cycles and are in agreement with the reliable test results in case of S-samples, and slightly larger in case of P-Samples. For Opalinus Clay shallow none of the triaxial tests analyzed by NAGRA allows one to define reliable *Eu*-values since the specimens were most probably not saturated or saturation could not be demonstrated. The drained E-Modulus *E* was derived by NAGRA from oedometer tests on S-samples.

The *E*-value suggested by NAGRA for Opalinus Clay shallow is at the upper limit of experimental data in the relevant effective stress range. For Opalinus Clay deep the *E*-value suggested by NAGRA is in reasonable agreement with the experimental data in the relevant effective stress range. However, for the depth range between 400 and 900m (Opalinus Clay deep) the data suggest a major increase of the E-Modulus with increasing effective confinement (i.e. from 2.4 GPa to 8.0 GPa). This may have a relevant effect on numerical and analytical calculations which address the maximum depth below ground surface and needs to be considered.

Major Conclusions

The major conclusions of the review of the geomechanical properties of intact Opalinus Clay relevant for engineering feasibility analysis and for determining the maximum depth below ground surface are:

- The constitutive framework developed by NAGRA is in agreement with the literature and experimental studies. The simplification introduced by NAGRA is reasonable for engineering feasibility studies provided that the consequences of omitting the Roscoe yield surface are considered for the choice of adequate elastic properties. For quantitative engineering design calculations more advanced constitutive models are required.
- For establishing the bedding plane strength results from Z- and X-samples needs to be distinguished. Assuming a Mohr-Coulomb failure criterion, X-samples may only provide a reasonable estimate of the bedding plane strength if the effective friction angle is 30°, but triaxial test results from Z-samples overestimate the bedding plane strength because the bedding plane orientation was not taken into consideration by NAGRA for the analysis of the effective strength parameters.
- The majority of the triaxial tests used by NAGRA to establish effective strength properties do not fulfil all requirements of a successful testing procedure. A detailed assessment revealed that only 2 tests fulfill the requirements (full saturation, completed consolidation, adequate loading rate) and can be used for establishing effective strength properties for Opalinus Clay deep. For Opalinus Clay shallow none of the tests fulfill these requirements. For constraining reliable effective strength properties further tests following a state-of-the-art testing and quality assessment procedure need to be conducted.
- All test results were classified and weighted by NAGRA. The classification system (quality levels) used for the individual test series is not consistent.
- Quality levels are associated with weighting factors. The effective friction and the effective cohesion were derived through a regression analysis through the weighted data points. The weighting factors suggested by NAGRA largely contradict the quality assessment in this report, and the finding that the majority of specimens were most probably unsaturated. The procedure of quantifying uncertainties is not reproducible because a large amount of inadequate test results overbalances the few adequate test results. This can lead to wrong conclusions.
- The quality assessment of the reviewers suggests that the effective strength properties suggested by NAGRA tend to overestimate the actual strength. The degree of overestimation cannot be quantified.
- The data used for establishing the undrained strength Su is largely not appropriate (e.g., partially saturated/dried specimens). Only 8 test results can be used for establishing the undrained shear strength.
- The suggested undrained shear strength overestimates the strength and is inconsistent with *Su* values calculated from the effective strength properties suggested by NAGRA (for the condition of zero volumetric strain) and also inconsistent with literature values.

• The suggested undrained E-Moduli are in case of S-samples in agreement with the data in the relevant effective confining stress ranges. In case of P-samples the value suggested by NAGRA is at the upper limit of experimental data in the relevant effective stress range. For Opalinus Clay shallow none of the test results analyzed by NAGRA allows one to define reliable values for the undrained E-Modulus. The drained E-Modulus for Opalinus Clay shallow is at the upper limit of experimental data. The suggested drained E-Modulus for Opalinus Clay deep is well within the experimental data. However, the drained E-Modulus increases substantially with increasing effective confining stress. This is in particular relevant for Opalinus Clay deep, for which a depth range between 400 and 900m is considered. In this depth range the experimental data suggest an increase by a factor of 3.3 for the undrained E-Modulus (i.e. from 2.4 to 8.0 GPa). This may have a relevant effect on numerical and analytical calculations which address the maximum depth below ground surface and needs to be considered. In addition, the simplifications introduced by NAGRA for the constitutive framework and their consequences for the choice of elastic properties were not considered by NAGRA.

Zusammenfassung

Für analytische und numerische Modellrechnungen zur bautechnischen Machbarkeit eines geologischen Tiefenlagers für radioaktive Abfälle im Opalinuston sowie für die Abschätzung der maximalen Tiefenlage hat die NAGRA einen geomechanischen Kennwertesatz erarbeitet, der auf einer Vielzahl von Laborresultaten beruht. Dabei wurden Resultate von einaxialen und triaxialen Druckversuchen als auch Oedometerversuchen verwendet, um die effektiven Festigkeitsparameter, die undrainierte Scherfestigkeit sowie die drainierten und undrainierten elastischen Eigenschaften des intakten Opalinuston zu bestimmen. Die Autoren dieses Berichts wurden vom ENSI beauftragt, diese geomechanischen Kennwerte im Hinblick auf ihre Vollständigkeit und Belastbarkeit zu prüfen und zu beurteilen.

Dieser Prüfbericht befasst sich mit dem konzeptionellen geomechanischen Modellansatz für den Opalinuston sowie den von der NAGRA eingeführten Vereinfachungen, gibt einen Überblick über die geomechanischen Grundlagen, die sowohl für die Beurteilung der Versuche an Opalinuston als auch für die Interpretation der Versuchsergebnisse notwendig sind. Er beurteilt die verschiedenen Versuchserein an Opalinuston und deren Interpretation durch die NAGRA.

Zusammenfassung der Vorgehensweise der NAGRA

Basierend auf Laborexperimenten, Bohrlochdaten und Erfahrungen mit anderen Tongesteinen hat die NAGRA eine Beschreibung fundamentaler Aspekte des Verhaltens von Opalinuston zusammengestellt und in einen konzeptionellen geomechanischen Stoffansatz überführt, welcher den Grundregeln der "critical-state" Bodenmechanik folgt. Dieser Stoffansatz zeigt, wie sich die Grenzbedingungen des elastischen Verhaltens, welche durch die Hvorslev Bruchgrenze, die Zugspannungsbegrenzung und die Roscoe Fliessgrenze ausgedrückt werden, mit den effektiven Normalspannungen und der Porenzahl verändern. Die NAGRA stellt fest, dass die für die Modellierung des hydraulisch-mechanisch gekoppelten Verhaltens des Opalinustons verfügbaren analytischen und numerischen Methoden keine Stoffansätze enthalten, die sämtliche Verhaltensaspekte abdecken können. Die NAGRA hat aus diesem Grund Vereinfachungen eingeführt, wie zum Beispiel das Weglassen der Roscoe Fliessgrenze, um einen vereinfachten Stoffansatz herzuleiten.

Für die Herleitung effektiver Festigkeitsparameter hat die NAGRA eine Vielzahl von einaxialen und triaxialen Druckversuchen an Opalinuston herangezogen. Gemäss dem konzeptionellen geomechanischen Stoffansatz der NAGRA besteht ein Zusammenhang zwischen der Porenzahl, welche mit zunehmender Tiefe und zunehmenden effektiven Normalspannungen abnimmt, und den geomechanischen Parametern wie der effektiven Festigkeit (effektiver Reibungswinkel, effektive Kohäsion) und den elastischen Eigenschaften. Dieser Zusammenhang wird im vereinfachten Stoffansatz nicht explizit berücksichtigt, was zur Herleitung von zwei unterschiedlichen Parametersätzen führt. Ein Parametersatz ist repräsentativ für eine Tiefenlage von weniger als 400m (Opalinuston untief), ein weiterer für eine Tiefenlage von 400 bis 900m (Opalinuston tief). Die effektive Festigkeit der Gesteinsmatrix für beide Tiefenlagen wurde anhand von triaxialen Druckversuchen an Prüfkörpern abgeleitet, bei denen die Schichtung parallel (P-Proben) oder senkrecht (S-Proben) zur Längsachse des Prüfkörpers lag. Die effektive Festigkeit entlang der Schichtung beruht auf Prüfkörpern, bei denen die Schichtung 30° (X-Proben) oder 45° (Z-Proben) geneigt zur Längsachse des Prüfkörpers lag. Die Beurteilung der Qualität der triaxialen Druckversuche und deren Gewichtung durch die NAGRA beruht auf den Testprotokollen sowie auf Schlüsselparametern, die während eines Versuchs erfasst wurden. Mit Ausnahme einer Versuchsserie (Jahns 2013) wurden den Versuchsserien globale Qualitätsstufen und Gewichtungen zugeordnet. Die so gewichteten Resultate wurden in einem nächsten Schritt verwendet, um den effektiven Reibungswinkel und die effektive Kohäsion des Opalinustons durch eine gewichtete, lineare Regression im q-p'-Raum zu bestimmen.

Aufgrund von Unsicherheiten bei der Durchführung von konsolidiert undrainierten und konsolidiert drainierten Versuchen hat die NAGRA einen alternativen Weg der Interpretation der Versuchsergebnisse eingeschlagen. Dabei wird von unkonsolidiert undrainierten Versuchsbedingungen ausgegangen, womit sich die Interpretation der Versuchsergebnisse auf totale Spannungen beschränkt. Genau wie bei den effektiven Festigkeiten wurden von der NAGRA die Resultate einer grossen Anzahl an triaxialen Druckversuchen herangezogen, um die undrainierte Scherfestigkeit *Su* der Gesteinsmatrix und entlang der Schichtung zu bestimmen. Diese Versuchsdaten umfassen Resultate an künstlich getrockneten oder befeuchteten Proben, an Proben vom Felslabor Mont Terri sowie an Proben der Bohrungen Schlattingen und Benken. Gemäss dem konzeptionellen geomechanischen Stoffansatz der NAGRA wurde eine Beziehung für die Zunahme der undrainierten Scherfestigkeit *Su* mit abnehmendem Wassergehalt hergeleitet. Diese Beziehung erlaubt es der NAGRA, Werte für die undrainierte Scherfestigkeit zu bestimmen je nach dem jeweiligen Wassergehalt, der in den unterschiedlichen Standortgebieten und Tiefenlagen zu erwartet ist.

Die drainierten und undrainierten elastischen Eigenschaften wurden von der NAGRA auf Grundlage von triaxialen Druckversuchen, Oedometer- und Permeameterversuchen bestimmt. Dazu wurden Resultate an Proben aus dem Felslabor Mont Terri sowie den Bohrungen Schlattingen und Benken zusammengeführt, um die elastischen Eigenschaften sowohl für Opalinuston untief als auch Opalinuston tief zu bestimmen.

Beurteilung der Unterlagen der NAGRA durch die Experten

Stoffansatz für Opalinuston

Der von der NAGRA beschriebene Stoffansatz ist im Einklang mit vielen anderen Studien an Tonsteinen (z.B. Aristorenas 1992). Der Stoffansatz ist übersichtlich beschrieben und sowohl mit Literatur als auch Laborergebnissen dokumentiert. Um den Einschränkungen der numerischen Methoden, die für die Machbarkeitsstudie verwendet werden, gerecht zu werden, hat die NAGRA Vereinfachungen des Stoffansatzes eingeführten. Eine starke Vereinfachung ist dabei das Weglassen der Roscoe Fliessgrenze und damit das Einführen eines linear-elastischen Stoffverhaltens bevor im effektiven Spannungsraum die Hvorslev Bruchgrenze oder die Zugspannungsbegrenzung erreicht wird. Unter der Annahme eines linearelastischen Stoffverhaltens sind die elastischen Eigenschaften bei der Erst- und Wiederbelastung exakt gleich. Dies hat zur Folge, dass die bei triaxialen Druckversuchen an Opalinuston üblicherweise beobachtete Nichtlinearität der Spannungs-Dehnungs-Beziehung im Vorbruchbereich im vereinfachten Stoffansatz nicht explizit enthalten ist. Folglich unterschätzen numerische und analytische Modelle, welche elastische Eigenschaften aus Be-/Entlastungszyklen berücksichtigen, die im Vorbruchbereich auftretende Deformation. Die Vereinfachungen der NAGRA werden für Machbarkeitsbetrachtungen nur dann als zulässig erachtet, wenn die Konsequenzen des Weglassens der Roscoe Fliessgrenze durch geeignete Wahl der elastischen Eigenschaften berücksichtigt werden. Für eine quantitative konstruktive Bemessung sind weiterführende Stoffmodelle erforderlich.

Festigkeit entlang der Schichtung

Für die Herleitung der effektiven Festigkeitsparameter entlang der Schichtung verwendet die NAGRA Resultate von triaxialen Druckversuchen an Z- und X-Proben. Unter Annahme einer Mohr-Coulombschen Bruchbedingung wird die minimale Festigkeit einer Probe dann erreicht, wenn die Schichtung in einem Winkel von $45^{\circ} - \phi'/2$ gegenüber der axialen Belastungsrichtung geneigt ist (mit ϕ' als effektiver Reibungswinkel der Schichtung). Demzufolge repräsentieren Resultate von triaxialen Druckversuchen an Z-Proben lediglich eine von der Schichtung beeinflusste Festigkeit. Die tatsächliche Festigkeit entlang der Schichtung wird jedoch überschätzt. Triaxiale Druckversuche an X-Proben ergeben nur dann die minimale Festigkeit, wenn der effektive Reibungswinkel 30° beträgt. Ansonsten wird auch hier die Festigkeit überschätzt.

Triaxiale Druckversuche

Für die Beurteilung und Interpretation der triaxialen Druckversuche wurden von den Experten sechs Bewertungskriterien herangezogen. Diese beziehen sich auf drei Versuchsphasen (Sättigungsphase, Konsolidationsphase und Bruchphase), die erforderlich sind, um belastbare effektive Festigkeitsparameter aus konsolidiert undrainierten (KU) und konsolidert drainierten (KD) Versuchen an wenig durchlässigen Gesteinen zu ermitteln. Belastbare effektive Festigkeitsparameter können nur dann bestimmt werden, wenn die Prüfkörper voll gesättigt sind, die Konsolidation abgeschlossen ist und die Herbeiführung des Bruchs so langsam vollzogen wird, dass entweder, bei undrainierter Belastung, die am Probenrand gemessen Porendrücke dem Porendruck in der Probe entsprechen oder, bei drainierter Belastung, der Porendruck in der Probe annähernd konstant bleibt. Für die Sättigungsphase wurde der theoretisch zur Sättigung minimal erforderliche Porenwassergegendruck sowie der Betrag oder die Entwicklung des B-Wertes nach Skempton (d.h. das Verhältnis zwischen der Porendruckänderung aufgrund der Änderung der isotropen Spannung bei undrainierter Belastung) herangezogen. Die Konsolidationsphase wurde anhand der erforderlichen, theoretisch bestimmten Zeitdauer zur vollständige Konsolidation sowie der festgestellten volumetrischen Verzerrung bzw. der festgestellten Änderung des Wassergehalts beurteilt. Die Bruchphase wurde anhand der erforderlichen, theoretisch bestimmten Zeitdauer zur Verwirklichung des Bruchs sowie des Ā-Wertes nach Skempton (d.h. das Verhältnis zwischen der Porendruckänderung aufgrund einer Änderung der differentiellen Spannung bei undrainierter Belastung) beurteilt.

Bei zwei Versuchsserien (Jahns 2013 und Rummel & Weber 1999) wurden Proben aus den Bohrungen Schlattingen und Benken verwendet. Die Experten beurteilen diese Versuchsserien als massgebend für die Ermittlung der Festigkeit in Tiefen von 400 bis 900m (Opalinuston tief) und damit ausschlaggebend für die potentiellen Standortgebiete. Demzufolge ist der Detaillierungsgrad der Berichterstattung für diese Versuchsserien umfangreicher als für jene an Proben vom Felslabor Mont Terri (Opalinuston untief). Trotzdem wurden die Bewertungskriterien für alle Versuchsserien gleichermassen angewendet.

Die KU Versuche von Jahns (2013) basieren nach Ansicht der Experten auf einer gut beschriebenen, konsistent und sorgfältig angewendeten Versuchsdurchführung, und die Resultate sind übersichtlich dokumentiert. Allerdings fehlt eine klare Unterscheidung zwischen der Sättigungssphase und der Konsolidationsphase, wodurch eine Beurteilung deutlich erschwert wird. Dies betrifft die Ermittlung der B-Werte während der Sättigungsphase wie auch die Entwicklung der volumetrischen Verzerrung bzw. der Änderung des Wassergehalts während der Konsolidationsphase. Die Anwendung der sechs Beurteilungskriterien für die drei Versuchsphasen zeigt, dass nur 2 Versuche alle Kriterien erfüllen. Die übrigen 22 Versuche erfüllen nicht alle Kriterien. Bei 6 Versuchen konnte eine vollständige Sättigung sehr wahrscheinlich erreicht werden, jedoch war die Konsolidation unvollständig und/oder die Zeit zur Herbeiführung des Bruchs zu kurz. Demzufolge sind bei diesen Versuchen die an den Stirnseiten der Probe gemessenen Porendrücke kleiner als in der Probe selbst und die effektive Axial- und Radialspannung wird überschätzt. Folglich werden die Festigkeiten unterschätzt. Bei 7 Versuchen konnte keine vollständige Sättigung erreicht werden. Unter teilgesättigten Verhältnissen ist mit kapillaren Saugspannungen zu rechnen und der Porendruck in der Probe ist kleiner als die an den Stirnseiten der Probe gemessenen Porendrücke. In diesem Fall wird die Axial- und Radialspannung unterschätzt. Folglich werden die Festigkeiten überschätzt. Für weitere 9 Versuche sind keine eindeutigen Aussagen zur Probensättigung möglich. Insgesamt können 2 Versuchsresultate für die Ermittlung der effektiven Festigkeitsparameter und 8 Versuchsresultate hinsichtlich der undrainierten Scherfestigkeit für Opalinuston tief verwendet werden.

Die KU Versuche von Rummel & Weber (1999) sind unvollständig dokumentiert und der Versuchsablauf entspricht nicht dem Stand der Technik für die Ermittlung von effektiven Festigkeitsparametern. Eine vollständige Sättigung konnte höchstwahrscheinlich bei keinem der Versuche erreicht werden (da die Porenwassergegendrücke sehr klein waren) und es wurden keine B-Werte ermittelt, die eine vollständige Sättigung belegen könnten. Demzufolge kann kein Versuchsergebnis zur Bestimmung effektiver Festigkeitseigenschaften herangezogen werden. Da die Prüfkörper sehr wahrscheinlich nicht gesättigt waren, werden die Festigkeiten überschätzt.

In Tiefen von weniger als 400m (Opalinuston untief) ergab die Prüfung der von der NAGRA betrachteten triaxialen Druckversuche, dass kein Versuch die Beurteilungskriterien erfüllt. Dies ist allerdings die Grundvoraussetzung zur Bestimmung effektiver Festigkeitseigenschaften. 2 Versuchsserien wurden ganz ohne Porendruckkontrolle durchgeführt. Bei allen übrigen Versuchsserien (KU und KD Versuche) konnte keine vollständige Sättigung erreicht bzw. belegt werden. Demzufolge waren die Prüfkörper, die für Opalinuston untief herangezogen wurden, sehr wahrscheinlich teilgesättigt, wodurch die aus den Versuchen ermittelten effektiven Festigkeitsparameter die tatsächliche Festigkeit überschätzen.

Für beide Tiefenbereiche wurden die effektiven Festigkeitsparameter von der NAGRA durch gewichtete Regressionsanalysen bestimmt. Dazu wurden vier Qualitätsstufen (A, B, C, D) mit entsprechenden Gewichtungen (100%, 75%, 50%, 25%) eingeführt. Im Fall der Versuchsserie von Jahns (2013) wurde, entsprechend den Empfehlungen von Favero et al. (2013), jedem einzelnen Versuch eine Qualitätsstufe zugeordnet. Die Beurteilung von Favero et al. (2013) entspricht grundsätzlich jener im vorliegenden Bericht. Allerdings bestehen unterschiedliche Schlussfolgerungen bezüglich der Belastungsraten in der Bruchphase, welche nach Einschätzung der Experten für den Grossteil der Versuche zu schnell war. Für alle anderen Versuchsserien hat die NAGRA globale Qualitätstufen und Gewichtungen verwendet.

Die Qualitätsbeurteilung der Versuchsresultate durch die NAGRA sowie deren Gewichtung stehen im Widerspruch mit der Beurteilung der Experten. Die Qualitätsstufen und Gewichtungen wurden von der NAGRA auch nicht einheitlich angewendet. Beispielsweise wurden Resultate aus Versuchsserien ohne jegliche Porendruckkontrolle mit einem Gewichtungsfaktor von 25% in den Analysen berücksichtigt, obwohl der Porendruck beim Bruch unbekannt ist. Diese Vorgehensweise der Quantifizierung der Unsicherheiten durch die Verwendung von Gewichtungsfaktoren ist nicht nachvollziehbar. Eine grosse Anzahl qualitativ minderwertiger Versuche vermag dann eine kleine Anzahl hochwertiger Versuche zu kompensieren, womit das Ergebnis einer gewichteten Regressionsanalyse zu Ungunsten der belastbaren Versuchsresultate verzerrt wird. Dies kann zu falschen Schlussfolgerungen führen. Insgesamt überschätzen die durch gewichtete Regression ermittelten effektiven Festigkeitsparameter die tatsächliche Festigkeit sowohl für Opalinuston untief als auch für Opalinuston tief. Das Ausmass der Überschätzung ist nicht quantifizierbar. Für die Festlegung der effektiven Festigkeitsparameter stehen nur 2 belastbare Versuchsresultate zur Verfügung. Hingegen können 8 Versuchsresultate verwendet werden, um die undrainierte Scherfestigkeit festzulegen.

Undrainierte Scherfestigkeit

Basierend auf grundlegenden geomechanischen Überlegungen ist eine alternative Interpretation der triaxialen Druckversuche unter Annahme des " $\Phi=0$ " Konzeptes mit der undrainierten Scherfestigkeit *Su* nur dann zulässig, wenn die Prüfkörper vollständig gesättigt sind und sich ein B-Wert von 1.0 ergibt. Die von der NAGRA berücksichtigten Versuchsresultate stammen von Proben, die entweder künstlich getrocknet/befeuchtet wurden vor dem Versuch, die gemäss dem vorliegenden Wassergehalt nach ihrer Lagerung und Vorbereitung ohne weitere Behandlung oder gemäss einem erhöhten Wassergehalt bei teilweiser oder vollständiger Sättigung mittels eines Porenwassergegendrucks getestet wurden. Die Ermittlung der undrainierten Scherfestigkeit auf Grundlage dieses Datensatzes ist aus folgenden Gründen ungeeignet: 1) Die Versuche müssen an vollständig gesättigten Prüfkörpern durchgeführt werden, und getrocknete Prüfkörper dürfen nicht verwendet werden, um eine Beziehung zwischen der undrainierten Scherfestigkeit und dem für die in-situ Verhältnisse kennzeichnenden Wassergehalt zu begründen; 2) Für den Grossteil der Versuche wurde eine vollständige Sättigung der Prüfkörper höchstwahrscheinlich nicht

wiederhergestellt bzw. nicht belegt. Folglich sind kapillare Saugspannungen zu erwarten, die zu einer Überschätzung der undrainierten Scherfestigkeit führen. Insgesamt wurden von den Experten nur 8 Versuche identifiziert, welche zur Festlegung der undrainierten Scherfestigkeit verwendet werden dürfen.

Von den Experten wurden zwei Konsistenzprüfungen in Bezug auf die undrainierte Scherfestigkeit durchgeführt. In einer ersten Prüfung wurden die von der NAGRA vorgeschlagenen *Su*-Werte mit jenen Werten verglichen, die sich unter der Annahme der Volumenkonstanz bei undrainierter Belastung und eines B-Wertes von 1.0 für eine bestimmte effektive Einspannung aus den ebenfalls von der NAGRA vorgeschlagenen effektiven Festigkeitsparametern errechnen lassen. Dieser Vergleich wurde für effektive initiale Spannungen in einer Tiefe von 500m und 900m durchgeführt, bei denen gemäss NAGRA ein typischer Wassergehalt zu erwarten ist. Der Vergleich zeigt bedeutende Inkonsistenzen. Die berechneten *Su*-Werte für die Gesteinsmatrix wie auch für die Schichtung liegen deutlich unter den von der NAGRA aus triaxialen Druckversuchen abgeleiteten Werten. In einer zweiten Prüfung wurden die von der NAGRA vorgeschlagenen *Su*-Werte mit Werten aus der Literatur und den 8 erfolgreichen Versuchen von Jahns (2013) verglichen. Damit lässt sich eine Beziehung zwischen der undrainierten Scherfestigkeit und der effektiven Einspannung herstellen, wobei ein linearer Zusammenhang zwischen der undrainierten Scherfestigkeit und der effektiven Einspannung angenommen wurde. Die so ermittelten *Su*-Werte liegen wiederum deutlich unter den von der NAGRA vorgeschlagenen Werten.

Ermittlung des E-Moduls

Für Opalinuston tief liegen 8 triaxiale Druckversuche an gesättigten Prüfkörpern vor, die zur Ermittlung belastbarer Werte für den undrainierten E-Modul *Eu* für P-, S- and X-Proben herangezogen werden können. Die von der NAGRA vorgeschlagenen *Eu*-Werte stammen aus Be-/Entlastungszyklen und stimmen im Fall der S-Proben mit den zuverlässigen Versuchsresultaten gut überein bzw. sind im Fall der P-Proben etwas grösser. Für Opalinuston untief liegen keine triaxialen Druckversuche vor, die eine belastbare Ermittlung des undrainierten E-Moduls zulassen. Die Prüfkörper waren sehr wahrscheinlich teilgesättigt bzw. eine vollständige Sättigung konnte nicht nachgewiesen werden. Der drainierte E-Modul *E* wurde von der NAGRA anhand von Oedometerversuchen an S-Proben bestimmt. Für Opalinuston untief liegt der von der NAGRA vorgeschlagene *E*-Wert im relevanten effektiven Spannungsbereich an der oberen Grenze der experimentell ermittelten Resultate. Für Opalinuston tief liegt der von der NAGRA vorgeschlagene *E*-Wert im nelevanten effektiven Spannungsbereich an der oberen Grenze der experimentell ermittelten Resultate. Für Opalinuston tief liegt der von der NAGRA vorgeschlagene *E*-Wert im relevanten effektiven Spannungsbereich innerhalb der experimentell ermittelten Bandbreite der Resultate. Allerdings ist festzuhalten, dass der drainierte E-Modul im massgebenden Tiefenbereich zwischen 400 und 900m mit zunehmender effektiver Einspannung deutlich zunimmt (von 2.4 auf 8.0 GPa). Diese Zunahme könnte für die Betrachtung der maximalen Tiefenlage relevant sein und sollte bei analytischen und numerischen Modellrechnungen berücksichtigt werden.

Wichtige Schlussfolgerungen

Die wichtigsten Schlussfolgerungen der Überprüfung der geomechanischen Parameter des Opalinustons für Machbarkeitsstudien und Betrachtungen zur maximalen Tiefenlage sind:

- Der von der NAGRA entwickelte Stoffansatz findet Bestätigung sowohl in der Literatur als auch in den experimentellen Ergebnissen. Die von der NAGRA eingeführten Vereinfachungen sind für Machbarkeitsstudien zulässig, solange die Konsequenzen des Weglassens der Roscoe Fliessgrenze durch eine geeignete Festlegung der elastischen Eigenschaften berücksichtigt werden. Für eine quantitative konstruktive Bemessung sind weiterführende Stoffmodelle zu verwenden.
- Bei der Herleitung der Festigkeit entlang der Schichtung ist zwischen triaxialen Druckversuchen an Z- und X-Proben zu unterscheiden. Unter Annahme der Bruchbedingung nach Mohr-Coulomb könnten Versuchsresultate an X-Proben eine zuverlässige Einschätzung der Festigkeit entlang der Schichtung ergeben, sofern der effektive Reibungswinkel 30° beträgt. Die Versuchsresultate an Z-

Proben überschätzen jedoch die Festigkeit entlang der Schichtung, weil deren Orientierung durch die NAGRA bei der Auswertung der effektiven Festigkeitsparameter nicht berücksichtigt wurde.

- Die meisten triaxialen Druckversuche, die von der NAGRA zur Festlegung der effektiven Festigkeitsparameter verwendet wurden, erfüllen nicht die Anforderungen an eine belastbare Versuchsdurchführung nach Stand der Technik. Eine detaillierte Betrachtung ergab, dass nur 2 Versuche alle Kriterien erfüllen (vollständige Sättigung, abgeschlossene Konsolidation, genügend langsame Herbeiführung des Bruchs) und zur Festlegung der effektiven Festigkeitsparameter für Opalinuston tief herangezogen werden können. Kein einziger Versuch, der für Opalinuston untief herangezogen wurde, kann diese Kriterien erfüllen. Um zuverlässige effektive Festigkeiten zu bestimmen, sind weitere Versuche entsprechend dem Stand der Technik auszuführen.
- Alle Versuche wurden von der NAGRA einer Qualitätskontrolle unterzogen und den Resultaten wurden entsprechende Gewichtungen zugeordnet. Die Anwendung der Qualitätsstufen für die einzelnen Versuchsserien ist nicht konsistent.
- Jeder Qualitätsstufe ist ein Gewichtungsfaktor zugeordnet. Der effektive Reibungswinkel und die effektive Kohäsion wurden durch die NAGRA anhand einer Regressionsanalyse mit den gewichteten Versuchsresultaten festgelegt. Die von der NAGRA verwendeten Gewichtungen widersprechen erheblich der Bewertung durch die Experten und der Tatsache, dass der Grossteil der Versuche höchstwahrscheinlich an teilgesättigten Prüfkörpern durchgeführt wurde. Diese Art der Quantifizierung von Unsicherheiten ist nicht nachvollziehbar, weil dadurch eine grosse Anzahl qualitativ minderwertiger Versuche durch ihre Stapelung höheres Gewicht erlangt als eine kleine Anzahl qualitativ hochwertiger Versuche. Dies kann zu falschen Schlussfolgerungen führen.
- Eine qualitative Beurteilung der Experten ergab, dass die von der NAGRA vorgeschlagenen effektiven Festigkeitsparameter die tatsächliche Festigkeit tendenziell überschätzen. Das Ausmass dieser Überschätzung ist nicht quantifizierbar.
- Die Datenbasis für die Ermittlung der undrainierten Scherfestigkeit *Su* ist zum grossen Teil nicht dafür geeignet, da die Versuche an teilgesättigten oder sogar künstlich getrockneten Prüfkörpern durchgeführt wurden. Es können nur 8 Versuche hinsichtlich der undrainierten Scherfestigkeit verwendet werden.
- Die von der NAGRA vorgeschlagenen Werte der undrainierten Scherfestigkeit überschätzen die tatsächliche undrainierte Scherfestigkeit. Sie sind ausserdem nicht konsistent mit *Su*-Werten, die sich aus den effektiven Festigkeitsparametern unter Bedingung der Volumenkonstanz berechnen lassen, und Angaben aus der Literatur.
- Die von der NAGRA vorgeschlagenen Werte für den undrainierten E-Modul stimmen im Fall der S-Proben im relevanten effektiven Spannungsbereich mit den belastbaren Versuchsresultaten gut überein bzw. sind im Fall der P-Proben im relevanten effektiven Spannungsbereich an der oberen Grenze im Vergleich mit den belastbaren Versuchsresultaten. Für Opalinuston untief liegen keine Laborergebnisse vor, die eine zuverlässige Bestimmung des undrainierten E-Moduls zulassen. Der von der NAGRA vorgeschlagene drainierte E-Modul liegt für Opalinuston untief an der oberen Grenze bzw. für Opalinuston tief gut innerhalb der experimentell ermittelten Bandbreite. Allerdings nimmt der drainierte E-Modul im massgebenden Tiefenbereich zwischen 400 und 900m mit zunehmender effektiver Einspannung deutlich zu (von 2.4 auf 8.0 GPa). Diese Zunahme könnte für die Betrachtung der maximalen Tiefenlage relevant sein und sollte bei analytischen und numerischen Modellrechnungen Berücksichtigung finden. Es zeigt sich hier ausserdem, dass die NAGRA die von ihr eingeführten Vereinfachungen des Stoffansatzes und die Konsequenzen auf die Wahl der elastischen Eigenschaften nicht berücksichtigt hat.

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1 Introduction

1.1 Mandate

In 2008 the Federal Council approved the concept of "Sachplan geologische Tiefenlager (SGT)" that regulates the site selection process for a nuclear waste repository in three consecutive stages.

In the first stage the National Cooperative for the Disposal of Radioactive Waste (NAGRA) suggested in 2008 six potential sites for low- and intermediate-level radioactive waste (SMA), and three for high-level radioactive waste (HAA). The aim of the current, second stage, is to limit these potential sites to at least two sites per waste type for further in-depth investigations in stage three. The reduction to at least two sites per waste type is based on comparative safety assessments of the various sites with a primary focus on long-term safety. A site can only be eliminated if, compared to other sites, clear disadvantages in safety exist.

In stage 2 of the sectorial plan NAGRA suggested in 2015 two potential sites, which are suitable for both SMA and HAA repositories. The Swiss Federal Nuclear Safety Inspectorate (ENSI) is currently reviewing the provided documents and the suggestions for potential sites that will be investigated in detail in stage 3.

Dr. Florian Amann and Dr. Martin Vogelhuber were commissioned by ENSI to review geomechanical properties of intact Opalinus Clay. This review includes an assessment of the adequacy of the laboratory tests commissioned by Nagra (i.e. triaxial compression tests and oedometer tests) and the derived strength and stiffness of the tested rock (effective strength properties, undrained shear strength, elastic properties). According to Nagra (2014b) these geomechanical properties are directly relevant for the assessment of the host rock and the repository perimeter (Indicator 47) and, in addition, indirectly relevant for Indicator 1 (the depth below surface in terms of technical feasibility) and Indicator 29 (the excavation damage zone in the near-field of underground excavations). The strength and stiffness of the tested rock affect three out of four criteria which are considered relevant for site selection decisions by ENSI (2013) as they have either direct or indirect relevance 1) for the effectiveness of the geological barrier, 2) for the long-term stability of the geological barrier, and 3) for the technical feasibility of the repository. In addition, this review report that addresses the intact rock properties of Opalinus Clay forms the basis for answering a series of key questions from ENSI associated with the constructability and long-term safety that will be addressed in a companion report (Amann et al. 2015).

1.2 Reviewed reports

The following reports have been reviewed in detail:

Favero, V., Ferrari, A., Laloui, L. (2013) Diagnostic analyses of the geomechanical data bases from the SLA-1 borehole. NAB 13-45

Giger, S., Marschall, P. (2014) Geomechanical properties, rock models and in-situ stress conditions for Opalinus Clay in Northern Switzerland. NAB 14-01

Jahns, E. (2007) RA experiment - Rock strength of Opalinus Clay subject to time of storage. TN 2007-30

Jahns, E. (2010) RA experiment - Opalinus Clay rock characterization. TN 2008-55rev

Jahns, E. (2013) Geomechanical laboratory tests on Opalinus Clay cores from the bore hole Schlattingen SLA-1. NAB 13-18

NAGRA (2014a) SGT Etappe 2, Vorschlag weiter zu untersuchender geologischer Standortgebiete mit zugehörigen Standortarealen für die Oberflächenanlage - Geologische Grundlagen - Geomechanische Unterlagen. NTB 14-02, Dossier IV

Olalla, C., Martin, M.E., Saez J. (1999) ED-B experiment - Geotechnical laboratory tests on Opalinus Clay rock samples. TN 98-57

Popp, T., Salzer, K. (2006) HE-D experiment (influence of bedding planes) - Triaxial deformation tests in a multi-anvil apparatus with ultrasonic monitoring, sampling and rock preparation, adaptation of laboratory techniques. TN 2005-34

Rummel, F., Hettkamp, T., Weber, U. (1999) DM experiment - Laboratory experiments for the determination of deformation mechanisms and a constitutive law for time dependent deformation behavior of the Opalinus Clay. TN 99-35

Rummel, F., Weber, U. (1999) Sondierbohrung Benken - Felsmechanische Untersuchungen an Bohrkernen. Unpubl. NAGRA Int. Report. NAGRA, Wettingen

Rummel, F., Weber, U. (2004) RA experiment - Rock mechanical testing and characterization on drillcores of boreholes BRA-1 and BRA-2. TN 2004-38

Schnier, H., Stührenberg, D. (2007) LT experiment - Strength tests on cylindrical specimens, documentation and evaluation (Phases 8 & 9). TR 2003-04

The following reports have been considered for plausibility consideration, but were not reviewed in detail:

Bock, H. (2009) RA experiment - Updated review of rock mechanics properties of the Opalinus Clay of Mont Terri URL based on laboratory and field testing. TR 2008-04

Chiffoleau, S., Robinet, J.C. (1999) HE Experiment: determination of the hydro-mechanical characteristics of the Opalinus Clay. TN 98-36

Ferrari, A., Favero, V., Manca, D., Laloui L. (2012) Geotechnical characterization of core samples from the geothermal well Schlattingen SLA-1 by LMS/EPFL. NAB 12-50

Horseman, S.T., Harrington, J.F. (2002). Laboratory experiments on gas migration in Opalinus Clay samples from the Benken borehole, Switzerland. Unpubl. NAGRA Int. Report. NAGRA, Wettingen.

Horseman, S.T., Harrington, J.F., Birchall, D.J., Noy, D.J., Cuss, R.J. (2006) Hydrogeologic analyses and synthesis (HA experiment) - Consolidation and rebound properties of Opalinus Clay: a long-term, fully drained test. TN 2003-03rev

Klee, G., Rummel, F. (2000) Sondierbohrung Benken: Hydrofrac Spannnungsmessungen Teil II -Ergebnisse der Laboruntersuchungen und Abschlussbewertungen. Unpubl. NAGRA Int. Report. NAGRA, Wettingen.

Mathier, J.F., Egger, P., Descoeudres, F. (1999) Sondierbohrung Benken: Felsmechanische Untersuchungen an Bohrkernen (Teil 2). Unpubl. NAGRA Int. Report. NAGRA, Wettingen.

NAGRA (2014b) SGT Etappe 2, Vorschlag weiter zu untersuchender geologischer Standortgebiete mit zugehörigen Standortarealen für die Oberflächenanlage - Sicherheitstechnischer Bericht zu SGT Etappe 2 - Sicherheitstechnischer Vergleich und Vorschlag der in Etappe 3 weiter zu untersuchenden geologischen Standortgebiete. NTB 14-01

Péron, H., Salager, S., Eichenberger, J., Rizzi, M., Laloui, L. (2009) Gas path through host rock and along seal section (HG-A) experiment - Experimental and numerical analysis of excavation damaged zone (EDZ) along tunnels. TN 2008-54

2 Geomechanical properties suggested by NAGRA

NAGRA's objective to establish geomechanical properties is to provide input properties for analytical and numerical methods for the engineering feasibility assessment (NAGRA 2014a). These properties include intact rock properties, rock mass strength and stiffness, and magnitude and orientation of the insitu stress components¹. For the engineering feasibility assessment different analytical and numerical approaches were used (i.e., effective stress analysis, total stress analysis), which require specific input properties for both strength and stiffness. For the effective stress analysis (short-term or long-term response) effective strength properties and drained elastic properties need to be defined. For the total stress analysis (only short-term response) the undrained shear strength and undrained elastic properties need to be defined.

2.1 Conceptual geomechanical model and approach

Based on laboratory experiments, borehole logging data and experience with other clay rocks NAGRA provides a description of fundamental constitutive aspects of the Opalinus Clay that includes (NAGRA 2014a, Giger & Marschall 2014):

- Effective stress dependency of porosity, water content, density, hydraulic conductivity and elastic properties
- Irreversible compression in loading-unloading-cycles (for consolidation pressures beyond apparent over-consolidation pressures)
- Swelling pressure and heave as a consequence of water uptake
- Transversely isotropic elastic behavior
- Dilatant failure behavior
- Anisotropic compressive and tensile strength
- Post-failure stress drop
- Strong dependency of strength and stiffness on capillary forces

These behavioral aspects lead to a conceptual geomechanical framework for Opalinus Clay that follows basic principles of critical state soil mechanics (Figure 1a. NAGRA 2014a, Giger & Marschall 2014). This model shows how the elastic limits, expressed by the Hvorslev yield surface, the tension cut-off and the Roscoe yield surface, are varying with changes in differential stress (q), effective mean stress (p') and void ratio.

NAGRA states that the analytical and numerical methods calculating the hydro-mechanical coupled response of Opalinus Clay do not offer constitutive relations that account for all of the above described behavioral aspects. This is in particular true for the strength and stiffness of the tested rock, which tend to increase with increasing effective normal stress or decreasing porosity (i.e., increasing compression). In addition, the Roscoe yield surface is considered to be irrelevant for the engineering feasibility assessment. Owing to the irrelevant aspects of the conceptual geomechanical model, a simplified elastic-plastic model was established (Figure 1b. NAGRA 2014a, Giger & Marschall 2014), which overcomes limitations in the analytical and numerical methods. This model accounts for the relevant elastic limits (i.e. Mohr-Coulomb failure envelope corresponding to the Hvorslev yield surface and a tension cut-off). Since both, the strength and stiffness of Opalinus Clay tend to increase with increasing depth, two sets of material parameters have been established which are either representative for Opalinus Clay at a depth up to 400m (called "Opalinus Clay shallow") or representative for Opalinus Clay at a depth range between 400 and 900m (called "Opalinus Clay deep") below ground surface. The required elastic and effective strength

¹ In this report only properties of the intact Opalinus Clay (i.e. for rock mass model GM 1) are addressed.

properties for the two depth ranges have been derived from laboratory test results (uniaxial and triaxial compression tests, and oedometer tests).



Figure 1: a) Conceptual geomechanical framework; b) simplified model (NAGRA 2014a)

2.2 Effective strength properties

The effective matrix strength was derived by NAGRA from triaxial compression tests, for which the bedding planes where either parallel (called P-samples) or normal (called S-samples) to the specimen's long axis. For determining effective strength properties of the matrix NAGRA does not distinguish between P- and S-samples. The effective bedding plane strength was derived from specimens where the bedding planes were inclined either 45° (called Z-sample) or 30° (called X-sample) with respect to the specimen's long axis.

A large series of tests was used, and the quality of the test results were assessed, classified and weighted by NAGRA based on the test protocols and completeness of key parameters being monitored during testing (Giger & Marschall 2014). Four quality classes (A to D) were distinguished. The best assigned quality (B) was attributed to test series, in which the pore pressure was controlled (i.e. measured) during testing and small strain rates were utilized (i.e. 1.0E-6 to 1.0E-1 1/s). In the test series attributed with quality D no pore pressure control (i.e. measurement) was used and the utilized strain rate was fast (i.e. 1.0E-5 1/s). The weighing factors for the individual quality classes range linearly between 100% for quality A and 25% for quality D. Usually, the same quality class was assigned to the entire triaxial test series. Only for the triaxial test series carried out by Jahns (2013) the quality classes suggested by Favero et al. (2013) for each individual triaxial test results were utilized by NAGRA.

The weighted data points were further used to establish the effective friction angle and the effective cohesion of Opalinus Clay (i.e. matrix and bedding) at the two depth ranges by a linear-regression analysis through all data points in q-p' space. For a depth up to 400m, data obtained from specimens at the Mont Terri Underground Research Laboratory (URL) was utilized (Jahns 2010, Jahns 2007, Schnier & Stührenberg 2007, Popp & Salzer 2006, Rummel & Weber 2004, Rummel et al. 1999, Olalla et al. 1999). For a depth range between 400 and 900m, data from the boreholes in Benken and Schlattingen was utilized (Jahns 2013, Rummel & Weber 1999). The regression analysis accounts for the individual weighting factors of the different quality classes. According to Giger & Marschall (2014) some uniaxial compression tests were considered in addition to the above mentioned triaxial compression tests to complement the data set in the low stress range. NAGRA's suggested effective strength properties for shallow and deep intact Opalinus Clay are summarized in Table 1 for the matrix and bedding planes.

	Matrix		Bedding	
	<i>ø</i> '(°)	<i>c</i> ' (MPa)	<i>ø</i> '(°)	<i>c</i> ' (MPa)
Opalinus Clay shallow	29	3.1	19	1.7
Opalinus Clay deep	33	7.1	24	3.9

Table 1: Effective strength properties established by NAGRA for the matrix and bedding for shallow and deep Opalinus Clay (NAGRA 2014a).

2.3 Undrained shear strength

Because of the uncertainties stemming from the predominantly conducted consolidated undrained tests (e.g. representativeness of measured pore pressures during consolidation and shearing, NAGRA 2014a) an alternative interpretation based on total stresses (as opposed to effective stresses) was performed assuming unconsolidated undrained testing conditions. A large series of triaxial compression test results² including artificially dried and wetted specimens (Rummel & Weber 1999, Rummel et al. 1999), test results from Mont Terri URL, Benken and Schlattingen (Jahns 2013, Jahns 2010, Rummel & Weber 2004, Rummel & Weber 1999, Rummel et al. 1999, Olalla et al. 1999) were analyzed to establish the undrained shear strength of both matrix and bedding planes (Figure 2a). The undrained shear strength S_u was defined as (NAGRA 2014a):

$$S_u = \frac{\sigma_{1f} - \sigma_{3f}}{2}$$

where σ_{1f} and σ_{3f} are the maximum and minimum principal total stresses at failure. The water content after testing of each specimen was utilized to establish a relationship between the water content w and the undrained shear strength S_u (Figure 2b).

The increase in undrained shear strength with decrease in water content was used as a basis to estimate undrained shear strength values for water content values representative of the actual depth at the potential repository sites. For the derivation of the undrained shear strength of the intact material³ a regression analysis using peak strength values was conducted for both matrix and bedding. A linear relation in the logarithmic diagram was assumed, which allowed to establish the following equation (NAGRA 2014a):

$S_u = Aexp(-Bw)$

where A is the magnitude of S_u for w = 0 (intersection of the regression line with the y-axis) and B is the slope of the regression line⁴. The suggested values for A and B for deriving the undrained shear strength of the intact material (for both matrix and bedding planes) are given in Table 2.

² Results from uniaxial compressive strength tests were not included due to suction effects (NAGRA 2014a).

 ³ Note that rock mass properties (i.e. properties for rock that contains weaknesses) are not discussed in this report.
⁴ Note that for defining undrained shear strength values for rock mass models GM 2 to GM 6 the slope *B* was considered constant. These rock mass models are not discussed in this report.



Figure 2: a) Data basis used for establishing unconsolidated undrained shear strength values for various water contents (NAGRA 2014a); b) Fitting of data for establishing the matrix strength of different rock mass types (note that GM 1 is representative for intact Opalinus Clay; GM 2 to 6 are not discussed in this report.

Table 2: Suggested values for A, B and calculated Su for the depth ranges <400m and 400-900m (NAGRA 2014a). Su is calculated based on the expected water content w at the two depth ranges (i.e. 3.6-4.3% at 900m and 3.8-5.2% at 500m; NAGRA 2014a).

	А	В	Su, OPA deep at 500m (MPa)	Su, OPA deep at 900m (MPa)
			suggested	suggested
Matrix	61.5	23.5	18.1-25.2	21.4-26.4
Bedding	42.4	28.9	9.4-14.1	11.5-15.0

For a water content of 3.6-4.3%, expected at a depth of 900m (NAGRA 2014a), the suggested S_u ranges between 21.4 and 26.4 MPa for the matrix and between 11.5 and 15.0 MPa for the bedding planes. For a lower depth (i.e. 500m) and a higher water content (i.e. 3.8-5.2%) the suggested S_u ranges between 18.1 and 25.2 MPa for the matrix and between 9.4 and 14.1 MPa for the bedding planes. Figure 3 shows how the undrained shear strength of the intact material (for both matrix and bedding planes) increases with decreasing water content based on the values of A and B given in Table 2.



Figure 3: Relation between undrained shear strength and water content for intact Opalinus Clay. The black line represent the relation for the matrix strength, the gray line for the bedding plane strength.

2.4 Elastic properties

The elastic properties (drained and undrained) were determined based on results of uniaxial and triaxial compression tests, oedometer tests and permeameter tests (Giger & Marschall 2014). Therefore, the test results from Mont Terri URL, Benken and Schlattingen were compiled to constrain the elastic properties representative for Opalinus Clay at a depth range \leq 400m (shallow) and 400 to 900m (deep). Concerning the uniaxial and triaxial compression tests, the E-Modulus was either determined as tangent modulus at 50% of the maximum differential stress (i.e. from the primary loading curve), or as secant modulus from unloading/reloading cycles at approximately 30-70% of the maximum differential stress. The E-Modulus from oedometer tests was determined indirectly from the oedometer modulus assuming a linear elastic material behavior.

For constraining the undrained E-Modulus of Opalinus Clay shallow, results from triaxial compression tests compiled by Bock (2009) between 2000 and 2009 were used. Results from S- and P-samples within a particular stress interval were averaged. Drained E-values for Opalinus Clay shallow were taken by NAGRA from oedometer tests on S-samples of three test series (Peron et al. 2009, Horseman et al. 2006 and Chiffoleau & Robinet 1999). For constraining the undrained E-Modulus of Opalinus Clay deep, results of uniaxial and triaxial compression tests from Jahns (2013), Rummel & Weber (1999), Mathier et al. (1999) and Klee & Rummel (2000) were used. Drained E values for Opalinus Clay deep were taken by NAGRA from the two test series by Ferrari et al. (2012; oedometer tests on S-samples) and Horseman & Harrington (2000; during a long term permeameter test on an S-sample).

According to Giger & Marschall (2014) the test results suggest that 1) the ratio between the undrained E-Moduli (E_u) of P-samples and those of S-samples derived in both cases from triaxial tests is in the range of 2:1, 2) the undrained E-Moduli for unloading/reloading cycles increase with increasing effective confining stress, 3) the undrained E-Moduli obtained from the primary loading curve at 50% of the peak strength are lower compared to the values for unloading/reloading cycles and they do not show a clear dependency on the effective confining stress, and 4) the drained E-Moduli (E) for unloading/reloading cycles obtained from oedometer tests also increase with increasing effective confining stress. The

absolute values are approximately 50% of the undrained E-Moduli obtained from unloading/reloading cycles during triaxial tests.

The results of the undrained Poisson's ratios (v_u) obtained from triaxial tests are considered unreliable (Giger & Marschall 2014) because in many cases the results strongly differ from the theoretically derived value of $v_u = 0.50$ which is expected for a linear elastic and isotropic material behavior under undrained conditions. Since only undrained triaxial tests and no drained triaxial tests exist, the drained Poisson's ratios (v) were estimated from the results of uniaxial compression tests.

The drained E-Modulus (based on oedometer tests on S-specimens) suggested by NAGRA (2014a) is E = 2 GPa for Opalinus Clay shallow and E = 4 GPa for Opalinus Clay deep. The undrained E-Modulus (based on uniaxial and triaxial compression tests on S- and P-specimens) suggested by NAGRA (2014a) is $E_u = 4/8$ GPa (normal/parallel to bedding) for Opalinus Clay shallow and $E_u = 9/18$ GPa (normal/parallel to bedding) for Opalinus Clay deep. The related values of the drained Poisson's ratio are v = 0.25 or 0.35 for Opalinus Clay shallow and v = 0.27 or 0.27 for Opalinus Clay deep (NAGRA 2014a).

3 Assessment of geomechanical considerations and properties

3.1 Assessment of the constitutive framework

The constitutive framework and the behavioral aspects described by NAGRA, in particular the effective stress dependent strength and stiffness of the tested rock, are in agreement with many other studies on clay shales (e.g. Aristorenas 1992) and are well described and documented in the literature. Laboratory studies conducted for NAGRA (e.g. Ferrari et al. 2012; data of Jahns 2013 reported in Favero et al. 2013) support the conceptual framework. The introduced simplified elastic-plastic model (i.e. with the Mohr-Coulomb failure envelope corresponding to the Hvorslev yield surface for different depth ranges; with a tension cut-off; without a Roscoe yield surface) might be reasonable for engineering feasibility studies. The simplifications, in particular the modification of the tension cut-off and the omission of the Roscoe yield surface, have some consequences, which need to be considered.

The stress-strain curves of Opalinus Clay obtained from triaxial tests suggest a highly non-linear stressstrain behavior in the pre-failure region. The non-linearity is most probably related to plastic deformations that occur far before reaching the peak strength. Ignoring the Roscoe yield surface this nonlinearity is not explicitly included in the simplified model and the elastic properties for loading and reloading are exactly the same (i.e. the E-Modulus obtained from first loading at 50% of the peak strength is exactly the same as the E-Modulus obtained from unloading/reloading cycles). Figure 4 shows a typical stress-strain curve obtained from a triaxial test on an Opalinus Clay specimen (NAGRA 2014a).



Figure 4: Stress-strain curve for a typical triaxial test on Opalinus Clay (modified from NAGRA 2014a). The blue line represents the behavior of specimen P 109 (Jahns 2013). The green line represents the response of a specimen assuming linear-elastic behavior (i.e. no plastic deformation prior to reach the peak strength) using an E-Modulus obtained from unloading/reloading. The axial strain at failure in the model differs by 0.14% from that in the triaxial test.

The blue line represents the behavior of specimen P 109 (Jahns 2013). The green line represents the response of a specimen assuming linear-elastic behavior (i.e. no plastic deformation in the pre-peak region) using an E-Modulus obtained from an unloading/reloading cycle. The axial strain at failure in the model (at 0.24%) differs by 0.14% from that in the actual triaxial test (at 0.38%). This means that the value in the model corresponds to only 64% of the value in the actual triaxial test. Thus, the simplification may lead to a relevant underestimation of the pre-peak deformation and therefore to a relevant overestimation of the stiffness of Opalinus Clay which needs to be considered in numerical and analytical

engineering design calculations. The simplified model is reasonable for engineering feasibility studies, providing that the consequences of omitting the Roscoe yield surface are considered for the choice of the elastic properties. For quantitative engineering design calculations more advanced constitutive models are required.

3.2 Assessment criteria for effective strength properties

3.2.1 General Remarks

For defining the elastic and effective strength properties from consolidated drained (CD) and consolidated undrained (CU) triaxial tests it is crucial that 1) the pore volume of the specimens is water saturated such that Terzaghi's principle of effective stress for saturated porous media is applicable, and 2) that the load is applied in a sufficiently slow manner such that a) under undrained loading conditions the measured pore pressure response is representative for the specimen response, and b) under drained loading conditions no pore pressure changes arise.

During undrained sample extraction from the deep underground, the pore water pressure drops below atmospheric pressure as the samples tend to expand during unloading and surface tension forces arise at the surface of the samples (i.e. capillary suction at the water-air boundary). The ability of the saturated rock to sustain negative pore water pressure without becoming partly saturated depends on the pore size of the rock (i.e. the smaller pore sizes the larger the maximum sustainable suction). If the imposed negative pore water pressure exceeds the maximum sustainable suction, water will drain from the sample and air will enter the pore space. The absolute value of the maximum sustainable suction (i.e. the water cavitation pressure or air entry pressure) can be several MPa for over-consolidated clays (Bishop et al. 1975). The degree of saturation can further decrease depending on the sealing methodology and the exposure time of the samples during preparation for testing at ambient conditions.

Another relevant process during sample unloading is that excess gas in the pore water will escape from the solution, and may either be trapped in pores (i.e. undrained gas exsolution) or drain from the sample (drained gas exsolution) depending on the pore size distribution (Hight 2001). The amount of gas that can be dissolved in a given volume of pore water is dependent on the pressure according to Henry's law of solubility (Lowe & Johnson 1960).

Since the saturation degree of the samples is likely lower than in-situ (i.e. due to sample extraction and sample preparation, there is a need to re-establish a fully saturated state before laboratory testing. Re-establishing full saturation of low permeability rock types, such as Opalinus Clay, is difficult because air bubbles (i.e. a compressible gas) may be trapped in pores and they need to be dissolved to duplicate in-situ conditions. If the specimen is not fully saturated, Terzaghis's principle of effective stress for saturated porous media may not be applicable (Jennings & Burland 1962, Bishop & Blight 1963). In such a case, the elastic and effective strength properties cannot be determined and/or are not representative for the in-situ conditions since capillary forces arising from a pressure difference between pore water and pore gas exist in the specimen. It should be mentioned here that according to Bishop & Blight (1963) the saturation degree may possibly change during testing as a consequence of specimen compression (i.e. an increase in degree of saturation due to a decrease in pore volume) and dilation (i.e. a decrease in the degree of saturation due to an increase in pore volume).

3.2.2 Testing procedure

Considerable effort has been devoted in the past decades to establish full saturation of specimens and measurement or control of pore pressure during triaxial testing in the laboratory (e.g. Lowe & Johnson 1960, Bishop & Henkel 1962, Wissa 1969), and a state-of-the-art testing procedure for CD and CU tests was established. This procedure requires several steps including 1) a saturation (or backpressure) phase,

2) a hydrostatic loading phase including full consolidation, and 3) a differential loading phase under either drained or undrained conditions.

3.2.3 Saturation (or Backpressure) Phase - Criteria

Saturation of the specimen is usually done in steps. In a first step pore air is removed by specifying a pore pressure gradient. In a second step the remaining pore air is dissolved by applying a sufficient backpressure.

Initially the specimen is isotropically loaded and a fluid backpressure (i.e. typically in the range between 0.1 and 0.5 MPa using de-aired water) is applied on one specimen end-face (inlet), while the fluid at the other specimen end-face (outlet) is kept at atmospheric pressure to allow gas to escape as pore water permeates the specimen. This hydraulic gradient is maintained for a period of time which depends on the specimen's hydraulic conductivity. Full saturation of the specimen requires an elevated backpressure at the specimen end-faces that force the trapped gas in the pore space into solution according to Henry's law of solubility. In theory, the backpressure u_0 required to dissolve air bubbles in a specimen depends on the boundary conditions and two end-members can be distinguished: 1) saturation with continuous water supply at the specimen end-faces, and 2) saturation with no water supply of water at the specimen end-faces.

In the case of continuous water supply (where an increasing water content and a constant rock volume is assumed) the minimum backpressure to saturate a specimen depends on the saturation degree S, Henri's number H (i.e. 0.02) and the atmospheric pressure p_0 (i.e. 0.1 MPa) and is expressed by (Lowe & Johnson 1960):

$$u_0 = ((1-H)(1-S)/H)p_0$$

The theoretical required back-pressure increases linearly with decreasing saturation degree before testing (e.g. $u_0 = 2.5$ MPa for S = 50%, $u_0 = 1.0$ MPa for S = 80% and $u_0 = 0.5$ MPa for S = 90%).

In the case of no water supply (where a constant water content and a decreasing rock volume is assumed) the minimum backpressure to saturate a specimen is higher and is expressed by (Lowe & Johnson 1960):

$$u_0 = (1 - S)/(HS)p_0$$

In practice, the backpressure and the confining stress are increased simultaneously in several stages and are maintained for several hours to days (Lowe & Johnson 1960, Bishop & Henkel 1962, Wissa 1969). For all backpressure stages the effective confining stress is kept approximately constant. After each stage (with a defined backpressure) both inlet and outlet are closed temporarily and Skempton's pore pressure coefficient $B = \Delta u / \Delta p$ is determined by the pore pressure change Δu in relation to the defined increase in hydrostatic stress Δp . After several backpressure stages Skempton's *B* coefficient for a porous media is ideally unity when the specimen is fully saturated (i.e. the load is taken by the pore fluid only). For many rock types, Skempton's *B* coefficient will not reach unity (i.e. B < 1) since the load is partly taken by the rock matrix (Skempton 1954).

Assuming incompressible grains and isotropic elastic material behaviour, Skempton's *B* coefficient can be expressed as $B = 1/(1 + nK/K_w)$ with a bulk modulus of the rock given by K = E/(3(1 - 2v)) and a bulk modulus of the water given by $K_w = 2$ GPa. For a porosity of n = 10%, a drained E-Modulus of E = 1.5 to 6.0 GPa and a drained Poisson's ratio of v = 0.25 (Giger & Marschall 2014) Skempton's *B* coefficient ranges between 0.83 and 0.97. For larger values of the drained E-Modulus, Skempton's *B* coefficient tends to be smaller (e.g. B = 0.75 for E = 10.0 GPa). The above drained elastic properties are in agreement with the results of the oedometer tests on S-samples from the borehole Schlattingen by Ferrari et al. (2012) and the range of calculated B-values is further used as a theoretical assessment criterion for measured B-values in the individual test series.

Giger & Marschall (2014) compiled oedometer tests on S-samples from the Mont Terri URL reported in Peron et al. (2009), Horseman et al. (2006) and Chiffoleau & Robinet (1999). These data suggest that the drained E-Modulus tends to be smaller for an effective confining stress of $\sigma'_3 = 1.0$ to 6.0 MPa as for the case of Opalinus Clay shallow (E = 0.2 to 2.3 GPa) than for an effective confining stress of $\sigma'_3 = 6.0$ to 14.0 MPa as for the case of Opalinus Clay deep (E = 0.7 to 5.2 GPa).

For the porosity different values were compiled by Giger & Marschall (2014) for Opalinus Clay shallow (n = 14.0 to 17.0% for samples from the Mont Terri URL with n = 16% as recommended value) and for Opalinus Clay deep (n = 8.8 to 13.8% for samples from the borehole Schlattingen with n = 11% as recommended value). Therefore, approximately similar theoretical B-values are to be anticipated for Opalinus Clay shallow and for Opalinus Clay deep.

Aristorenas (1992) showed on Opalinus Clay specimens taken from a borehole close to the Wisenberg Tunnel that for the effective confining stresses tested in his study (i.e. 0.5 to 5.0 MPa) Skempton's *B* coefficient ranged between 0.8 and 1.0. Based on CU tests on Opalinus Clay samples taken at the Mont Terri URL Wild et al. (2015) showed, that Skempton's *B* coefficient dropped from approximately 0.9 to 0.7 for an increase in effective confining stress from 0.5 to 4.0 MPa. The decrease in the B-value is primarily related to the increase in the bulk modulus of the rock with increasing effective confining stress. The examples given above suggest that Skempton's *B* coefficient 1) can be smaller than unity at full saturation and 2) is dependent on the effective confining stress. In order to confirm full saturation for such rock types several backpressure stages with simultaneously increasing backpressure and confining stress (i.e. approximately constant effective confining stress) need to be repeated until the B-value remains approximately constant (Wissa 1969; Aristorenas 1992).

Therefore, the assumptions for material parameters made for the theoretical derivation seem to be meaningful, and B-values much below 0.83 (with n = 10%, E = 6.0 GPa and $\nu = 0.25$) or below 0.75 (with n = 10%, E = 10.0 GPa and $\nu = 0.25$) suggest that the specimen will be not fully saturated during differential loading under drained or undrained conditions.

3.2.4 Consolidation Phase - Criteria

Subsequent to specimen saturation a consolidation phase that aims to establish a uniform effective stress state in the specimen is carried out. The time required for approximately 90% dissipation of the excess pore pressure (t_c) can be estimated, if drainage is allowed on both end-faces of the specimen, based on the consolidation theory and is expressed by (Bishop & Henkel 1962):

$$t_c = 0.196 H^2 / c_i$$

where H is the sample height and c_v is the consolidation coefficient defined as:

$$c_v = kK/\gamma_w$$

where k is the hydraulic conductivity, K the bulk modulus of the rock and γ_w the unit weight of water. For isotropic consolidation (radial strain is inhibited in the oedometer test, but not in the triaxial compression test). Head (1992) recommends to consider the bulk modulus of the rock (K) instead of the oedometer modulus of the rock (E_{oed}) for the derivation of the consolidation coefficient. The two parameters are related by $K/E_{oed} = (1 + \nu)/(3(1 - \nu))$. For Opalinus Clay the bulk modulus of the rock and the hydraulic conductivity depend on the effective confining stress. Based on the results of the oedometer tests on S-samples from the borehole Schlattingen by Ferrari et al. (2012) the hydraulic conductivity k ranges between 1.0E-14 and 5.0E-13 m/s and the bulk modulus of the rock K ranges between 1.0 and 4.0 GPa (corresponding to a drained E-Modulus E between 1.5 and 6.0 GPa with the assumption of $\nu = 0.25$ for the drained Poisson's ratio (Giger & Marschall 2014). According to Ferrari et al. (2012) three different stress ranges are considered: $\sigma'_3 < 5.0$ MPa, $\sigma'_3 = 5.0$ to 10.0 MPa and $\sigma'_3 >$ 10.0 MPa. With increasing effective confining stress the hydraulic conductivity is decreasing (k = 2.0E-13 to 5.0E-13 m/s for $\sigma'_3 < 5.0$ MPa, k = 1.0E-14 to 2.0E-14 m/s for $\sigma'_3 > 10.0$ MPa) and the bulk modulus of the rock is increasing (K = 1.0 to 2.0 GPa for $\sigma'_3 < 5.0$ MPa, K = 2.0 to 4.0 GPa for $\sigma'_3 > 10.0$ MPa). The resulting consolidation coefficient ranges between $c_v = 0.002$ and 0.100 mm²/s and decreases with increasing effective stress ($c_v = 0.020$ to 0.100 mm²/s for $\sigma'_3 < 5.0$ MPa, $c_v = 0.002$ to 0.008 mm²/s for $\sigma'_3 > 10.0$ MPa). Note that the above numbers are representative for S-samples (i.e. specimens tested normal to bedding). Both the hydraulic conductivity and the bulk modulus of the rock tend to be higher for P-samples (i.e. specimens tested parallel to bedding) and thus higher consolidation coefficients are to be anticipated. For Opalinus Clay shallow the bulk modulus of the rock is smaller than for Opalinus Clay deep. For hydraulic conductivities in the same range, smaller consolidation coefficients are to be anticipated for Opalinus Clay shallow than for Opalinus Clay deep.

In practice the consolidation degree is typically assessed by examining the time-dependent development of volumetric strain and/or change in water content associated with excess pore pressure dissipation. The sample is considered to be consolidated if no further volumetric strain and/or change in water content occurs.

3.2.5 Shearing Phase - Criteria

During drained shearing of rock types with very low permeability such as Opalinus Clay, even small loading rates may lead to a heterogeneous distribution of excess pore pressure within the specimen and thus the effective strength properties derived from the test results are unreliable. Such a heterogeneous distribution of effective stresses within the specimen can largely be avoided in CD tests by selecting an appropriate loading rate. Bishop & Henkel (1962) suggest that the error in determining effective strength properties is negligible when the specimens are loaded in such a way that 95% of the excess pore pressure can dissipate under drained loading conditions. In case drainage is allowed on both end-faces of the specimen, the time required to dissipate 95% of the excess pore pressure (t_f) in CD tests that are brought to failure can be estimated, according to the following equation (i.e. assuming an aspect ratio of the specimen of 1:2):

$t_f = 1.667 H^2 / c_v$

During undrained shearing, Opalinus Clay tends to compact or dilate which causes the pore pressure in the specimen to change. The amount of pore pressure change depends on the elastic properties and the tendency of the material to compact or dilate during the failure process. In triaxial cells, pore pressure transducers are usually connected to the specimen via filter material at both specimen's end-faces and via drainage lines close to the specimen. The fluid filled drainage system can experience minor volume changes during undrained shearing due to its compliance (Wissa 1969, Bishop & Henkel 1962). This can induce fluid flow into or out of the specimen to achieve pressure equilibrium between the specimen and the drainage system. The time to reach a pressure equilibration depends on the compliance of the drainage system. Blight (1964) also showed that the equilibration of the pore pressure between the central failure zone and the end-faces of the specimen may require significant time and is the principal source of inaccuracy in CU tests. As a consequence of both issues, the measured pore pressure change may not be representative for the pore pressure within the specimen in case of too fast loading rates, and thus the determination of effective strength properties based on such test results is unreliable.

Since the amount of fluid flow is substantially smaller under undrained loading conditions, CU tests can be performed faster than CD tests. Blight (1964) suggested that CU tests can be brought to failure about 4 times faster than CD tests, which leads to the following equation (i.e. assuming an aspect ratio of the specimen of 1:2):

$$t_f = 0.400 H^2 / c_v$$

This recommendation is based on pore pressure measurements during CU tests in the centre and at both end-faces of the specimen with only minor effects of the drainage system (Blight 1964). Depending on the compliance of the fluid filled drainage system used in the individual laboratory the time required to reach failure can be even longer.

In addition to the above theoretical considerations, the adequacy of the chosen loading rates can be assessed by examining the magnitude of pore pressure change that evolves during undrained shearing. Skempton's pore pressure coefficient $\bar{A} = \Delta u / \Delta q$ is defined as the ratio between the pore pressure change Δu and the differential stress Δq , and is typically determined either at failure or at 50% of the maximum differential stress. Assuming incompressible grains and isotropic elastic material behaviour, Skempton's \overline{A} coefficient is expressed as follows: $\overline{A} = 1/(3(1 + nK/K_w))$. If the compressibility of the pore water is small compared to the compressibility of the rock, the condition of maintaining a constant water content during undrained shearing implies a constant rock volume and Skempton's \bar{A} coefficient equals $\overline{A} = 0.33$. Here the bulk modulus of the water is given by $K_w = 2$ GPa leading to lower values of Skempton's \overline{A} coefficient depending on the porosity n, the drained E-Modulus E and the drained Poisson's ratio v. For n = 10%, E = 1.5 to 6.0 GPa and v = 0.25 Skempton's \overline{A} coefficient ranges between 0.28 and 0.32. For larger values of the drained E-Modulus, Skempton's \overline{A} coefficient tends to be smaller (e.g. $\overline{A} = 0.25$ for E = 10.0 GPa). Note that the above considerations for determining the \overline{A} -value (just as for determining the B-value) assume that the compliance of the drainage system and the compressibility of the water in the drainage system are both negligible. The porosity is larger for Opalinus Clay shallow than for Opalinus Clay deep. On the other side, the E-Moduli tend to be smaller for Opalinus Clay shallow which has a contrary effect on the Ā-value.

Therefore, the assumptions made regarding the material parameters for the theoretical derivation seem to be meaningful and \overline{A} -values much below 0.28 (with n = 10%, E = 6.0 GPa and $\nu = 0.25$) or below 0.25 (with n = 10%, E = 10.0 GPa and $\nu = 0.25$) suggest that the specimen was either not fully saturated during differential loading under undrained conditions or the loading rate was too fast to capture the actual pore pressure response within the specimen. This means that the \overline{A} -value, as an indicator for an appropriate loading rate, can only be applied if saturation of the specimen is assured during undrained shearing.

3.2.6 Assessment criteria for an adequate testing procedure

For each testing phase two criteria were utilized to assess the CD and CU tests which are summarized in Table 3. Whereas the first criterion is based on theory, the second criterion is based on actually measured data.

Testing Phase	Theoretical Criterion	Data based Criterion
Saturation	C-Ia Backpressure magnitude	C-Ib Assessment of Skempton's B
	With water supply:	Repeated B-value checks
	$u_0 = ((1 - H)(1 - S)/H)p_0$	
	Without water supply:	
	$u_0 = (1 - S)/(HS)p_0$	
	u_0 required backpressure	
	S saturation degree	
	H Henri's number (0.02)	
	p ₀ atmospheric pressure (0.1 MPa)	
Consolidation	C-IIa Theoretical consolidation time	C-IIb Development of volumetric strain and change in water content
	$t_c = 0.196 H^2/c_v$	
	t_c required time to fulfill consolidation	
	H sample height	
	c_v consolidation coefficient	
	$c_v = kK/\gamma_w$	
	k hydraulic conductivity	
	K bulk modulus of rock	
	γ_w unit weight of water	
	k, K depend on sample orientation to	
	bedding and effective confining stress	
Shearing	C-IIIa Theoretical shearing time	C-IIIb Assessment of Skempton's \overline{A}
	Drained conditions:	
	$t_f = 1.667 H^2/c_v$	
	Undrained conditions:	
	$t_f = 0.400 H^2/c_v$	
	t _f required time to reach failure	
	H sample height	
	c_v consolidation coefficient	
	$c_v = kK/\gamma_w$	
	k hydraulic conductivity	
	K bulk modulus of rock	
	γ_w unit weight of water	
	k, K depend on sample orientation to	
	bedding and effective confining stress	

Table 3: Summary of criteria used to assess triaxial test results.

3.2.7 Consequences of an inadequate testing procedure

The following example demonstrates the possible consequences of an inadequate testing procedure with regard to sample pre-saturation and control of pore pressure during triaxial testing.

Figure 5 shows the results of triaxial tests on kakiritic gneisses, slates and phyllites from the northern Tavetsch massiv (Vogelhuber 2007). These kakiritic rocks are prone to squeezing and were crossed by the Gotthard high-speed railway tunnel in the area of Sedrun with an overburden of more than 800 m. In order to establish the mechanical characteristics of the kakiritic rocks, a comprehensive testing program was carried out. Particular attention was paid to pre-saturation of the samples and control of pore water pressure during testing. The hatched area contains the effective stress states at failure for 63 CD and CU tests, which were executed either as single- or as multi-stage triaxial tests. Despite the complex structure of the kakiritic rocks, remarkably consistent test results were obtained for the effective strength parameters (c' = 0.7 MPa and $\phi' = 31.0^{\circ}$ as upper bound of the yield condition, c' = 0.5 MPa and $\phi' = 21.0^{\circ}$ as lower bound of the yield condition), shown in Figure 5. The observed scatter is due to the heterogeneity of the samples.



Figure 5: Triaxial tests on kakiritic gneisses, slates and phyllites from the northern Tavetsch massiv.

Besides the tests described above, several triaxial tests have been carried out using equipment often employed in rock mechanics for conventional triaxial testing, i.e. without sample saturation and control or monitoring of pore water pressure. For the examination of these UU tests the change in pore pressure is ignored and for the sake of simplicity the effective stresses are considered to be equal the total stresses. Here a much larger scatter of the test results was observed. A detailed investigation revealed that the test results depend essentially on the moisture content before the start of the test. The strength parameters may be either overestimated (i.e. when the moisture content is low due to previous drying, see sample No. 84b in Figure 5 with c' = 2.1 MPa and $\phi' = 34.9^{\circ}$) or underestimated (i.e. when the moisture content is high due to previous wetting, see sample 88b in Figure 5 with c' = 0.3 MPa and $\phi' = 10.3^{\circ}$). The large scatter observed in these tests is obviously the result of a completely unsuitable testing procedure. This example emphasizes the importance of pre-saturation of samples and control or monitoring of pore water pressure during triaxial testing.

It is interesting to note that the obtained effective strength parameters from CD tests were not affected by the moisture content before testing. Here, the previous drying/wetting has no influence on the test results because pre-saturation of the samples brings them in both cases to the same effective stress conditions after consolidation (i.e. before shearing). This was shown by additional tests on samples which were either dried or wetted prior to testing (see sample No. 84a with c' = 0.6 MPa and $\phi' = 26.3^{\circ}$ or sample No. 88a with c' = 0.4 MPa and $\phi' = 30.9^{\circ}$ in Figure 5). The above example illustrates how an inadequate testing procedure and an incomplete saturation affect the determination of the effective strength properties and serve as major sources of erroneous conclusions.

In this report two basic cases leading to erroneous conclusions are considered: 1) the strength is underestimated in case the consolidation is incomplete (i.e. the excess pore pressure within the sample is not fully dissipated) or the specimens are loaded too fast (i.e. the measured excess pore pressure at the end-faces of the sample does not represent the pore pressure within the sample), and 2) the strength is overestimated if saturation is incomplete (i.e. capillary suction exists). Whereas the former case (strength is underestimated) is illustrated in Figure 6a, the latter case (strength is overestimated) is illustrated in Figure 6b (both in principal effective stress space and q-p' space).



Figure 6: Underestimation/Overestimation of strength properties for the case of an incomplete consolidation or too fast loading (a) and for the case of an incomplete saturation (b).

3.3 Assessment criteria for undrained shear strength properties

3.3.1 Background

The unconsolidated undrained (UU) triaxial tests provide a "quick" estimate of the undrained shear strength of saturated porous media for loading situations where the time to dissipate excess pore pressures is insufficient (e.g. short-term behavior during tunnel construction in low permeability rocks). In this kind of test the pore water pressures that develop during undrained shearing are typically not measured and only total stresses are considered for determining the undrained shear strength S_u . Thus, in contrast to CD and CU tests, a consolidation phase, is in principle, not required and there are no restrictions on the loading time during the shearing phase as discussed in the previous section. After applying the target confining stress no consolidation is allowed and the differential stress is increased under undrained conditions until failure is reached. Therefore, the drainage lines of the fluid filled drainage system are closed at all test stages.

Results obtained from UU tests on saturated isotropic soils typically show that the value the undrained shear strength S_u is independent of the applied confining stress and the interpretation of the failure envelope in total stresses suggests a friction angle of $\phi_u = 0^\circ$. The reason for obtaining a value for the undrained shear strength S_u that is independent of the confining stress is illustrated in Figure 7 and described in the following.



Figure 7: a) τ - σ diagram with representation of a CU test where the specimen is consolidated to a confining stress at point A and sheared until failure occurred (Mohr circle 1). Mohr circle 2 represents corresponding test results if the confining stress was further increased from point A to point B without allowing any consolidation before shearing. b) q-p/p' diagram which shows effective strength properties of two CU tests consolidated to an effective confining stress of different magnitude X and Y (solid symbols). The open symbols represent the undrained shear strength from UU tests representative of specimens initially consolidated either to X (circles) or to Y (squares).

In Figure 7a (τ - σ diagram), the sample is assumed to be initially consolidated at a given confining stress (point A). In a first test, the confining stress is maintained at point A and the axial stress is increased while the radial stress remains constant until failure is reached (i.e. a CU test) with a value of $q_{f,A}$ for the

deviatoric stress at failure (Mohr circle 1). In a second test, the confining stress is further increased from point A to point B by $\Delta\sigma_3$ without allowing any consolidation and then sheared in the same manner as before. Here a value of $q_{f,B}$ for the deviatoric stress at failure (Mohr circle 2) results which is exactly the same as $q_{f,A}$. This implies that Skempton's *B* coefficient is unity (i.e. B = 1) and therefore the pore pressure change Δu during hydrostatic loading under undrained conditions is equal to the confining stress change $\Delta\sigma_3$ from point A to point B given by $\Delta u = B\Delta\sigma_3$. The effective stresses in the second test remain unchanged and equal to the effective stresses in the first test, suggesting a friction angle of zero ($\phi_u = 0^\circ$) with respect to total stress conditions (dashed bold line in Figure 7a). Only in this case the undrained shear strength S_u is independent of the confining stress and equal to the radius of the Mohr circle at failure which can be expressed as:

$$S_u = q_f/2$$

In Figure 7b (q-p/p' diagram), the undrained shear strength S_u of two samples which have been initially consolidated to different effective confining stresses X and Y (i.e. representing different values for water content) is given by two different horizontal lines. The Mohr-Coulomb failure envelope (dashed bold line in Figure 7b) implies that the value of S_u increases with increasing effective confining stress.

The basic background for conducting reliable UU tests, and the requirements of the $\phi_u = 0^\circ$ concept to be valid, suggests that the samples need to be saturated (i.e. S = 100%) and Skempton's *B* coefficient be unity (i.e. B = 1). In addition, the undrained shear strength S_u is not a unique number but represents the interpretation of the failure envelope in total stresses for a given initial effective confining stress.

3.3.2 Su of partially saturated specimens

In an ideal case, undrained sample extraction from the deep underground causes the pore water pressure to drop as a consequence of the tendency of the samples to expand during unloading. Assuming hydrostatic in-situ stress conditions and a linear elastic, isotropic material behavior with B = 1 (i.e. the compressibility of the pore water is small compared to the compressibility of the rock specimen), the effective stress in the specimen remains unchanged. Therefore, UU tests on such samples provide results for the undrained shear strength S_u which are representative for the initial effective stress at the sampling depth.

If the pore water pressure drops to values below the maximum sustainable suction, the samples will desaturate and the effective stress in the specimen will differ from the initial effective stress at the sampling depth. Under these circumstances the effective stress in the specimen is unknown. In addition, sample storage and sample preparation (i.e. exposure to the laboratory environment) can contribute significantly to a further decrease of the saturation degree. Wild et al. (2014) showed that de-saturation occurs rapidly within the first minutes after sample dismantling and exposure to ambient conditions. This results in an increase of the effective stress in the specimen which cannot be quantified. Terzaghi's principle of effective stress for saturated porous media is not applicable anymore.

For a saturation degree S < 100% the evaluation of the undrained shear strength S_u according to the concept of $\phi_u = 0^\circ$ does not apply since Skempton's *B* coefficient is smaller than unity (i.e. B < 1). As a consequence, the undrained shear strength S_u obtained from UU tests on partially saturated samples does not represent the shear strength under undrained conditions for the effective stress conditions in-situ.

It is noted here, that for a confining stress increase during UU tests on partially saturated samples, the saturation degree *S* may approach unity if the initial capillary suction is small (e.g. Bishop & Eldin 1950, Fredlund & Vanapalli 2002). The confirmation of full saturation during UU tests on initially partially saturated samples requires, however, a series of tests starting at the same effective stress state and utilizing several confining stress increases.
3.3.3 Su of fully saturated specimens with B smaller than unity

As shown in the previous sections, the $\phi_u = 0^\circ$ concept is based on the assumption that the Skempton's *B* coefficient is unity. However, for clay shales the Skempton's *B* coefficient can be smaller than unity even for a saturation degree S = 100% (Aristorenas 1992, Giger & Marschall 2014) and may decrease with increasing effective confining stress due to an increase in the bulk modulus of the rock (Cook 1999, Wild et al. 2015). As a consequence, an increase of the confining stress during UU tests leads to an increase of the effective confining stress and the $\phi_u = 0^\circ$ concept is no longer valid (i.e. $\phi_u > 0^\circ$). Under undrained conditions the differential stress at failure (i.e. the maximum difference between the axial stress and the radial stress) of rock types with B < 1 can be expressed using the following equation (see Appendix A1 for the assumptions and the derivation of the equation):

$$\sigma_{1f} - \sigma_{3f} = \frac{3(m-1)(1-B)}{3+B(m-1)} (\sigma_{3f} - \sigma_0) + \frac{3(m-1)}{3+B(m-1)} \sigma'_0 + \frac{3}{3+B(m-1)} f_c$$

with the coefficients $m = (1 + \sin\phi')/(1 - \sin\phi')$ and $f_c = 2c'\cos\phi'/(1 - \sin\phi')$ related to the Mohr-Coulomb failure criterion and the initial effective stress σ'_0 .

Figure 8 shows the shear strength of saturated rocks under undrained conditions with B = 1 (Figure 8a and b) and B < 1 (Figure 8c and d) in the effective stress space (continuous blue line) as well as in the total stress space (continuous red line). Figure 8a and c (i.e. $\sigma_1 - \sigma_3$ and $\sigma'_1 - \sigma'_3$ diagram respectively) and Figure 8b and d (i.e. q-p and q-p' diagram respectively) are two different representations of the same relationship. As an example an effective friction angle $\phi' = 30^{\circ}$, an effective cohesion c' = 1 MPa, a porosity n = 15%, a drained E-Modulus E = 10 GPa, a drained Poisson's ratio $\nu = 0.25$, a bulk modulus of the water $K_w = 2$ GPa (i.e. B = 0.67) and an initial effective stress $\sigma'_0 = 7.5$ MPa are assumed. Both, the effective stress paths (ESP) and the total stress paths (TSP) are shown in Figure 8. For B = 1.0 the $\phi_u = 0^{\circ}$ concept applies when considering only total stresses (Figure 8b) and the shear strength of a saturated rocks under undrained conditions can be expressed by a single value (i.e. the undrained shear strength $S_u = 5.54$ MPa). For B = 0.67 (Figure 8d) the undrained cohesion would be $c_u = 2.90$ MPa and the undrained friction angle would be $\phi_u = 10.8^{\circ}$.

The example illustrates that for rock types with B < 1 the assumption of $\phi_u = 0^\circ$ underestimates the undrained shear strength S_u for confining stresses and/or mean stresses higher than the tested value and overestimates the undrained shear strength S_u for confining stresses and/or mean stresses smaller than the tested value. Skempton's *B* coefficient needs to be unity for the concept of $\phi_u = 0^\circ$ to be valid when considering only total stresses. Even though it was shown that Skempton's *B* coefficient can be smaller than unity for Opalinus Clay (Jahns 2013), the assumption of B = 1 is acceptable for conceptual engineering design methods addressing the short-term undrained rock mass response.



Figure 8: Undrained shear strength, total stress path (TSP) and effective stress path (ESP) in principal stress space and q-p/p' space for a rock with B = 1.0 (a, b) and B < 1.0 (c, d).

4 Assessment of the tested sample geometries and related strength properties

Both, the strength and stiffness of Opalinus Clay depends on the angle between the loading direction and the bedding plane orientation. Assuming a transversal isotropic medium, the stiffness of specimens loaded parallel (P-samples) to bedding is smaller than that of specimens loaded normal (S-samples) to bedding. However, the strength of specimens loaded parallel (P-sample) and normal (S-sample) to bedding are the same (i.e. this assumption was made by NAGRA for establishing the effective matrix strength and the undrained shear strength). For any geometrical loading configuration with the bedding plane orientation inclined to the loading direction, the strength may drop. This is illustrated in Figure 9, which shows how the uniaxial strength varies for an increasing bedding plane orientation with respect to the load axis. For an angle $\beta = 0^{\circ}$ the load axis is parallel to the bedding plane orientation. For an angle $\beta = 90^{\circ}$ the load axis is normal to the bedding plane orientation. The example is based on the assumption of a Mohr-Coulomb failure criterion using the effective strength of the matrix and the bedding given in Table 1 for Opalinus Clay deep. The minimum uniaxial strength results for an angle of $\beta = 45^{\circ} - \phi'/2$ for the bedding plane orientation with respect to the loading direction (with ϕ' as the effective friction angle along the bedding plane, see Figure 9). Therefore, triaxial tests using X-samples (i.e. the angle between the load axis and the bedding plane is 30°) provide a reasonable estimate of the bedding plane strength (provided that the effective friction angle is in the order of 30°). This is in contrast to triaxial tests using Z-samples (i.e. the angle between the load axis and the bedding plane is 45°) which overestimate the bedding plane strength.



Figure 9: Variations uniaxial strength with increasing angle between loading axis and bedding plane orientation.

5 Assessment of effective strength properties

In the following sections test series utilized by NAGRA for establishing effective strength properties are assessed following the assessment criteria in the previous sections. The assessment focuses on whether test results are suitable for establishing effective strength properties or not.

All triaxial compression tests reported in NAGRA (2014a) and Giger & Marschall (2014) were analyzed. In contrast, the uniaxial compression tests were not analyzed due to the inherent uncertainties stemming from capillary suction and test boundary conditions. A further focus of this assessment was on triaxial test results on samples taken from the boreholes Schlattingen (Jahns 2013) and Benken (Rummel & Weber 1999). These test series have utilized samples which are considered most relevant for characterizing the effective strength properties of Opalinus Clay at the actual siting regions (i.e. the case of Opalinus Clay deep). Triaxial test results from samples taken from the shallow subsurface at the Mont Terri URL (Jahns 2010, Jahns 2007, Schnier & Stührenberg 2007, Popp & Salzer 2006, Rummel & Weber 2004, Rummel et al. 1999, Olalla et al. 1999) seem to be less representative (i.e. the case Opalinus Clay shallow). The level of detail in reporting the assessment of triaxial test results is therefore different in the following sections. Even though, all the assessment criteria have been applied equally to all test series.

Basic properties and the assessments of the test series reported in Jahns (2013) and Rummel & Weber (1999) are given in Appendix A2 (basic physical properties reported in Jahns 2013), A3 (assessment of triaxial test results reported in Jahns 2013), A4 (basic physical properties reported Rummel & Weber 1999), and A5 (assessment of triaxial test results reported in Rummel & Weber 1999).

Throughout the document the porosity *n* of the samples was calculated from the dry unit weight γ_d and the unit weight of the solids γ_s with $n = 1 - \gamma_d/\gamma_s$, and the saturation degree *S* of the samples was calculated from the dry unit weight of the rock γ_d , the unit weight of water γ_w , the water content *w* and the porosity *n* with $S = (w\gamma_d)/(n\gamma_w)$.

5.1 Jahns 2013, NAB 13-18

5.1.1 General

The triaxial tests reported in Jahns (2013) have been performed on specimens with diameter D = 25mm and height H = 50mm. In total 24 Opalinus Clay specimens with a bedding plane orientation normal (11 S-samples), parallel (6 P-samples) and inclined (4 X-samples⁵ and 3 Z-samples⁶) to the core axis were obtained by overcoring larger diameter cores taken from the borehole Schlattingen in a depth range of 900 to 910m. Prior to overcoring these cores were stored in pressure vessels or resin filled core liners. The exposure time to the ambient laboratory conditions was kept as short as possible.

All 24 triaxial tests were reviewed and classified in Favero et al. (2013). According to Giger & Marschall (2014) the results of 19 out of 24 triaxial tests (8 S-samples, 4 P-samples, 4 X-samples, 3 Z-samples) were used for establishing effective strength properties.

5.1.2 Water content, porosity and saturation degree

The porosity *n* calculated from the dry unit weight $\gamma_d = 24.0$ to 24.6 kN/m³ and the unit weight of the solids $\gamma_s = 26.8$ to 27.2 kN/m³ given in Jahns (2013) ranges between 9.0 and 10.9% and is consistent with the reported values for the porosity. The saturation degree *S* can be calculated from the water content and ranges between 99 and 113% (with w = 3.8 to 4.9%) before testing and between 110 and 131% (with w = 4.3 to 5.7%) after testing. A saturation degree higher than 100% is physically impossible and raises the question if the basic rock mechanical properties (dry unit weight, unit weight of the solids, water

⁵ i.e. 30° inclined bedding planes with respect to the core axis.

⁶ i.e. 45° inclined bedding planes with respect to the core axis.

content) were properly determined. The actual values for the saturation degree before and after testing remain, therefore, unknown.

5.1.3 Triaxial testing procedure

The testing procedure comprises the following testing phases:

- 1) Combined saturation and consolidation phase under hydrostatic loading conditions with a confining stress σ_3 ranging between 7.6 and 22.6 MPa and a backpressure u_0 ranging between 3.0 and 9.0 MPa. The utilized fluid was demineralized water with 9.24 g/l NaCl.
- 2) Differential loading phase under undrained conditions with an axial strain rate $\Delta \varepsilon_1 / \Delta t$ in the range of 1.0E-4 1/s to 1.0E-7 1/s.

During axial displacement controlled undrained shearing at constant radial stress, the pore pressure as well as the axial and radial strain and the change in water content were continuously monitored. The strains were monitored within the pressure vessel.

Difficulties in assessing the test series based on the described assessment criteria arise from the applied testing procedure. Typically the consolidation phase follows the saturation phase. The two distinct testing phases have been combined in Jahns (2013) and are performed in parallel. This has an effect on certain assessment criteria. Incomplete consolidation may influence the reliability of the measured B-value for the assessment of a complete saturation. Incomplete saturation may lead to swelling rather than consolidation during hydrostatic loading, which affects the validity of the measured volumetric strain and/or change in water content.

5.1.4 Assessment of the test phases

Saturation Phase criterion, C-Ia and C-Ib

The saturation process is limited to the application of a backpressure (dissolving pore air in the pore water) and does not include the application of a pore pressure gradient (removing pore air out of the pore water) prior to the backpressure phase. The minimum backpressure used for this test series was $u_0 = 3.0$ MPa which allows, in theory, saturating a specimen with a saturation degree of S = 39% before testing (if continuous supply of water is provided and the saturation process is long enough). Despite uncertainties regarding the actual values for the saturation degree before testing, full saturation of the specimens is in principle possible for all triaxial tests conducted by Jahns (2013) with the chosen values of the backpressure (Figure 10).



Figure 10: Required backpressure versus saturation degree for the two cases of continuous water supply and no water supply. For all specimens tested by Jahns (2013) the calculated saturation degree before testing and the utilized backpressure is shown. Note that the calculated saturation degree is higher than 100% for most of the specimens, which is physically impossible.

Skempton's *B* coefficient was not determined in a first series of tests (5 specimens), but in a second series of tests (19 specimens). As outlined in section 3 the B-value is anticipated for Opalinus Clay specimens to range between B = 0.83 and B = 0.97 (with a minimum value of B = 0.75 and a maximum value of B = 1.00) for full saturation.



Figure 11: Assessment of B-values obtained by Jahns (2013). The blue lines indicate the theoretical threshold of B-values for a typical range of elastic properties of Opalinus Clay according to Ferrari et al. 2012.

For 6 tests (specimens S03, S102, S106, P13, Z19, Z21) B-values considerably higher than B = 1.0 were determined (Figure 11). This can only be explained by an incomplete consolidation phase. However, the test results for Skempton's *B* coefficient suggest that the specimens must have been almost saturated.

Confirmation of full saturation is not possible due to the influence of the consolidation phase and clear conclusions are not possible.

For another 7 tests (specimens S05, S06, S07, P09, P10, P14, Z23) the B-values were lower than B = 0.5 suggesting partially saturated conditions (Figure 11). For another 6 tests (specimens P109, P115, X24, X25, X27, X30) the B-values were considerably higher than B = 0.5 and lower than B = 1.0 suggesting fully saturated conditions (Figure 11). Full saturation can additionally be confirmed for 4 of 6 specimens by approximately similar test results for Skempton's *B* coefficient in two consecutive loading steps.

Consolidation Phase, criterion C-IIa and C-IIb

The minimum theoretical time t_c required to fulfill consolidation of a specimen with H = 50mm ranges between $t_c = 1.4$ to 7h for $\sigma'_3 < 5$ MPa and $t_c = 17$ to 68h for $\sigma'_3 > 10$ MPa. According to Jahns (2013) the utilized consolidation time was approximately 24h for the first part of the test series, and at least 6h for the second part of the test series. According to the diagnostic analysis conducted by Favero et al. (2013) the minimum duration was 9h and the maximum duration was 140h. Therefore, the utilized consolidation time for the 5 specimens of the first test series and the 19 specimens of the second test series is in a similar range as the minimum required consolidation time (Figure 12). For none of the triaxial tests the duration was clearly shorter than the theoretically required time to fulfill consolidation.



Figure 12: Assessment of the consolidation time utilized by Jahns (2013). The blue lines indicate the theoretical threshold of the minimum required consolidation time for a typical range of elastic properties of Opalinus Clay according to Ferrari et al. 2012.

The time dependent development of the volumetric strain and the change in water content is not reported in Jahns (2013), but in the in the diagnostic analysis conducted by Favero et al. (2013). As mentioned above, the saturation phase and the consolidation phase overlap in this test series and complicate the assessment of whether these two phases were successfully completed. Incomplete saturation affects both the volumetric strain and the change in water content. Instead of pore water drainage out of the specimens due to dissipation of excess pore pressure following an increase in confining stress, pore water infiltration into the specimens as a consequence of the applied backpressure can occur (i.e. a swelling process as opposed to a consolidation process).

The detailed assessment shows that for 15 tests water infiltration and volume expansion (i.e. swelling) occurred, and only for 9 tests water drainage and volume compaction (i.e. consolidation) occurred. The detailed examination of the variation of change in water content and volumetric strain suggest that the consolidation or swelling process was not finished for most of the specimens. Swelling was completed for 1 and not completed for 12 specimens. For another 2 specimens the volumetric strain and the change in water content were insignificant, suggesting acceptable completion of the swelling phase. Consolidation was completed for 5 and not completed for 2 specimens. For another 2 specimens the volumetric strain and the change in water content were insignificant, suggesting acceptable completion of the swelling phase. Consolidation phase.

Thus, the consolidation or swelling process was adequately completed for 10 tests (specimens 01 to 05, S102, S106, P109, X27, X30). For 2 another tests (specimens S03, P13) the consolidation process was not finished and therefore the duration of the consolidation phase was clearly too short. For another 12 tests (specimens S05, S06, S07, P09, P10. P14, P115, X24, X25, Z19, Z21, Z23) the swelling process was not finished and therefore the influence of the testing procedure (i.e. saturation phase parallel to consolidation phase) does not allow an assessment of the adequate duration of the consolidation phase.

Failure Phase criterion, C-IIIa and C-IIIb

The minimum theoretical time t_f required to reach failure of a specimen with H = 50mm ranges between $t_f = 2.8$ to 14h for $\sigma'_3 < 5$ MPa and $t_f = 35$ to 139h for $\sigma'_3 > 10$ MPa. According to Jahns (2013) and Favero et al. (2013) the utilized shearing time ranges between 1min and 17h (excluding the time required for unloading-reloading cycles). In the first part of the test series (5 specimens), the axial strain rate was varied by a factor of 1000 to analyze the influence of the loading rate on the measured pore pressure response. For the slowest axial strain rate of $\Delta \varepsilon_1 / \Delta t = 1.0\text{E-}7$ 1/s the shearing time was 17h and in the same range as the theoretically required time to reach failure (Figure 13). In the second test series (19 specimens), a tenfold higher axial strain rate of $\Delta \varepsilon_1 / \Delta t = 1.0\text{E-}6$ 1/s was applied corresponding to a minimum duration of 0.6h and a maximum duration of 1.8h. This is considerably shorter than the minimum required shearing time (Figure 13). It should, however, be noted that the time required to reach failure might be shorter for P-samples (i.e. specimens tested parallel to bedding) than for S-samples (i.e. specimens tested normal to bedding), since both the hydraulic conductivity and the bulk modulus of the rock tend to be larger in this testing configuration. Thus, the chosen axial strain rate in the second part of the test series ($\Delta \varepsilon_1 / \Delta t = 1.0\text{E-}6$ 1/s) might be adequate for P-samples, but certainly not for S-samples.



Figure 13: Assessment of the loading time utilized by Jahns (2013). The blue lines indicate the theoretical threshold of the minimum required shearing time for a typical range of elastic properties of Opalinus Clay according to Ferrari et al. 2012.

In theory, Skempton's \bar{A} coefficient ranges for fully saturated and slow enough loaded Opalinus Clay specimens between $\bar{A} = 0.28$ and $\bar{A} = 0.32$ (with a minimum value of $\bar{A} = 0.25$ and a maximum value of $\bar{A} = 0.33$) according to section 3. The test results from the first test series (5 specimens; only S-samples) show that the \bar{A} -value decreases significantly with an increasing axial strain rate (Table 4), irrespectively if the \bar{A} -value is determined at 50% of the maximum differential stress or at failure. \bar{A} -values of $\bar{A} = 0.35$ for $\Delta \varepsilon_1 / \Delta t = 1.0\text{E-}7$ 1/s, $\bar{A} = 0.27$ for $\Delta \varepsilon_1 / \Delta t = 1.0\text{E-}6$ 1/s, $\bar{A} = 0.13$ for $\Delta \varepsilon_1 / \Delta t = 1.0\text{E-}5$ 1/s and $\bar{A} = 0.06$ for $\Delta \varepsilon_1 / \Delta t = 1.0\text{E-}4$ 1/s were identified (at 50% of the peak strength, Figure 14). This suggest that both axial strain rates of $\Delta \varepsilon_1 / \Delta t = 1.0\text{E-}7$ 1/s and $\Delta \varepsilon_1 / \Delta t = 1.0\text{E-}6$ 1/s were adequate and the 2 tests (specimens 03, 05) were obviously conducted under fully saturated conditions (although B-values have not been determined for these 2 tests).

Table 4: Axial strain rate versus measured \bar{A} -values at 50% and 100% of the peak strength for the first test series

Axial strain rate (1/s)	Skempton's \overline{A} (50% peak strength)	Skempton's \overline{A} (100% peak strength)
1.0E-7	0.35	0.31
1.0E-6	0.27	0.27
1.0E-5	0.13	0.14
1.0E-4	0.06	0.05

For the second test series (19 specimens; beside S-samples also P-, X- and Z-samples) with a chosen axial strain rate of $\Delta \varepsilon_1 / \Delta t = 1.0$ E-6 1/s, Skempton's \overline{A} coefficient ranges between $\overline{A} = 0.01$ and $\overline{A} = 0.17$ (at 50% of the peak strength, Figure 14), which suggests that the chosen axial strain rate of $\Delta \varepsilon_1 / \Delta t = 1.0$ E-6 1/s is not adequate. This is in particular true for 6 tests (specimens P109, P115, X24, X25, X27, X30). Theses triaxial tests were conducted under fully saturated conditions according to the determined B-values (0.70 to 0.85 for the 2 P-samples, 0.65 to 0.90 for the 4 X-samples), but the loading rate was too fast according to the determined \overline{A} -values (0.07 to 0.08 for the 2 P-samples, 0.07 to 0.15 for the 4 X-samples).



Figure 14: Assessment of Skempton's \overline{A} . The blue lines indicate the theoretical threshold of \overline{A} for a typical range of elastic properties of Opalinus Clay.

The derivation of Skempton's \overline{A} coefficient to assess the adequacy of the axial strain rate is based on the assumption of an isotropic elastic material behaviour. In fact, Opalinus Clay behaves anisotropic, which may affect the \overline{A} -values. CU tests conducted at the Chair of Engineering Geology at ETH Zurich revealed \overline{A} -values for S-samples ranging between 0.36 and 0.58 ($\overline{A} = 0.47$ on average; 6 specimens) and for P-samples ranging between 0.12 and 0.22 ($\overline{A} = 0.17$ on average; 6 specimens). Although uncertainties remain, small values for Skempton's \overline{A} coefficient, in particular smaller than 0.15, suggest that the chosen axial strain rate is not adequate if full saturation of the specimens is proven by Skempton's *B* coefficient.

5.1.5 Conclusion

The triaxial tests reported in Jahns (2013) follow a consistent testing procedure, which is well described, documented and carefully applied. A clear separation between the saturation phase and the consolidation phase is, however, missing and makes the assessment of the entire test series significantly more difficult. The recommended testing procedure should comprise a saturation phase followed by a consolidation phase with an elevated confining stress and a constant backpressure.

Consistent assessment criteria were used to assess the completeness and correctness of all testing phases. There are only 2 triaxial tests that satisfy all the assessment criteria (expect that full saturation of the specimens was proven with the \bar{A} -value rather than with the B-value). The remaining 22 triaxial tests do not satisfy all the assessment criteria. As a consequence, the measured pore pressure response is unreliable and does not allow to use these test results for establishing effective strength properties. For these test results three scenarios were considered (see also Figure 15), which allow a qualitative assessment about the resulting effective strength properties:

 Full saturation of the specimen was established, but the consolidation phase was incomplete or/and the shearing phase was executed too fast (6 tests: specimens P109, P115, X24, X25, X27, X30). This results in excess pore pressure within the specimen during hydrostatic and differential loading. In this case, the measured pore pressure at the end-faces is smaller than the actual pore pressure and therefore the effective normal stresses are overestimated. As a consequence the strength of the tested Opalinus Clay is underestimated (orange data points in Figure 15; Vogelhuber 2007).

- 2) Full saturation of the specimen was not established, which makes a further assessment of both the consolidation phase and the shearing phase irrelevant (7 tests: specimens S05, S06, S07, P09, P10, P14, Z23). This means that capillary suction (i.e. negative pore pressure) arises within the partially saturated specimen during hydrostatic and differential loading. In this case, the measured pore pressure at the end-faces is higher than the actual pore pressure and therefore the effective normal stresses are underestimated. As a consequence the strength of the tested Opalinus Clay is overestimated (red data points in Figure 15; Vogelhuber, 2007).
- 3) The saturation state of the specimens cannot clearly be assessed (9 tests: specimens 01, 02, 04, S03, S102, S106, P13, Z19, Z21). Here, it is unclear if the strength of the tested Opalinus Clay is over-or underestimated (grey data points in Figure 15).



Figure 15: Assessment of the failure strength obtained by Jahns (2013).

For a reliable assessment of the effective friction angle and the effective cohesion only 2 tests are suitable (samples 03 and 05 with full saturation, adequate time for consolidation, adequate time to reach failure; green data points in Figure 15). Another 6 tests can be utilized (samples P109, P115, X24, X25, X27 and X30 with full saturation, adequate time for consolidation, inadequate time to reach failure) together with the above mentioned 2 tests for determining the undrained shear strength S_u and the undrained E-Modulus E_u .

According to Figure 15 (i.e. representation of data points in q-p' space for S- and P-samples on the left side, and for X- and Z-samples on the right side) the 2 adequate test results almost match with NAGRA's suggested effective strength properties of the rock matrix for Opalinus Clay deep. However, there are no reliable test results, which allow to examine the effective strength properties along the bedding planes which were suggested by NAGRA for Opalinus Clay deep. Data points, which lie clearly above the failure criterion in the q-p' space (continuous light blue line) overestimate the strength of the tested rock. Data points, which lie clearly below the failure criterion in the q-p' space (continuous light blue line) underestimate the strength of tested rock (or a clear statement whether the strength is over- or underestimated is not possible).

For effective strength properties along the bedding planes, X-samples (i.e. angle of 30° between load axis and bedding plane) and Z-samples (i.e. angle of 45° between load axis and bedding plane) were utilized. As outlined in section 4, NAGRA's practice for determining the effective friction angle ϕ' and the effective cohesion c' from these test results leads to an overestimation of strength for Z-samples. However, for X-samples the analysis by NAGRA is approximately accurate provided that the effective friction angle in the order of 30°.

Classification of the test results (quality levels, weighting factors)

The classification of the test quality in Jahns (2013) is based on measured B-values. Triaxial tests with low B-values are labelled with "Comment: B > 0.8" (6 tests) and were not considered for establishing effective strength properties. The classification of the test quality in Favero et al. (2013) is based on several aspects (i.e. proper sample saturation, equilibration during consolidation) and 4 quality levels from A to D were utilized. The highest quality A was not assigned. 9 tests were assigned with the second highest quality B. This assessment differs from the assessment in the present report, because in Favero et al. (2013) an axial strain rate $\Delta \varepsilon_1 / \Delta t$ of 1.0E-6 1/s during undrained shearing is not considered too fast. Despite the different assessment of the loading rates, the assessments are comparable. This means that the 9 tests with the quality B according to Favero et al. (2013) virtually agree with the above mentioned 8 tests (specimens 03, 05, 109, P115, X24, X25, X27, X30) which were carried out on fully saturated and complete consolidated specimens.

NAGRA (2014a) and Giger & Marschall (2014) utilized almost the same quality levels as suggested by Favero et al. (2013) and assigned weighting factors of 100% for quality A, 75% for quality B, 50% for quality C and 25% for quality D. The conducted regression analysis for establishing the effective friction angle and the effective cohesion accounts for the individual weighting factors of the different quality classes. This mathematization of uncertainties introduced by NAGRA is highly questionable. With this approach, a high number of unsuitable test results (with low quality) potentially overbalances a small number of suitable test results (with high quality), which is not acceptable.

5.2 Rummel & Weber 1999

5.2.1 General

The triaxial tests reported in Rummel & Weber (1999) have been performed on specimens with diameter D = 30mm and height H = 65mm. In total 59 Opalinus Clay specimens with a bedding plane orientation normal (19 S-samples), parallel (21 P-samples) and inclined (0 X-samples and 19 Z-samples) to the core axis were obtained by overcoring larger diameter cores taken from the borehole Benken in a depth range of 560 to 630m. Prior to overcoring these cores were stored in pressure vessels.

The 59 triaxial tests include specimens with natural water content before testing and dried/wetted specimens. Only specimens which were not dried or wetted prior to testing are discussed in this report⁷. According to Giger & Marschall (2014) only 18 triaxial tests were performed on specimens with natural water content before testing (5 S-samples, 7 P-samples, 6 Z-samples). The results of 14 out of 18 triaxial tests (4 S-samples, 5 P-samples, 5 Z-samples) were used by Giger & Marschall (2014) for establishing effective strength properties. The remaining 4 triaxial tests with a high confining stress of $\sigma_3 = 40$ MPa were considered irrelevant.

⁷ From Rummel & Weber (1999) it is not clear which specimens were dried or wetted. It is assumed that test results utilized in NAGRA (2014a) are solely triaxial tests on specimens with natural water content.

5.2.2 Water content, porosity and saturation degree

Assuming $\gamma_s = 27.1 \text{ kN/m}^3$ for the unit weight of the solids (this is the average value taken from Jahns (2013), the calculated porosity *n* ranges between 8.5 and 12.5% (with $\gamma_d = 23.7$ to 24.8 kN/m³ for the dry unit weight). The calculated saturation degree *S* ranges between 59 and 112% (with w = 3.1 to 4.8%) before testing and between 76 and 103% (with w = 2.7 to 4.5%) after testing. The water content was lower after testing than before testing for about half of the samples. This could be related to the use of AURALUX FE (i.e. a lubricant for metalworking) instead of water during the saturation phase.

Note that a saturation degree higher than 100% is physically impossible and may be related to the fact that the average value of $\gamma_s = 27.1 \text{ kN/m}^3$ taken from Jahns (2013) for the unit weight of the solids is not representative for each individual specimen. The calculated values for the saturation degree before and after testing are, therefore, to certain degree uncertain.

5.2.3 Triaxial testing procedure

The testing procedure comprises the following testing phases:

- 1) Saturation phase with a confining stress σ_3 in the range of 2.0 to 3.0 MPa and a backpressure u_0 typically in the range of 0.3 to 0.4 MPa (one test: 3.5 MPa). The utilized fluid was ARALUX FE.
- 2) Consolidation phase under hydrostatic loading conditions with a confining stress σ_3 of 5.0, 10.0 or 20.0 MPa.
- 3) Differential loading phase under undrained conditions with axial strain rate $\Delta \varepsilon_1 / \Delta t$ of 1.0E-6 1/s.

During axial displacement-controlled undrained shearing at constant radial stress, the pore pressure, the axial strain and the volumetric strain (derived from the oil volume loss and gain in the triaxial chamber) were continuously monitored.

Figure 16 shows the axial stress and the pore pressure versus time for a typical triaxial test in Rummel & Weber (1999). Note that this is the only example for this test series showing the entire testing procedure. It was considered representative for all triaxial tests. However, the representativeness of this example is not clear since the example was obviously taken from Rummel et al. (1999) without making any reference and the shown consolidation time is 13h instead of the reported consolidation time of 24h.



Figure 16: Development of axial stress and pore pressure with time during saturation, consolidation and shearing phase for the given triaxial test example. The pore pressure curves in the lower part of the figure show both the pore pressure measured up-stream and down-stream.

5.2.4 Assessment of the test phases

Saturation Phase criterion, C-Ia and C-Ib

The saturation degree before testing needs to be, in theory, at least S = 94% in order to facilitate full saturation with the applied backpressure of $u_0 = 0.3$ to 0.4 MPa) for most of the test series (Figure 17). No water supply is provided since AURALUX FE has been utilized. This means that for the majority of the triaxial tests by Rummel & Weber (1999) the chosen values of the backpressure were too small to facilitate full saturation of the specimen.

Skempton's *B* coefficient was not determined to confirm full saturation. The provided example of the complete testing procedure (Figure 16), which is considered representative for the test series according to Rummel & Weber (1999), allows to determine a B-value of less than 0.01 from the hydrostatic stress increase ($\Delta p = 7$ MPa) and the associated pore pressure increase ($\Delta u = 0.02$ MPa) as well as a \bar{A} -value of less than 0.01 from the differential stress increase ($\Delta q = 45$ MPa) and the associated pore pressure increase ($\Delta u = 0.02$ MPa) under undrained loading conditions. This shows that the specimen was not fully saturated prior to the consolidation and shearing phase.



Figure 17: Required back-pressure versus saturation degree for the cases of constant water supply and no water supply. The calculated saturation degrees before testing and utilized back-pressure are shown for all specimens tested in Rummel & Weber (1999).

Consolidation Phase, criterion C-IIa and C-IIb

The minimum theoretical consolidation time t_c for a specimen height of H = 65mm is $t_c = 2.3$ to 12h for $\sigma'_3 < 5$ MPa and $t_c = 29$ to 115h for $\sigma'_3 > 10$ MPa. The actual duration to fulfill consolidation of 24h is in a similar range as the minimum required consolidation time. Therefore, it might be adequate for lower values and inadequate for higher values of the effective confining stress. This assessment cannot be confirmed by the time dependent development of the volumetric strain and the change in water content during the consolidation phase since this is not reported in Rummel & Weber (1999).

The example shown in Figure 16 suggests that the consolidation phase was performed under undrained conditions without maintaining a constant backpressure on both end-faces of the specimen and without a continuous measurement of volumetric strain and change in water content. This example shows that after an increase of the hydrostatic stress and a related increase of the pore pressure, the pore pressure is decreasing upstream and increasing downstream with time. For such hydraulic boundary conditions the consolidation theory is not applicable to assess the theoretically required time for complete consolidation.

Failure Phase criterion, C-IIIa and C-IIIb

The minimum theoretical shearing time t_f for a specimen height of H = 65mm is $t_f = 4.7$ to 23h for $\sigma'_3 < 5$ MPa and $t_f = 59$ to 235h for $\sigma'_3 > 10$ MPa. The actual duration to reach failure ranges between 0.4 and 2.8h, which corresponds to an axial strain rate of $\Delta \varepsilon_1 / \Delta t = 1.0$ E-6 1/s. Therefore, the loading rate is too high irrespective of the assumptions made for the consolidation coefficient (which depends on the hydraulic conductivity and the bulk modulus of the rock) used for an estimation of the theoretically required time.

Only the electronic data provided by NAGRA in addition to Rummel & Weber (1999) contains data that show the upstream and downstream pore pressure magnitudes during undrained shearing for all triaxial tests. This allows to assess the pore pressure change associated with an increase in differential stress (i.e. Skempton's \bar{A} coefficient). The results are:

1) For specimen 6A1p the upstream pore pressure was 0.07 MPa and the downstream pore pressure was 0.70 MPa at the beginning of the shearing phase. During undrained shearing the pore pressure

increased to 1.00 MPa downstream, but remained constant at 0.07 MPa upstream. The substantial difference between these two values suggests that the consolidation phase was not completed and the shearing phase was executed too fast (i.e. the pore pressure in the specimen was not uniform).

- 2) For specimen 8A1z the measured pore pressure on the end-faces of the specimen was 3.49 MPa upstream and 3.53 MPa downstream. For this magnitude of pore pressure it is in principle possible to saturate the specimen (assuming that the backpressure is applied for a sufficient amount of time). However, full saturation of the specimen cannot be confirmed since the B-value is not reported. During undrained shearing both upstream and downstream pore pressures remain approximately constant, which means that the Ā-value is virtually zero. This result is either related to a too high loading rate or to the unsaturated state of the specimen.
- 3) For the remaining 12 tests (specimens 2A1s, 2A3s, 4A1s, 26A1s, 1A1p, 1A2p, 4A2p, 8A3p, 1A1z, 5A1z, 5A2z, 8A2z) the pore pressure at the beginning of the shearing phase ranges between 0.01 and 0.33 MPa upstream and between 0.07 and 0.39 MPa downstream. During undrained shearing the pore pressure remained approximately constant for all these tests corresponding to Ā-values of virtually zero.

5.2.5 Conclusion

The triaxial tests reported in Rummel & Weber (1999) are incompletely described and documented. They do not represent a state-of-the-art testing procedure for CU tests for establishing effective strength properties. Major issues are:

- 1) The pore pressure change during undrained shearing is not reported, and the magnitudes of pore pressure applied during the saturation and the consolidation phase contradict with actually used values according to the electronic data provided by NAGRA.
- 2) The provided example for the development of the axial stress and the pore pressure during all test phases of a typical triaxial test was obviously taken from an earlier report (without reference) and is therefore irrelevant.
- 3) The saturation procedure is most likely inadequate due to the too low backpressure and the use of ARALUX FE (i.e. a lubricant for metalworking) instead of a water filled drainage system. In addition, the state of saturation of the specimens was not evaluated by determining Skempton's *B* coefficient.
- 4) The testing phase prior to the shearing phase was labeled "consolidation phase" but seems to be performed under undrained rather than drained conditions (as per definition for the consolidation phase). In this case the minimum required consolidation time derived from the consolidation theory is not applicable. In addition, the volumetric strain and the change in water content is not reported and cannot be utilized to confirm complete equilibration of pore pressure in the specimens.
- 5) The utilized axial strain rate during undrained shearing was most probably too high. Confirmation of this assessment by determining Skempton's \overline{A} coefficient is not possible since values of virtually zero (i.e. almost no pore pressure response during differential loading) are related to the unsaturated state of the specimens.



Figure 18: Assessment of the failure strength obtained by Rummel & Weber (1999).

As a consequence of the unsaturated state of the specimens, at least 13 of 14 test results overestimate the strength of the tested Opalinus Clay (see also Figure 18). Based on the above assessment none of the triaxial tests is suitable for a reliable assessment of the effective friction angle and the effective cohesion. In addition, the undrained shear strength S_u and the undrained E-Modulus E_u cannot be reliably determined.

According to Figure 18 (i.e. representation of data points in q-p' space for S- and P-samples on the left side, and for X- and Z-samples on the right side) no reliable test results are available, which allow the determination of the effective strength properties for the rock matrix and/or along the bedding planes for Opalinus Clay deep as suggested by NAGRA. For effective strength properties along the bedding planes only Z-samples (i.e. angle of 45° between load axis and bedding plane) were utilized. As a consequence, the strength of tested rock is overestimated because of 2 reasons: 1) an inappropriate testing procedure (strength is overestimated due to capillary suction), and 2) an inappropriate determination of the effective strength properties from the test results, because the tests were conducted on specimens where the bedding plane orientation was not in the most unfavourable orientation with respect to the specimen long axis. It is possible to quantify the magnitude of overestimation for the second case, but not for the first case.

Classification of the test results (quality levels, weighting factors)

NAGRA assigned a quality C for the entire test series which corresponds to a weighting factor of 50% for the regression analysis. This contradicts with the fact that full saturation of the specimens could not be confirmed (B-values were not determined), and various criteria suggest that a fully saturated state could most probably not be established.

A weighting factor of 50% suggests that two tests with partly saturated specimens are equally valuable than one test that satisfies all assessment criteria (i.e. full saturation of the specimen and accurate control of pore pressure during hydrostatic and differential loading). This illustrates that the chosen approach of NAGRA is not feasible. It is not possible to establish the effective strength properties for Opalinus Clay with the tests reported in Rummel & Weber (1999) because the specimens are not in a fully saturated state and therefore the effective normal stresses during both the consolidation phase and the shearing phase are unknown. This cannot be compensated by a reduced weighting factor of 50% instead of 100%.

5.3 Test series with specimens from the Mont Terri URL

5.3.1 Overview

For establishing the effective strength properties of Opalinus Clay at shallow depth (i.e. at a depth up to 400m) in total 7 test series were considered in NAGRA (2014a) and Giger & Marschall (2014). These test series utilize Opalinus Clay specimens taken at the Mont Terri Underground Research Laboratory.

Two test series (Schnier & Stührenberg 2007, Popp & Salzer 2006) were performed as classical rock mechanics triaxial tests (i.e. without sample saturation and control or monitoring of pore water pressure). Another 3 test series (Rummel & Weber 2004, Rummel et al. 1999, Olalla et al. 1999) are reported as CU tests. Another 2 test series (Jahns 2010, Jahns 2007) are reported as CD tests.

5.3.2 Classical rock mechanics triaxial tests

Schnier & Stührenberg (2007) performed in total 38 triaxial test without any pore pressure control (i.e. the testing device without pore fluid system). They were executed either at room temperature (18 of 21 tests with results) or at 80° Celsius (all 17 tests with results). Some of the test results were executed not as single- but as multi-stage triaxial tests, which allowed to establish peak and residual strength in the first stage (14 test results for establishing effective strength parameters according to Giger & Marschall 2014), and residual strength in the subsequent stages (40 test results according to Giger & Marschall 2014). It was stated by Schnier & Stührenberg (2007) that "due to unequal storage times between drilling and testing the conditions of the samples were different" or "because of the long storage time and insufficient storage conditions samples started to fall apart on bedding planes after some weeks or months". The observation that longer storage of the samples resulted in lower water content leads to the conclusion that the specimens were most likely not saturated prior to testing. It is also stated by Schnier & Stührenberg (2007) that the triaxial tests were executed "with unsaturated specimens".

Popp & Salzer (2006) performed in total 11 triaxial test without any pore pressure control (i.e. the testing device without pore fluid system). According to Giger & Marschall (2014) only 8 test results were considered for establishing effective strength parameters. 2 tests were rejected without obvious reason and 1 test was most probably considered irrelevant because of the very high confining stress of $\sigma_3 > 50$ MPa.

Both test series were conducted without any control of pore pressure, meaning that the testing procedure does not include a saturation and a consolidation phase. In addition, the shearing phase was executed within few minutes according to the chosen axial strain rate of $\Delta \varepsilon_1 / \Delta t = 1.0\text{E-5}$ 1/s. The tests reported in Schnier & Stührenberg (2007) were performed on specimens with a diameter D = 100mm and a length H = 200 to 250mm, and the tests reported in Popp & Salzer (2006) on specimens with diameter D = 80mm and length H = 160mm. The specimens were most likely not saturated during testing and therefore the effective normal stresses remain unknown. It is clear that such tests cannot be used for a reliable assessment of the effective friction angle and the effective cohesion. All test results overestimate the strength of the tested Opalinus Clay. NAGRA used these test results in their analysis of the effective strength properties with a quality D and a weighting factor of 25%. Therefore, NAGRA assessed the 22 tests by Schnier & Stührenberg (2007) and Popp & Salzer (2006) without any pore pressure control on partially saturated specimens as equally valuable as the 8 tests by Jahns (2013) with pore pressure control on fully saturated specimens (i.e. quality B, weighting factor 75%). This is not reproducible.

5.3.3 Consolidated undrained triaxial tests

Rummel & Weber (2004) performed in total 36 triaxial tests on specimens with natural water content before testing. Only 30 test results (10 S-samples, 10 P-samples, 10 Z-samples) were reported in Giger & Marschall (2014) for establishing effective strength parameters. According to Rummel & Weber (2004)

the testing procedure includes a consolidation phase followed by a deformation phase. The consolidation phase was conducted under hydrostatic loading conditions at a confining stress of 5.0, 10.0 or 15.0 MPa and a backpressure of 0.3 MPa that was maintained for about 24h. The utilized fluid was ARALUX FE. For the deformation phase an axial strain rate $\Delta \varepsilon_1 / \Delta t$ of 1.0E-6 1/s was chosen. The tested specimens have a diameter of D = 30mm and a height of H = 65mm. The description of the testing procedure is partly incomplete and partly not reproducible. From the data provided in Rummel & Weber (2004) it remains unknown if the consolidation phase was performed under drained or undrained conditions. Further, the development of the axial and radial stress, the pore pressure, the volumetric strain and the change in water content as a function of the axial strain are only for one test completely documented. The shown testing procedure for this test suggests an increase of the axial and radial stress (by several MPa) at the end of the consolidation phase. This contradicts the general aim of the consolidation phase, which is performed to establish a uniform pore pressure field in the specimen prior to the shearing phase. This issue affects the reliability of the whole test series. An additional issue is related to the continuously monitored change in pore pressure during undrained shearing. The pore pressure differs for at least 3 tests substantially (by 0.4 to 1.1 MPa) between upstream and downstream.

Rummel et al. (1999) performed in total 34 triaxial tests on specimens which were artificially dried or wetted before testing. The tested specimens have a diameter of D = 30mm and a height of H = 60mm. Only 10 test results (2 S-samples, 5 P-samples, 3 Z-samples) were reported in Giger & Marschall (2014) for establishing effective strength parameters. The testing procedure according to Rummel et al. (1999) is similar to the procedure used in Rummel & Weber (1999). The saturation phase was conducted using a confining stress of 2.0 to 3.0 MPa and a backpressure of 0.3 to 0.4 MPa that was applied for a period of 2 to 20h. The consolidation phase was conducted with a confining stress of 10.0 MPa over a period of 12 to 65h. The utilized fluid was ARALUX FE. The differential loading phase was conducted under undrained conditions with an axial strain rate $\Delta \varepsilon_1/\Delta t$ of 1.0E-6 1/s.

For this test series the conclusions are the same as for the tests reported in Rummel & Weber (1999). The specimens were most likely not saturated during testing and therefore the effective normal stresses remain unknown (i.e. no adequate backpressure process, no B-values in the saturation phase to confirm the saturated state of the specimens, very low \bar{A} -values in the shearing phase probably associated with the unsaturated state of the specimens). This suggests that all test results overestimate the strength of the tested Opalinus Clay, and cannot be used for a reliable assessment of the effective friction angle and the effective cohesion. Both test series were assigned by NAGRA with a quality C and a weighting factor of 50%. This is an identical classification as for the tests reported in Rummel & Weber (1999) and therefore consistent. However, it is not reproducible that the 40 tests by Rummel & Weber (2004) and Rummel et al. (1999) are, according to the assessment of NAGRA, considered to be by a factor of 3 to 4 more valuable than the 8 tests by Jahns (2013) with control of pore pressure on fully saturated specimens (i.e. quality B, weighting factor 75%). In contrast to Jahns (2013), Skempton's *B* coefficient was not determined, and the backpressure was at least 10 times smaller.

Ollala et al. (1999) performed in total 18 triaxial tests, but only 12 triaxial tests are documented in the corresponding report. According to Olalla et al. (1999) 2 tests on P-samples are "triaxial compression tests with backpressure" and 10 tests (8 on P-samples, and 2 on special samples with an angle of 60° between the load axis and the bedding plane) are "triaxial compression tests without backpressure". It is not clear why tests with a backpressure of up to 4.8 MPa and 6.0 MPa respectively are labeled "without backpressure" whereas tests with a tenfold lower backpressure of 0.6 MPa are labeled "with backpressure". In Giger & Marschall (2014) only the 2 tests labeled "with backpressure" are considered for establishing effective strength parameters. NAGRA used these test results in their analysis of the effective strength properties with a quality D and a weighting factor of 25%.

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The description of the testing procedure in Olalla et al. (1999) is hardly reproducible and many details are not reported. The reporting of the test series is thus incomplete and also not consistent with the test results (i.e. tests "without backpressure" and tests "with backpressure"). This complicates the assessment of this test series. It is not clear if the consolidation phase was completed, if the shearing phase was executed slowly enough, and if the specimens were fully saturated prior to consolidation and shearing. Undrained shearing was executed with a deformation rate of 0.005 %/min (approximately $\Delta \varepsilon_1 / \Delta t = 1.0\text{E-6 l/s}$) for the tests "without backpressure" or 0.002 %/min (approximately $\Delta \varepsilon_1 / \Delta t = 5.0$ E-7 1/s) for the tests "with backpressure". The tests were performed with specimens of diameter D = 70 mm and length H = 140 to 150mm. B-values were not determined. For 7 of 12 specimens the backpressure is smaller than 0.1 MPa and the A-values was virtually zero, which suggests that the specimens are not saturated and thus the corresponding test results overestimate the strength of the tested Opalinus Clay. For 5 of 12 specimens, however, the backpressure was considerable higher and ranges between 0.3 and 6.0 MPa. A-values calculated for these triaxial tests range between 0.07 and 0.19 for tests "without backpressure" (i.e. backpressure of 0.3 to 6.0 MPa) and between 0.08 and 0.25 for tests "with backpressure" (i.e. backpressure of 0.6 MPa). The saturation state of the specimens cannot be clearly assessed. A reliable conclusion regarding the tests reported in Olalla et al. (1999) is therefore not possible.

5.3.4 Consolidated drained triaxial tests

Jahns (2010) performed in total 9 triaxial tests with the purpose "to obtain reliable data of deformation properties of intact core material under drained boundary conditions". All test results (6 P-samples, 3 Zsamples) were reported in Giger & Marschall (2014) for establishing effective strength parameters. The testing procedure consists of a consolidation phase (with confining stresses of 3.0, 6.0 or 9.0 MPa and a backpressure of 0.6 MPa that was maintained for 40 to 90h) and a deformation phase (with axial strain rate of 1.0E-6, 5.0E-7 or 1.0E-7 1/s for drained shearing conditions). The utilized fluid in the drainage system was brine. For a first test series (3 P-samples) the deformation rate was varied, but the confining stress was the same (i.e. 6 MPa) for all tests. For a second the test series (3 P-samples, 3 Z-samples) the confining stress was varied, but the loading rate was the same (i.e. 5.0E-7 1/s). Similar to the tests reported in Jahns (2013), the saturation phase and the consolidation phase were largely performed in parallel. The chosen backpressure of 0.6 MPa requires, in theory, a minimum saturation degree of S =88% before testing in order to facilitate full specimen saturation (if a continuous supply of water is provided and the saturation phase is sufficiently long). On the basis of the experimentally determined water content of w = 2.8 to 6.1% (mean value w = 4.7%) before testing, a saturation degree of S = 41 to 89% (mean value S = 67%) before testing can be derived using basic physical properties (porosity n =16%, unit weight of the solids $\gamma_s = 27.1 \text{ kN/m}^3$ proposed for Opalinus Clay shallow in Giger & Marschall (2014). This suggests that the chosen value of 0.6 MPa for the backpressure was too small to saturate the specimens. The minimum theoretical time to fulfill consolidation is considerably shorter (0.5 to 23h instead of 1.4 to 68h) than used for the triaxial tests by Jahns (2013). This is related to the reduced specimen height of H = 35mm instead of H = 60mm (with the same aspect ratio of 1:2). The utilized consolidation time of 40 to 90h is therefore long enough. However, a significant amount of brine entered into the specimens during the consolidation phase indicating a swelling process rather than a consolidation process.

The minimum theoretical time to reach failure is longer (1.9 to 96h instead of 1.4 to 68h) than used for the triaxial tests by Jahns (2013). This is related to the change in hydraulic boundary conditions (from undrained to drained), which has a larger effect on the required shearing time than the reduction of the specimen height. It is therefore not clear if the axial strain rate of $\Delta \varepsilon_1 / \Delta t = 5.0\text{E-7}$ 1/s selected for the second part of the test series is small enough.

Jahns (2007) performed in total 23 triaxial tests. All test results (8 S-samples, 8 P-samples, 7 Z-samples) were used in Giger & Marschall (2014) for establishing effective strength parameters. The testing procedure is comparable to the procedure used by Jahns (2010). It consists of a consolidation phase (with confining stresses of 4.0, 6.0 or 10.0 MPa and a backpressure of 0.5 MPa that was maintained "overnight") followed by a deformation phase (with an axial strain rate of 1.0E-6 1/s for drained shearing conditions). The utilized fluid was brine. The chosen backpressure of 0.5 MPa requires, in theory, a minimum saturation degree of S = 90% before testing in order to facilitate full specimen saturation (if a continuous supply of water is provided and the saturation phase is sufficiently long). On the basis of the experimentally determined water content of w = 5.8 to 6.3% (mean value w = 6.0%) before testing, a saturation degree of S = 83 to 90% (mean value S = 85%) before testing can be derived using the basic physical properties (porosity n = 16%, unit weight of the solids $\gamma_s = 27.1 \text{ kN/m}^3$) proposed for Opalinus Clay shallow in Giger & Marschall (2014). It was stated by Jahns (2007) that the backpressure "was applied with 0.5 MPa after the consolidation was finished", and therefore immediately before the shearing phase. For such a testing procedure the saturation process is clearly too short to saturate the specimens irrespective of the applied backpressure. The reporting of the consolidation phase is incomplete (no clear statement about the consolidation time, no documentation of volumetric strain or change in water content). Based on the CU tests reported in Jahns (2013) it was shown that the axial strain rate of $\Delta \varepsilon_1 / \Delta t = 1.0 \text{E-6 1/s}$ is too high for a specimen height of H = 60 mm. Thus, for the CD tests reported in Jahns (2010) the same deformation rate for the same specimen dimensions must be significantly too high. In theory, drained shearing has to be executed about 4 times slower than undrained shearing.

The conclusion is quite the same as for the tests reported in Rummel & Weber (1999). The specimens were most likely not saturated during testing and therefore the effective normal stresses remain unknown (i.e. no adequate backpressure phase, no determination of B-values in the saturation phase to confirm the saturated state of the specimens). This suggests that all test results overestimate the strength of the tested Opalinus Clay, and cannot be used for a reliable assessment of the effective friction angle and the effective cohesion. NAGRA assigned the tests by Jahns (2010) with a quality B (i.e. a weighting factor of 75%) and those by Jahns (2007) with a quality C (i.e. a weighting factor of 50%). This classification by NAGRA was obviously done on the basis of different loading rates and different specimen dimensions. However, the key parameter for assessing the test results is the saturation state of the specimens. It is not reproducible that the 9 tests by Jahns (2010) are, according to the assessment of NAGRA, equally valuable than the 8 tests by Jahns (2013) with control of pore pressure on fully saturated specimens (i.e. quality B, weighting factor 75%). In contrast to Jahns (2013), Skempton's *B* coefficient was not determined and the backpressure was at least 5 times smaller.

5.4 Conclusion regarding effective strength properties

The assessment of all triaxial test results used by NAGRA for establishing effective strength properties for intact Opalinus Clay for a depth below 400m (Opalinus Clay shallow) and for a depth range between 400 and 900m (Opalinus Clay deep) was based on the application of six assessment criteria in a consistent way. Three assessment criteria are based on established theoretical considerations, and three assessment criteria are based on the reported test results.

The test results from Jahns (2013) and Rummel & Weber (1999) were utilized by NAGRA for establishing the effective strength parameters in the case of Opalinus Clay deep. The test results from Jahns (2013) were assessed by Favero et al. (2013) using a strict quality assessment scheme that focuses on a proper sample saturation and pore pressure equilibration during consolidation. The classification of Favero et al. (2013) is basically in agreement with the assessment in this report. The main difference is the assessment of the loading rate during undrained shearing, which was not the primary focus of the quality assessment scheme of Favero et al. (2013). In this report it was shown that the loading rate used

for the majority of triaxial tests in Jahns (2013) was most probably too high to obtain reliable values for the pore pressure at failure. Only 8 out of the 24 specimens can be considered saturated, from which 6 specimens were most probably loaded too fast and the resulting strength is underestimated (Figure 19). In Giger & Marschall (2014) the data points from Jahns (2013) were used following virtually the same quality levels A to D as suggested by Favero et al. (2013). In addition, NAGRA assigned different weighting factors for different quality levels (Figure 19). The analysis of the test results from Rummel & Weber (1999) reveals that specimen saturation was not established and all specimens were most probably loaded too fast. Thus, none of the 14 specimens can be used to determine effective strength properties (Figure 19). In Giger & Marschall (2014) the data points from Rummel & Weber (1999) were assigned with a quality level C (Giger & Marschall 2014) and a weighing factor of 50% (i.e. two of these tests have the same weight as one test fulfilling all assessment criteria).

				est?	AIL A		all C	J.P	100%	25010	50%	25010	0%	etest	dere	st. erest	- po	100% 0%
		N	uniber	Jairy le	ality le	Astry le	Jainty le	eighting	alehting	eighting	eintheine	ziehtine An	proprio	ensth	st. eneth	ostater	eighting	218htines
deeb	Jahns 2013	24		8	6	5		8	6	5	5	2	6	7	9	2	22	
OFA	Rummel & Weber 1999	14			14				14					13	1		14	
Ī	Jahns 2010	9		9				9						9			9	
	Jahns 2007	23			23				23					23			23	
	Schnier & Stührenberg 2007	14				14				14				14			14	
	Popp & Salzer 2006	8				8				8				8			8	
	Rummel & Weber 2004	30			30				30					29	1		30	
	Rummel et al. 1999	10			10				10					10			10	
	Olalla et al. 1999	12			2				2		10			7	5		12	
Assessment Assessment according to NAB 14-01 by Amann/Vogelhuber										t huber								

Figure 19: Quality assessment and weightings given in NAB 14-01 and in this report for test series used by NAGRA to establish effective strength properties for Opalinus Clay deep and shallow.

Similar to the approach for establishing the effective strength parameters for the depth range between 400 and 900m, triaxial test results used to establish effective strength parameter for a depth lower than 400m (i.e. in the case of Opalinus Clay shallow) were classified and weighted. For tests series without any pore pressure control (i.e. Schnier & Stührenberg 2007, Popp & Salzer 2006) a quality level D and a weighing factor of 25% was assigned according to Giger & Marschall (2014), even though the pore pressure at failure is unknown. For the other test series (i.e. Jahns 2010, Jahns 2007, Rummel & Weber 2004, Rummel et al. 1999, Olalla et al. 1999) utilized by NAGRA, the assessment in this report shows that a saturated state of the specimens was not established and in many cases the specimens were loaded too fast.

A comparison of the quality levels assigned by NAGRA for the different test series for Opalinus Clay deep and Opalinus Clay shallow, shows major inconsistencies in many cases. Furthermore, NAGRA's concept of mathematizing uncertainties by introducing weighting factors is not acceptable. With such an approach, a large amount of inadequate tests (with low quality) overbalances individual adequate tests

(with high quality) in a regression analysis through the weighted data points. This approach may lead to wrong conclusions. Full saturation was not established for most of the specimens according the assessment in this report. This means that the majority of test results tend to overestimate the strength of tested Opalinus Clay (Figure 19). However, the magnitude of overestimation cannot be quantified. For a quantitative evaluation of the strength of the tested Opalinus Clay, only two reliable triaxial tests exist, which is a too small database for establishing effective strength properties.

6 Assessment of undrained shear strength properties

The data set used by NAGRA is shown in Figure 2a. It contains data points stemming from triaxial tests referred to as CU tests (Jahns 2013, Rummel & Weber 2004, Rummel & Weber 1999, Rummel et al. 1999, Olalla et al. 1999) or CD tests (Jahns 2010) on samples which were either dried/wetted before testing, conducted at the water content after sample storage and sample preparation (i.e. use of ARALUX FE in the pore fluid system) or conducted at an elevated water content due to partial or full saturation in the backpressure phase (i.e. use of water in the pore fluid system). Using these data points for interpreting them as UU tests as well as for establishing the undrained shear strength S_u is not appropriate due to the following reasons:

- 1) For determining the undrained shear strength S_u the pore space needs to be saturated with pore water and test results from dried samples cannot be used for establishing a relation between the undrained shear strength S_u and the water content representative for the in-situ conditions. This reduces the data set shown in Figure 2a to data points from triaxial tests with a water content larger than at least 3.1% or 3.8% (i.e. 3.1% or 3.8% are the lower limits of the water content after sample dismantling from storage reported in Rummel & Weber 1999 and Jahns 2013).
- 2) It has been shown in the previous section 5 that for the majority of the triaxial tests a fully saturated state could not be re-established. Capillary suction must be expected which influences the results for the undrained shear strength S_u . Tests on partially saturated samples may overestimate the shear strength under undrained conditions and are not representative for the in-situ effective stress conditions.
- 3) The determination of the undrained shear strength S_u is only reasonable when considering the results of CU tests (i.e. with constant water content during undrained shearing) but certainly not for CD tests (i.e. with varying water content during drained shearing). Therefore, the integration of the test results from Jahns (2010) in Figure 2a is not reproducible.

6.1 Consistency with effective strength properties

As shown in the previous section 5, the effective strength properties suggested by NAGRA (Giger & Marschall 2014) tend to overestimate the actual strength. However, for consistency reasons the suggested effective strength properties should yield in calculated values for the undrained shear strength S_u that are similar to those derived from the data set shown in Figure 2a. Under undrained conditions with B = 1 (i.e. the volume of the rock remains constant during loading) the differential stress at failure of a rock can be calculated from the effective friction angle ϕ' and the effective cohesion c' for a given initial effective stress σ'_0 using the following equation (see Appendix A1 for the assumptions and the derivation of the equation):

$$\sigma_{1f} - \sigma_{3f} = \frac{3(m-1)}{m+2}\sigma'_0 + \frac{3}{m+2}f_c$$

with the coefficients $m = (1 + sin\phi')/(1 - sin\phi')$ and $f_c = 2c' cos\phi'/(1 - sin\phi')$ related to the Mohr-Coulomb failure criterion. Because the differential stress at failure does not depend on the confining stress during UU tests, the $\phi_u = 0^\circ$ concept is valid and the cohesion c_u can be referred to as the undrained shear strength S_u :

$$S_u = \left(\frac{3(m-1)}{m+2}\sigma'_0 + \frac{3}{m+2}f_c\right)/2$$

For a water content of 3.6-4.3%, expected at a depth of 900m (NAGRA 2014a), the suggested S_u ranges between 21.4 and 26.4 MPa for the matrix and between 11.5 and 15.0 MPa for the bedding planes. For a lower depth (i.e. 500m) and a higher water content (i.e. 3.8-5.2%) the suggested S_u ranges between 18.1 and 25.2 MPa for the matrix and between 9.4 and 14.1 MPa for the bedding planes. Table 5 shows both, S_u values suggested by NAGRA derived from the data shown in Figure 2 and S_u values calculated from NAGRA's recommended effective friction angle ϕ' and effective cohesion c' for the case of Opalinus clay deep.

Table 5: Su suggested by NAGRA compared to Su values calculated from effective strength properties suggested by NAGRA.

	Su, OPA deep	at 500m (MPa)	Su, OPA deep at 900m (MPa)				
	suggested	calculated	suggested	calculated			
Matrix	18.1-25.2	12.8	21.4-26.4	16.3			
Bedding	9.4-14.1	7.7	11.5-15.0	10.5			

The comparison in Table 5 reveals major inconsistencies between suggested and calculated values for the undrained shear strength S_u . For the matrix strength the calculated S_u values are 1.4 to 2.0 (for a depth of 500m) and 1.3 to 1.6 (for a depth of 900m) times lower than the suggested S_u values by Nagra. For the bedding strength the calculated S_u values are 1.2 to 1.8 (for a depth of 500m) and 1.1 to 1.4 (for a depth of 900m) times lower than the values suggested by NAGRA. These inconsistencies are most likely associated with the fact that the S_u values suggested by NAGRA (2014a) were derived from test results for which the majority of specimens was not fully saturated during undrained shearing. However, the use of fully saturated specimens is a critical precondition for obtaining reliable S_u values.

6.2 Consistency with data from the literature and NAB 13-18

For only 8 CU tests reported by Jahns (2013) full saturation of the specimens as well as a complete consolidation phase could most probably be achieved and the corresponding test results (2 tests - specimens 03 and 05 - probably with a slow enough loading rate during the shearing phase, 6 tests - specimens P109, P115, X24, X25, X27 and X30 - probably with a too fast loading rate during the shearing phase) can be used for establishing the undrained shear strength S_u . These test results were analyzed together with data from Aristorenas (1992) on specimens obtained from two boreholes near the Wisenberg Tunnel and data from Wild et al. (2015) on specimens obtained from the Mont Terri URL (S-samples and P-samples). Figure 20a shows the S_u values versus the effective confining stress after consolidation (which agrees with the effective confining stress before shearing) for P-, S- and X-samples obtained from the above studies. Figure 20b shows a linear regression analysis through the available data points separately for P- and S-samples, which relate S_u values of the two cases of bedding plane orientation (P-samples with load axis parallel to bedding, S-samples with load axis normal to bedding) to the effective confining stress. Regarding the X-samples, the scatter in the available data points does not allow to establish a similar relationship.

 S_u values calculated from the slope of the regression analysis through results from all P- and S-samples (see Figure 20c) suggest an undrained shear strength of $S_u = 10.9$ MPa at a depth of 500m and $S_u = 18.7$ MPa at a depth of 900m. These S_u values reasonably agree with S_u values calculated from the

effective strength properties suggested by NAGRA, but are significantly lower than S_u values derived by NAGRA from the triaxial test results (according to Table 5).



Figure 20: a) Su values from the literature and Jahns (2013) versus effective confining stress after consolidation; b) linear regression analysis through data points obtained from P- and S-samples; c) linear regression analysis through all data points (i.e. P- and S-samples).

6.3 Conclusion regarding undrained shear strength

For establishing reliable values of the undrained shear strength S_u during undrained shearing (assuming unconsolidated undrained testing conditions), the specimens have to be saturated. This was not the case for the majority of the utilized data shown in Figure 2a and thus the S_u values suggested by NAGRA tend to overestimate the actual shear strength under undrained conditions. In addition, these S_u values are not consistent with S_u values calculated from the effective strength properties suggested by NAGRA (for the condition of zero volumetric strain). The suggested S_u values are between 1.1 (Matrix, 500m) and 2.0 (Bedding, 900m) times larger than the calculated S_u values. The analysis of valid test results from Jahns (2013) as well as reliable data points from the literature shows that a relation between the undrained shear strength and the effective confining stress after consolidation can be established. In this way, the estimated S_u values are in agreement with S_u values calculated from the effective strength properties suggested by NAGRA.

7 Assessment of the elastic properties

As shown in the previous sections, for the majority of the triaxial tests the specimens were not saturated or saturation could not be demonstrated. For the case of Opalinus Clay deep only 8 CU tests reported by Jahns (2013) were probably conducted on saturated specimens with completeness of the consolidation phase. Therefore, the corresponding triaxial test results (2 S-samples, 2 P-samples and 4 X-samples) can be used to define reliable values for the undrained E-Modulus. According to Giger & Marschall (2014) the suggested values for analytical or numerical analyses are $E_u = 9/18$ GPa (normal/parallel to bedding) and were derived from unloading/reloading cycles on S- and P-samples. For the 2 saturated S-samples (specimens 03 and 05) values of $E_u = 8.8$ and 8.9 GPa representative for an effective confining stress of 13.0 MPa in both cases were identified by Jahns (2013). For the 2 saturated P-samples (samples P109 and P115) values of $E_u = 15.4$ and 13.8 GPa with an effective confining stress of 7.6 and 4.6 MPa respectively were identified by Jahns (2013). Therefore, the values suggested by NAGRA are in reasonable agreement with laboratory results for both S- and P-samples when considering that the undrained E-Modulus for unloading/reloading cycles increases with increasing effective confining stress. For the case of Opalinus Clay shallow none of the triaxial test results analyzed by NAGRA allows to define reliable values for the undrained E-Modulus since probably none of the specimens was fully saturated.

The drained E-Modulus was derived from oedometer tests and a long term permeameter test (NAGRA 2014a, only S-samples). According to Giger & Marschall (2014) the suggested values for analytical or numerical analyses are E = 2 GPa for Opalinus Clay shallow and E = 4 GPa for Opalinus Clay deep irrespective of the orientation of the load axis (normal/parallel to bedding). For Opalinus Clay shallow, the relevant effective confining stress is in the range of $\sigma'_3 = 1.0$ to 6.0 MPa. The data basis from the Mont Terri URL shown in Giger & Marschall (2014) suggests that for the relevant effective confining stress range a drained E-Modulus of E = 0.2 to 2.3 GPa was determined. A value of 2 GPa for Opalinus Clay shallow, as suggested by NAGRA, is on the upper limit of the experimental data. For Opalinus Clay deep, the relevant effective confining stress is in the range of $\sigma'_3 = 6.0$ to 14.0 MPa. The data basis from the Mont Terri URL shown in Giger & Marschall (2014) suggests a drained E-Modulus of E = 0.7 to 5.2 GPa for the relevant effective confining stress range. However, the oedometer tests on samples from the borehole Schlattingen by Ferrari et al. (2012) and the permeameter test on a sample from the borehole Benken by Horseman & Harrington (2000) are considered to be more relevant for the case of Opalinus Clay deep. These data suggest a drained E-Modulus obtained for unloading/reloading cycles which is strongly dependent on the effective confining stress and increases from E = 2.4 GPa for approximately $\sigma'_3 = 6.0$ MPa to E = 8.0 GPa for approximately $\sigma'_3 = 14.0$ MPa (with evaluation of the oedometer tests according to Favero et al. 2013). The value suggested by NAGRA, for the drained E-Modulus for Opalinus Clay deep (E = 4 GPa) is within the range of experimental data. However, for the depth range between 500 and 900m (Opalinus Clay deep) the data suggest a major increase of the E-Modulus with increasing effective confinement (i.e. from 2.4 GPa to 8 GPa). This may have a relevant effect on numerical and analytical calculations which address the maximum depth below ground surface.

As discussed in section 3.1 the simplification introduced by NAGRA for the geomechanical behavior ignores plastic deformations in the pre-failure region. As a consequence, numerical calculations based on a linear-elastic model with elastic properties obtained from unloading/reloading may underestimate the strain at failure. This was not considered by NAGRA for the recommended values for numerical and analytical models.

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Appendix A1 Derivation of undrained shear strength of fully saturated specimen

For the following derivation it is assumed that Terzaghi's principle of effective stress is valid. The initial stress condition is assumed to be hydrostatic and can be expressed by the total stress σ_0 and the pore pressure u_0 (i.e., effective stress $\sigma'_0 = \sigma_0 - u_0$).

During undrained test conditions the water content remains constant (i.e. no water can flow into or out of the specimen) and the pore pressure changes as the specimen is loaded. Assuming that the compressibility of the solid constituent is negligible, a change in volume of the saturated specimen is only possible if the compressibility of the water is considered:

$$\varepsilon_{vol} = \frac{n}{K_w} (u - u_0)$$

where n is the porosity and K_w the bulk modulus of water. The pore space as well as the pore water are considered to be fully connected.

Considering the dependency of Skempton's pore pressure coefficient *B* on the porosity, the bulk modulus of the specimen *K* and the bulk modulus of water K_w (i.e. $B = 1/(1 + nK/K_w)$), the volumetric strain can be expressed as follows:

$$\varepsilon_{vol} = \frac{1-B}{KB}(u-u_0)$$

For a given porosity, Skempton's pore pressure coefficient B can take values between 0 and 1 depending on the ratio between the bulk modulus of the specimen and the bulk modulus of the water. If the compressibility of the pore water is small compared to the compressibility of the rock specimen, B would be close to unity. Therefore, the volume of the rock would not change during undrained loading.

For a standard triaxial compression test the maximum principal stress equals the axial stress σ_{1f} and the minimum principal stress is equal to the radial stress σ_{3f} .

Assuming a linear elastic, ideally plastic as well as isotropic material behavior the volumetric strain can be expressed as the sum of the elastic part (ε_{vol}^{EL}) and the plastic part (ε_{vol}^{PL}) as follows:

$$\varepsilon_{vol} = \frac{1}{3K} \left(\left(\sigma'_{1f} - \sigma'_{0} \right) + 2\left(\sigma'_{3f} - \sigma'_{0} \right) \right) + \varepsilon_{vol}^{PL}$$

This expression leads to the following relationship between a change in pore pressure and a change in axial and radial stresses:

$$u - u_0 = B\left(\frac{\sigma_{1f}}{3} + \frac{2\sigma_{3f}}{3} - \sigma_0\right) + BK\varepsilon_{vol}^{PL}$$

Together with the Terzaghi's principle of effective stress, this equation describes the hydro-mechanical coupling during an undrained triaxial test given the assumptions stated above. In the following, the angle of dilatancy ψ is assumed to be zero. Therefore, the flow rule can be reduced to $\varepsilon_{vol}^{PL} = 0$ (i.e. the plastic part of the volumetric strain becomes zero).

Taking the Mohr-Coulomb yield criterion $\sigma'_{1f} = m\sigma'_{3f} + f_c$ into account and considering the fact that the volume remains constant under plastic conditions, an expression for the maximum difference between axial stress and radial stress can be derived:

$$\sigma_{1f} - \sigma_{3f} = \frac{3(m-1)(1-B)}{3+B(m-1)} \left(\sigma_{3f} - \sigma_0\right) + \frac{3(m-1)}{3+B(m-1)} \sigma'_0 + \frac{3}{3+B(m-1)} f_c$$

where the coefficients m and f_c are related to the effective friction angle ϕ' and the effective cohesion c':

$$m = \frac{1 + \sin\phi'}{1 - \sin\phi'}$$
$$f_c = \frac{2c'\cos\phi'}{1 - \sin\phi'}$$

For B < 1 the differential stress at failure $(\sigma_{1f} - \sigma_{3f})$ under undrained conditions is dependent on the radial stress.

For B = 1 the expression simplifies to:

$$\sigma_{1f} - \sigma_{3f} = \frac{3(m-1)}{m+2} \sigma'_0 + \frac{3}{m+2} f_c$$

and it can be seen that the differential stress at failure $(\sigma_{1f} - \sigma_{3f})$ under undrained conditions is not dependent on the radial stress.

With respect to total stress conditions it is only possible to describe the shear strength of a saturated specimen by a friction angle $\phi_u = 0^\circ$ and a cohesion c_u equal to half of the maximum differential stress at failure if B = 1. The cohesion c_u is then often referred to as the undrained shear strength S_u :

$$S_u = \left(\frac{3(m-1)}{m+2}\sigma'_0 + \frac{3}{m+2}f_c\right)/2$$

The undrained shear strength S_u , cannot be considered as actual material constant (i.e., an intact rock material property). It is dependent on the effective friction angle ϕ' , the effective cohesion c', in the general case on the plastic part of the volumetric strain ($\varepsilon_{vol}^{PL} > 0$), and on the initial effective stress $\sigma'_0 = \sigma_0 - u_0$. Furthermore, the expression above is only valid for stress conditions applied in a standard triaxial test where the intermediate principal stress is equal to the minimum principal stress. This cannot, for example, be directly transferred to a tunnel excavation where the stress conditions are different (i.e. the intermediate principal stress from the minimum principal stress).

The undrained shear strength S_u is therefore linked to the mechanical and hydraulic boundary conditions and values that have been determined in the laboratory under undrained conditions have to be considered together with the mechanical and hydraulic conditions applied in the tests.

sample	orientation to bedding	failure mode	sample diameter	sample height	bulk weight	dry weight	water content	degree of saturation	confining pressure	back pressure	Δε1/Δt
-	-		D (mm)	H (mm)	γ (g/cm3)	γd (g/cm3)	w (%)	Sr (%)	σ1 = σ3 (MPa)	u0 (MPa)	Δε1/Δt (1/s)
01	(90°) S	matrix	25.3	50.4	2.53	2.43	4.2	100	22.00	9.00	1.0E-04
02	S	matrix	25.3	50.3	2.53	2.43	4.2	104	22.00	9.00	1.0E-05
03	s	matrix	25.3	50.5	2.53	2.43	4.2	100	22.00	9.00	1.0E-06
04	S	matrix	25.3	50.1	2.53	2.43	4.2	99	22.00	9.00	1.0E-04
05	S	matrix	25.2	50.6	2.54	2.44	4.1	103	22.00	9.00	1.0E-07
S03	s	matrix	25.4	50.7	2.55	2.43	4.9	112	12.61	5.04	1.0E-06
S05	s	matrix	25.4	50.5	2.55	2.45	4.1	104	7.61	3.04	1.0E-06
S06	s	matrix	25.4	50.5	2.54	2.44	4.2	102	12.61	5.04	1.0E-06
S07	s	matrix	25.4	50.6	2.56	2.46	4.1	105	22.61	9.04	1.0E-06
S102	s	matrix	25.5	49.9	2.54	2.43	4.5	107	22.61	9.04	1.0E-06
S106	s	matrix	25.5	50.5	2.55	2.45	4.3	104	7.61	3.04	1.0E-06
P09	(0°) P	matrix	25.4	50.8	2.55	2.44	4.2	106	7.61	3.04	1.0E-06
P10	Ρ	matrix	25.4	50.8	2.53	2.43	4.2	102	12.61	5.04	1.0E-06
P13	Ρ	matrix	25.4	50.7	2.52	2.43	4.0	103	22.61	9.04	1.0E-06
P14	Ρ	matrix	25.4	50.7	2.55	2.43	4.6	112	22.61	9.04	1.0E-06
P109	Ρ	matrix	25.4	49.5	2.51	2.39	4.7	104	12.61	5.04	1.0E-06
P115	Ρ	matrix	25.5	50.7	2.51	2.41	4.5	99	7.61	3.04	1.0E-06
X24	(30°) X	bedding	25.4	50.8	2.55	2.44	4.4	113	7.61	3.04	1.0E-06
X25	x	bedding	25.4	50.7	2.55	2.45	4.1	110	12.61	5.04	1.0E-06
X27	x	bedding	25.5	50.8	2.53	2.42	4.3	108	22.61	9.04	1.0E-06
X30	x	bedding	25.5	50.7	2.54	2.44	3.8	103	22.61	9.04	1.0E-06
Z19	(45°) Z	bedding	25.4	50.7	2.56	2.46	4.2	105	7.61	3.04	1.0E-06
Z21	z	bedding	25.4	50.8	2.57	2.46	4.2	107	12.61	5.04	1.0E-06
Z23	z	bedding	25.5	50.8	2.53	2.43	4.4	103	22.61	9.04	1.0E-06

Appendix A2 Basic physical properties reported in Jahns (2013)

Amann/Vogelhuber

sample	orientation	quality levels	saturation fulfilled?	consolidation long enough?	shearing slow enough?	adequacy of test	effective strength
-	-	-	-	-	-	-	-
01	(90°) S	-	no statement possible (biased by consolidation)	probably slow enough (swelling not signficant)	too fast (very low t_f)	inadequate	no statement possible
02	S	-	no statement possible (biased by consolidation)	slow enough (consolidation completed)	too fast (very low t_f)	inadequate	no statement possible
03	S	(weight 0.75) B	probably saturated (no B, but high A)	slow enough (consolidation completed)	probably slow enough (low t_f, but high A)	probably adequate	probably correct
04	S	-	no statement possible (biased by consolidation)	probably slow enough (swelling not signficant)	too fast (very low t_f)	inadequate	no statement possible
05	S	В	probably saturated (no B, but high A)	slow enough (consolidation completed)	slow enough (high t_f, high A)	probably adequate	probably correct
S03	S	(weight 0.50) C	no statement possible (biased by consolidation)	too short (consolidation not completed)	probably too fast (low t_f)	inadequate	no statement possible
S05	S	C	not saturated (low B, low A)	no statement possible (biased by swelling)	probably too fast (low t_f)	inadequate	overestimated
S06	S	(weight 0.25) D	not saturated (low B, low A)	no statement possible (biased by swelling)	probably too fast (low t_f)	inadequate	overestimated
S07	S	D	not saturated (low B, low A)	no statement possible (biased by swelling)	probably too fast (low t_f)	inadequate	overestimated
S102	S	c	no statement possible (biased by consolidation)	slow enough (high t_c, consolidation completed)	probably too fast (low t_f)	inadequate	no statement possible
S106	S	В	no statement possible (biased by consolidation)	slow enough (high t_c, swelling completed)	probably too fast (low t_f)	inadequate	no statement possible
P09	(0°) P	-	not saturated (low B, low A)	no statement possible (biased by swelling)	probably too fast (low t_f)	inadequate	overestimated
P10	Р	-	not saturated (low B, low A)	no statement possible (biased by swelling)	probably too fast (low t_f)	inadequate	overestimated
P13	Р	D	no statement possible (biased by consolidation)	too short (consolidation not completed)	probably too fast (low t_f)	inadequate	no statement possible
P14	Р	В	not saturated (low B, low A)	no statement possible (biased by swelling)	probably too fast (low t_f)	inadequate	overestimated
P109	Р	В	probably saturated (high B, but increasing)	probably slow enough (consolid. not significant)	probably too fast (low t_f, low A)	inadequate	underestimated
P115	Р	В	saturated (high B, remaining constant)	no statement possible (biased by swelling)	probably too fast (low t_f, low A)	inadequate	underestimated
X24	(30°) X	В	saturated (high B, remaining constant)	no statement possible (biased by swelling)	probably too fast (low t_f, low A)	inadequate	underestimated
X25	х	В	saturated (high B, remaining constant)	no statement possible (biased by swelling)	probably too fast (low t_f, low A)	inadequate	underestimated
X27	х	C	saturated (high B, remaining constant)	slow enough (consolidation completed)	probably too fast (low t_f, low A)	inadequate	underestimated
X30	х	C	probably saturated (high B, but increasing)	probably slow enough (consolid. not significant)	probably too fast (low t_f, low A)	inadequate	underestimated
Z19	(45°) Z	D	no statement possible (biased by consolidation)	no statement possible (biased by swelling)	probably too fast (low t_f)	inadequate	no statement possible
Z21	z	C	no statement possible (biased by consolidation)	no statement possible (biased by swelling)	probably too fast (low t_f)	inadequate	no statement possible
Z23	Z	D	not saturated (low B, low A)	no statement possible (biased by swelling)	probably too fast (low t_f)	inadequate	overestimated
		Nagra	Amann/Vogelhuber	Amann/Vogelhuber	Amann/Vogelhuber	Amann/Vogelhuber	Amann/Vogelhuber

Appendix A3 Assessment of triaxial test results reported in Jahns (2013)

Appendix A4 Basic physical properties reported in Rummel & Weber (1999)

sample	orientation to bedding	failure mode	sample diameter	sample height	bulk weight	dry weight	water content	degree of saturation	confining pressure	back pressure	Δε1/Δt
			D (mm)					C= (0()	-1 -2 (140-)		A = 1 / A + / 1 / -)
			D (mm)	H (mm)	γ (g/cm3)	γa (g/cm3)	W (%)	Sr (%)	σ1 = σ3 (MPa)	uu (MPa)	Δε1/Δt (1/s)
2A1s	(90°) S	matrix	29.7	66.8	2.53	2.45	3.2	81	10.16	0.01/0.11	1.0E-06
2A3s	S	matrix	29.5	66.4	2.53	2.45	3.2	82	10.00	0.17 / 0.22	1.0E-06
4A1s	S	matrix	29.4	58.2	2.52	2.41	4.4	96	5.03	0.14 / 0.39	1.0E-06
26A1s	S	matrix	29.8	66.5	2.51	2.41	4.4	94	20.00	0.04 / 0.12	1.0E-06
1A1p	(0°) P	matrix	29.6	66.1	2.56	2.46	3.7	100	10.00	0.17 / 0.18	1.0E-06
1A2p	Р	matrix	29.6	67.6	2.55	2.46	3.7	97	10.00	0.07 / 0.07	1.0E-06
4A2p	Р	matrix	29.6	67.5	2.52	2.42	4.4	98	5.00	0.21/0.12	1.0E-06
6A1p	Р	matrix	29.7	67.2	2.51	2.41	4.2	91	20.02	0.07 / 0.70	1.0E-06
8A3p	Р	matrix	29.6	66.9	2.45	2.37	3.1	59	20.00	0.31/0.31	1.0E-06
1A1z	(45°) Z	bedding	29.7	62.9	2.53	2.44	3.7	89	10.00	0.23 / 0.37	1.0E-06
5A1z	z	bedding	29.7	66.9	2.51	2.40	4.8	100	5.00	0.16 / 0.30	1.0E-06
5A2z	z	bedding	29.6	66.8	2.55	2.43	4.8	112	20.02	0.33 / 0.25	1.0E-06
8A1z	z	bedding	29.9	66.8	2.56	2.48	3.1	90	9.97	3.49 / 3.53	1.0E-06
8A2z	z	bedding	29.8	67.8	2.55	2.47	3.1	87	5.01	0.16 / 0.16	1.0E-06
	-							Amann/Vogelhuber			

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effective strength	adequacy of test	shearing slow enough?	consolidation long enough?	saturation fulfilled?	quality level	orientation	sample
	-				-	-	-
ite overestimated	inadequate	probably too fast (low t_f)	no statement possible (not documented)	not saturated (no B, low u0, low A)	(weight 0.50) C	(90°) S	2A1s
ite overestimated	inadequate	probably too fast (low t_f)	no statement possible (not documented)	not saturated (no B, low u0, low A)	C	S	2A3s
ite overestimated	inadequate	probably too fast (low t_f)	no statement possible (not documented)	not saturated (no B, low u0, low A)	С	S	4A1s
ite overestimated	inadequate	probably too fast (low t_f)	no statement possible (not documented)	not saturated (no B, low u0, low A)	С	S	26A1s
ite overestimated	inadequate	probably too fast (low t_f)	no statement possible (not documented)	not saturated (no B, low u0, low A)	C	(0°) P	1A1p
ite overestimated	inadequate	probably too fast (low t_f)	no statement possible (not documented)	not saturated (no B, low u0, low A)	С	Р	1A2p
ite overestimated	inadequate	probably too fast (low t_f)	no statement possible (not documented)	not saturated (no B, low u0, low A)	C	Р	4A2p
ite overestimated	inadequate	probably too fast (low t_f)	no statement possible (not documented)	not saturated (no B, low u0, low A)	С	Р	6A1p
ite overestimated	inadequate	probably too fast (low t_f)	no statement possible (not documented)	not saturated (no B, low u0, low A)	C	Р	8A3p
ite overestimated	inadequate	probably too fast (low t_f)	no statement possible (not documented)	not saturated (no B, low u0, low A)	C	(45°) Z	1A1z
ite overestimated	inadequate	probably too fast (low t_f)	no statement possible (not documented)	not saturated (no B, low u0, low A)	C	Z	5A1z
ite overestimated	inadequate	probably too fast (low t_f)	no statement possible (not documented)	not saturated (no B, low u0, low A)	C	Z	5A2z
ite no statement possible	inadequate	probably too fast (low t_f)	no statement possible (not documented)	no statement possible (no B, but high u0)	С	Z	8A1z
ite overestimated	inadequate	probably too fast (low t_f)	no statement possible (not documented)	not saturated (no B, low u0, low A)	С	Z	8A2z
er Amann/Vogelhuber	Amann/Vogelhuber	Amann/Vogelhuber	Amann/Vogelhuber	Amann/Vogelhuber	Nagra		

Appendix A5 Assessment of triaxial test results reported in Rummel & Weber (1999)

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