RA Experiment:
Updated Review of the Rock Mechanics Properties of the Opalinus Clay of the Mont Terri URL based on Laboratory and Field Testing

H. Bock

Q+S Consult, Germany
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Title RA Experiment
Updated Review of the Rock Mechanics Properties of the Opalinus Clay of the Mont Terri URL based on Laboratory and Field Testing

By Helmut Bock

Date 30th June, 2009
Executive summary

This Technical Report presents an update of the rock mechanics properties and an extension of the data base of the Opalinus Clay of the Mont Terri Underground Rock Laboratory.

A first review of the rock mechanics properties was carried out in the year 2000. That review was documented in Mont Terri Technical Report TR 2000-02 (Bock, 2001). In the following years TR 2000-02 has established itself as a reference source for numerous investigations of the Mont Terri project, be it laboratory testing, field testing, on-site monitoring or numerical modelling. An update of the rock mechanics data base was indicated as several new investigations, relevant to rock mechanics parameters, have been carried out since 2000. Furthermore, some new rock parameters, in particular thermal parameters, came into the focus of ongoing rock mechanics investigations and which were not considered within the 2000 report.

The review is based on 42 selected documents published since 2000. These documents include 5 Mont Terri Project Technical Reports, 30 Mont Terri Project Technical Notes and 7 other document sources, relevant for rock mechanics parameter values at Mont Terri.

As an outcome of the review an actualised and extended table of rock mechanics parameters is provided at the end of the report (ref. to Tab. 6-1 on Page 50 and Tab. 6-2 on Page 51 f.).
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6 Recommendation for the selection of the rock mechanics parameters

Acknowledgement

Appendix

A Source documents
B Plots from relevant laboratory and field tests and numerical modelling assumptions
1 Introduction

A review of the rock mechanics properties of the Opalinus Clay of the Mont Terri Underground Rock Laboratory was carried out in the year 2000 and documented in Technical Report TR 2000-02 (Bock, 2001). Since then that TR has established itself as an important reference source for numerous investigations of the Mont Terri Project, be it laboratory or field testing, on-site monitoring or numerical modelling.

The Mont Terri Project, St. Ursanne, Switzerland requested \textsuperscript{1} Prof. Dr.-Ing. H. Bock of Q+S CONSULT, Bad Bentheim, Germany to undertake a further review and an update of the rock mechanical data base. The renewed review should include the following:

a. screening of all Mont Terri Project TRs and TNs (Technical Notes) and other relevant document sources, published since 2000, on relevant rock mechanics properties of the Opalinus Clay;

b. based on the above, identification and compilation of papers and reports which contain new information on rock mechanical properties of the Opalinus Clay;

c. recommendation of an actual rock mechanical data set for the Opalinus Clay, and

d. documentation of the work in a Mont Terri Project TR.

The comprehensive rock mechanics data set, previously developed and documented in TR 2000-02, provided the base for this further review. Following identification of the relevant new source documents (Section 2), the set of established rock mechanics parameter values of the Opalinus Clay at Mont Terri is actualised and, in those instances where a parameter had not as yet been considered in TR 2000-02, extended by the inclusion of some new parameters. The latter, for instance, applies to thermo-mechanical parameters of the Opalinus Clay which in recent years were intensively considered within various research programmes.

In line with TR 2000-02 a distinction is made between “state / index

\textsuperscript{1} Contract order of 30\textsuperscript{th} June, 2008; Mont Terri Project Number 32.0300.RA2_PH13.
parameters” (Section 3) and “design parameters” (Section 4). Section 5 considers the degree of structural and mechanical anisotropy of the Opalinus Clay controlled by bedding, a feature which, in recent years, has attracted increased attention. Furthermore, Section 5 briefly explores the feasibility and meaningfulness of rock mass classification systems for underground openings in the Opalinus Clay. In comparing the new parameter values, developed in Sections 3 and 4, with those of the TR 2000-02 report, a judgement is arrived at as to the most likely set of rock mechanics parameter values prevailing at Mont Terri, leading to recommendations for actualised rock mechanical parameter values (Section 6).

2 Source Documents

The review documents were selected in co-operation with Dr. Tim Vietor (Nagra, Wettingen). These documents, termed “source documents”, are grouped in Table 2-1 and specified in Appendix A. They include Mont Terri Technical Reports (TRs), Mont Terri Technical Notes (TNs) and open literature and other reports.

<table>
<thead>
<tr>
<th>Type of document</th>
<th>Internal No. in this Report</th>
<th>Abbreviated List</th>
</tr>
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<tbody>
<tr>
<td>Open literature and other reports</td>
<td>A.3.1 – A.3.7</td>
<td>Wenk et al. (2008); Gens et al. (2007); Marschall et al. (2008); Blümling et al. (2007); Corkum &amp; Martin (2007); Naumann &amp; Plischke (2005); Lux et al. (2007).</td>
</tr>
</tbody>
</table>

Tab. 2-1. Source documents of the 2008 review (for details ref. to List of References and Appendix A)
The source documents of the rock mechanics parameters of the Opalinus Clay at Mont Terri relate to laboratory tests, in-situ (field) tests as well as monitoring and numerical modelling. Many documents, particularly the Mont Terri Technical Reports, cover more than a single of the above-mentioned aspects. For this reason it was not considered necessary to differentiate source documents along the lines of TR 2000-02, e.g. categorised to individually cover either laboratory, field, or EDZ experiments.

3 Physical Characterisation of the Opalinus Clay - Index and State Parameters -

Index (or state) parameters provide an indication of the character of soils or rocks. They assist in their classification, give a better understanding of their nature and provide an indication of their most likely behaviour in technical operations such as drilling, crushing and excavating.

Table 3-1 lists those source documents which contain some new information on index/state parameters.

<table>
<thead>
<tr>
<th>Source Document (new)</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
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<td>A.3.6 Naumann et al.</td>
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<tr>
<td>A.3.7 Lux et al.</td>
<td>x</td>
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</tbody>
</table>

Tab. 3-1. New information on index / state parameters since TR 2000-02.

Legend: x Accepted for parameter value evaluation
        (x) Rejected for parameter value evaluation (for reasons, ref. to text)

1 Bulk density (natural) 2 Bulk density (dry) 3 Grain density
4 Water content 5 Water loss porosity n 6 Ultrasonic velocity v_p and v_s
7 Atterberg limits 8 Carbonate content 9 Sulphate content
10 Fracture toughness 11 Bridgman pinch-off 12 Abrasivity index
For new results refer to the following sections and Tables 3-2 to 3.8.

### 3.1 Bulk density $\rho$ (in natural condition)

**Definition:**

\[
\rho = \frac{M}{V} = \frac{(M_s + M_W)}{V} \quad \cdots \quad (1)
\]

with: 
- $M$ = mass of bulk sample
- $M_s$ = mass of solids
- $M_W$ = mass of water
- $V$ = volume of bulk sample

**References:** see TR 2000-02

ISRM (Ulusay and Hudson, 2007\(^2\), p. 83 – 92).

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Number of samples n</th>
<th>Parameter value mean ± standard deviation</th>
<th>References</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bulk Density (natural</td>
<td>335</td>
<td>$\rho = 2430 \pm 20 \text{ [kg/m}^3]] $</td>
<td>see Table 3-1</td>
</tr>
<tr>
<td>conditions)</td>
<td>239</td>
<td>$\rho = 2450 \pm 30 \text{ [kg/m}^3]] $</td>
<td>TR 2000-02</td>
</tr>
</tbody>
</table>

**Tab. 3-2.** Bulk Density of the Opalinus Clay at Mont Terri in natural conditions: New results (top) and comparison with 2000 data (bottom)

**Comment:**

C 3.1 Distinguishing the bulk density for the various geological facies (e.g. sandy, carbonate-rich, shaly facies) does not yield any significant differences.

### 3.2 Bulk density $\rho_d$ (dry conditions)

**Definition:**

\[
\rho_d = \frac{M_s}{V} \quad \cdots \quad (2)
\]

with: 
- $M_s$ = mass of solids
- $V$ = volume of bulk sample

**Reference:** see TR 2000-02

---

\(^2\) Ulusay and Hudson (2007): The complete ISRM Suggested Methods (see List of References).
### Tab. 3-3. Bulk Density (dry) of the Opalinus Clay at Mont Terri. New results (top) and comparison with 2000 data (centre) and Nagra NTB 02-03 (2002) data (bottom)

Comment: see C 3.1

#### 3.3 Grain density $\rho_s$

**Definition:**

\[
\rho_s = \frac{M_s}{V_s} \quad \text{...................................................... (3)}
\]

with:

- $M_s = \text{mass of solids}$
- $V_s = \text{volume of solids}$

**Reference:** see TR 2000-02

### Tab. 3-4. Grain Density of the Opalinus Clay at Mont Terri. New results (top) and comparison with 2000 data (centre) and Nagra NTB 02-03 (2002) data (bottom)

Comment: see C 3.1
3.4 Water (or moisture) content $w$

This parameter relates to the water content of the rock in its natural state. It specifically relates to the “free” water within the pores of the Opalinus Clay which is subject to gravimetric forces. It does not relate to adsorbed or structurally bound water (Horseman et al. 1996). Investigations carried out by Chiffoleau and Robinet (in TN 98-36) indicate that, for the Opalinus Clay at Mont Terri, the quantity of “free” water is less than that of the adsorbed water. According to Nagra (2002) the portion of “free” water is about 26%.

Definition:

$$w = \left( \frac{M_w}{M_s} \right) \times 100 \, [\%] \quad \text{......................... (4a)}$$

with:

- $M_w$ = mass of “free” water
- $M_s$ = mass of solids (including adsorbed water)

Reference: see TR 2000-02

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Number of samples $n$</th>
<th>Parameter value mean $\pm$ standard deviation</th>
<th>References</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water content</td>
<td>251</td>
<td>$w = 6.4 \pm 0.8 , [%]$</td>
<td>see Table 3-1</td>
</tr>
<tr>
<td></td>
<td>116</td>
<td>$w = 6.1 \pm 1.9 , [%]$</td>
<td>TR 2000-02</td>
</tr>
</tbody>
</table>

Tab. 3-5. Water content of the Opalinus Clay at Mont Terri in natural conditions. New results (top) and comparison with 2000 data (bottom)

Comments:

C 3.2 All test results considered within this Report are based on the oven drying method applying temperatures between 105°C and 115°C over a duration of 1 to 7 days.

C 3.3 The following equation (4b) $\rho_d = \rho / \left[ 1 + \frac{w}{100} \right]$ ............ (4b) specifies the relationship between the bulk density $\rho$ (in natural condition), the dry density $\rho_d$ and the water content:

For control, substitution of the new values for $\rho$ and $w$ from Tables 3-1 and 3-4 into (4b) yields $\rho_d = 2284 \, [\text{kg/m}^3]$. This value is just within the range of $2330 \pm 50 \, [\text{kg/m}^3]$ as specified in Tab. 3-2.
3.5 Water loss porosity n

Definition: \[ n = \left( \frac{V_v}{V} \right) \cdot 100 \, [%] \] .................................................. (5 a)
\[ n = (1 - \frac{\rho_d}{\rho_s}) \cdot 100 \, [%] \] ........................................... (5 b)

with: \( V_v \) = volume of voids
\( V \) = bulk sample volume
\( \rho_d \) = dry density
\( \rho_s \) = grain density

References: see TR 2000-02
NAGRA (2002)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Number of samples n</th>
<th>Parameter value mean ± standard deviation</th>
<th>References</th>
</tr>
</thead>
<tbody>
<tr>
<td>Porosity</td>
<td>21</td>
<td>( n = 13.7 \pm 2.5 , [%] )</td>
<td>see Table 3-1</td>
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<td></td>
<td>17</td>
<td>( n = 13.7 \pm 3.1 , [%] )</td>
<td>TR 2000-02</td>
</tr>
<tr>
<td></td>
<td>63</td>
<td>( n = 15.7 \pm 2.2 , [%] )</td>
<td>NTB 02-03</td>
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</table>

Tab. 3-6. Water loss porosity of the Opalinus Clay at Mont Terri. New results (top) and comparison with 2000 data (centre) and Nagra NTB 02-03 (2002) data base (bottom)

Comment:

C 3.4 The determination of the porosity was carried out by substituting the values \( \rho_d = 2330 \pm 50 \, [\text{kg/m}^3] \) and \( \rho_s = 2700 \pm 20 \, [\text{kg/m}^3] \), as determined in Sections 3.2 and 3.3, into Equation (5 b).

3.6 Ultrasonic velocities \( v_p \) and \( v_s \) and dynamic elastic constants

Definition: \[ E_{\text{dyn}} = 2 \cdot v_s^2 \cdot \rho \left( 1 + \nu_{\text{dyn}} \right) \] ......................... (6 a)
\[ \nu_{\text{dyn}} = 0.5 \cdot \frac{[2 - (v_p/v_s)^2]}{[1 - (v_p/v_s)^2]} \] ................ (6 b)

with: \( E_{\text{dyn}} \) = dynamic Young’s Modulus
\( \nu_{\text{dyn}} \) = dynamic Poisson’s ratio
\( \rho \) = density
\( v_p \) = compressional wave velocity
\( v_s \) = shear wave velocity
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// ss = index for P-samples (in direction of bedding)  
┴ ss = index for S-samples (normal to bedding)

Reference: see TR 2000-02

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Number n of samples</th>
<th>Parameter value mean ± standard deviation</th>
<th>Reference</th>
<th>Remarks</th>
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<td>(v_p \parallel ss)</td>
<td>88</td>
<td>3 340 ± 130 [m/s]</td>
<td>see Table 3-1</td>
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</tr>
<tr>
<td></td>
<td>111</td>
<td>3 410 ± 240 [m/s]</td>
<td>TR 2000-02</td>
<td></td>
</tr>
<tr>
<td>(v_s \parallel ss)</td>
<td>64</td>
<td>1 900 ± 100 [m/s]</td>
<td>see Table 3-1</td>
<td></td>
</tr>
<tr>
<td></td>
<td>111</td>
<td>1 960 ± 120 [m/s]</td>
<td>TR 2000-02</td>
<td></td>
</tr>
<tr>
<td>(v_p / v_s \parallel ss)</td>
<td>-</td>
<td>1.76</td>
<td>1.74 in TR 2000-02</td>
<td></td>
</tr>
<tr>
<td>(v_p \perp ss)</td>
<td>45</td>
<td>2 620 ± 130 [m/s]</td>
<td>see Table 3-1</td>
<td></td>
</tr>
<tr>
<td></td>
<td>48</td>
<td>2 620 ± 400 [m/s]</td>
<td>TR 2000-02</td>
<td></td>
</tr>
<tr>
<td>(v_s \perp ss)</td>
<td>30</td>
<td>1 520 ± 40 [m/s]</td>
<td>see Table 3-1</td>
<td></td>
</tr>
<tr>
<td></td>
<td>33</td>
<td>1 510 ± 250 [m/s]</td>
<td>TR 2000-02</td>
<td></td>
</tr>
<tr>
<td>(v_p / v_s \perp ss)</td>
<td>-</td>
<td>1.72</td>
<td>1.73 in TR 2000-02</td>
<td></td>
</tr>
<tr>
<td>(E_{dyn} \parallel ss)</td>
<td>average from above</td>
<td>22.0 [GPa]</td>
<td>Note 3.6.1</td>
<td>TR 2000-02</td>
</tr>
<tr>
<td></td>
<td></td>
<td>23.5* [GPa]</td>
<td></td>
<td></td>
</tr>
<tr>
<td>(v_{dyn} \parallel ss)</td>
<td>average from above</td>
<td>0.26 [-]</td>
<td>Note 3.6.1</td>
<td>TR 2000-02</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.25</td>
<td></td>
<td></td>
</tr>
<tr>
<td>(E_{dyn} \perp ss)</td>
<td>average from above</td>
<td>14.0 [GPa]</td>
<td>Note 3.6.1</td>
<td>TR 2000-02</td>
</tr>
<tr>
<td></td>
<td></td>
<td>14.0 [GPa]</td>
<td></td>
<td></td>
</tr>
<tr>
<td>(v_{dyn} \perp ss)</td>
<td>average from above</td>
<td>0.25 [-]</td>
<td>Note 3.6.1</td>
<td>TR 2000-02</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.25</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Tab. 3-7. Ultrasonic velocity and dynamic elastic parameters.  
New results (highlighted in grey in the respective tops) and comparison with data base of 2000 (respective bottoms)
C 3.5 The determination of $E_{\text{dyn}}$ and $\nu_{\text{dyn}}$ was carried out by substitution of the relevant mean values of $v_p$ and $v_s$ and of $\rho = 2430$ [kg/m$^3$] (Section 3.1) into Equations (6a) and (6b).

C 3.6 Amongst others, Popp and Salzer (TR 2007-04) and Wolter (TN 2002-46) measured the ultrasonic velocity of samples which were hydrostatically loaded of up to 50 MPa. There is a slight dependency between $v_p$ and the hydrostatic pressure in the sense that higher pressures lead to higher velocities and, implicitly, also to a higher dynamic Young’s moduli (Appendix B; Fig. B-1).

3.7 Atterberg limits

Definition: \[ P.I. = w_l - w_p \] .................................................... (7)

with: \[ P.I. = \text{plasticity index} \]
\[ w_l = \text{water content at liquid limit} \]
\[ w_p = \text{water content at plastic limit} \]

Reference: see TR 2000-02

Within the 2008 source documents there is no new information available.

3.8 Carbonate content $C_{\text{RCO}_3}$

Definition: \[ C_{\text{RCO}_3} = \left( \frac{M_R}{M_s} \right) \cdot 100 \text{ [%]} \] (8)

with: \[ M_R = \text{mass of Ca-, Mg-, Fe-, Sr-, Ba-ions} \]
\[ M_s = \text{mass of solids} \]

Reference: see TR 2000-02

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Number of samples $n$</th>
<th>Carbonate content mean $\pm$ standard deviation</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>$C_{\text{RCO}_3}$ content</td>
<td>9</td>
<td>$C_{\text{RCO}_3} = 11.8 \pm 3.8$ [%] (*)</td>
<td>see Table 3-1</td>
</tr>
<tr>
<td></td>
<td>33</td>
<td>$C_{\text{RCO}_3} = 9.4 \pm 5.9$ [%] (*)</td>
<td>TR 2000-02</td>
</tr>
</tbody>
</table>

Tab. 3-8. Carbonate content of the Opalinus Clay at Mont Terri. New results (top) and comparison with data base of 2000 (bottom)
Comment:

C 3.7 The comparatively very high standard deviation (*) is indicative of a relatively inhomogeneous distribution of the $\text{RCO}_3$ content within the Opalinus Clay at Mont Terri.

3.9 Sulphate content $C_{\text{CaSO}_4}$

Definition:  
$$C_{\text{CaSO}_4} = \left( \frac{M_{\text{CaSO}_4}}{M_s} \right) \cdot 100\% \quad \text{(9)}$$

with:  
- $M_{\text{CaSO}_4}$ = mass of Ca-sulphates (anhydrite and gypsum)
- $M_s$ = mass of solids

Within the 2008 source documents there is no new information available.

3.10 Fracture toughness $K_{\text{IC}}$

Definition:  
$$K_{\text{IC}} = A_{\text{min}} \cdot \frac{F_{\text{max}}}{D^{1.5}} \quad \text{(10)}$$

with:  
- $F_{\text{max}}$ = failure load
- $D$ = diameter of the specimen
- $A_{\text{min}}$ = dimensionless factor
  
  $$A_{\text{min}} = 3.33 \left[ 1.835 + 7.15 \frac{a_o}{D} + 9.85 \left( \frac{a_o}{D} \right)^2 \right]$$

  with $a_o$ = initial crack length

Reference: see TR 2000-02

Within the 2008 source documents there is no new information available.

3.11 Bridgman pinch-off strength

Definition:  
$$p_{\text{mco}} = p_{\text{mc}} - \sigma_1 \quad \text{(11)}$$

with:  
- $p_{\text{mco}}$ = hydraulic tensile strength of the specimen
- $p_{\text{mc}}$ = confining pressure at tensile failure
- $\sigma_1$ = axial stress

Reference: see TR 2000-02

Within the 2008 source documents there is no new information available.
3.12 Abrasivity index

“Abrasivity” is a term which refers to the potential of a soil or rock to wear down excavation tools, e.g. in connection with drilling, grinding, cutting or dredging. In underground construction the abrasivity is of considerable interest in the owner-contractor relationship, particularly when a road header or a TBM is employed in the excavation work (for an actual example, refer to Appendix B; Fig. B-2).

Tests on the abrasivity of the Opalinus Clay at Mont Terri have yet to be carried out. Besides the methods described in the ISRM Suggested Methods (Ulusay and Hudson, 2007) there are the following possibilities for a characterisation of the abrasivity:

a. Rock Abrasivity Index RAI

Definition: \[ \text{RAI} = \text{AQu} \times \text{UCS} \] [%] ......................... (12a)

with: \( \text{AQu} = \) Equivalent quartz content
\( \text{UCS} = \) unconfined compressive strength

Reference: Plinninger et al. (2002)

b. Cherchar Abrasivity Index CAI

Definition: \[ \text{CAI} = 10 \times \frac{d}{k} \] [-] ............................... (12b)

with: \( d = \) diameter of wear area of standardised test needle
\( k = \) constant (= 1 mm)

Reference: Cherchar (1985)

c. Abrasivity Coefficient LAK in a LCPC-Test

Definition: \[ \text{LAK} = \frac{\Delta W}{M} \] [g/t] ................................. (12c)

with: \( \Delta W = \) weight loss of a stirring tool in the standardised LCPC test
\( M = \) unit mass

References: ANFOR (1990) and (2000)

4 Rock Mechanics Design Parameters

Design parameters provide quantitative materials characteristics for calculations, modelling and predictions. They are used as input parameters in the design and numerical modelling of geotechnical structures, such as finite element or limit equilibrium computations of underground structures.

Rock mechanics design parameters of the Opalinus Clay at Mont Terri were determined by laboratory and/or field testing and by numerical back analysis of prototype experiments. Table 4-1 lists those source documents which contain some new information on the rock mechanics design parameters.

<table>
<thead>
<tr>
<th>Source Document</th>
<th>Information on Rock Mechanics Design Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>21 22 23 24 25 26 27 28 29 30 31 32 33 34</td>
</tr>
<tr>
<td>A.1.1 TR 2003-04</td>
<td>(x) x x x</td>
</tr>
<tr>
<td>A.1.2 TR 2006-01</td>
<td>(x) (x) x x x</td>
</tr>
<tr>
<td>A.1.3 TR 2007-02</td>
<td>(x) x (x) x x x x x</td>
</tr>
<tr>
<td>A.1.4 TR 2007-04</td>
<td>x x x x</td>
</tr>
<tr>
<td>A.1.5 TR 2007-05</td>
<td>x (x)</td>
</tr>
<tr>
<td>A.2.9 TN 2001-28</td>
<td>(x)</td>
</tr>
<tr>
<td>A.2.10 TN 2002-46</td>
<td>(x)</td>
</tr>
<tr>
<td>A.2.12 TN 2002-50</td>
<td>(x) x (x)</td>
</tr>
<tr>
<td>A.2.13 TN 2003-03</td>
<td>(x) x x</td>
</tr>
<tr>
<td>A.2.14 TN 2003-17</td>
<td>x</td>
</tr>
<tr>
<td>A.2.17 TN 2004-38</td>
<td>x x x</td>
</tr>
<tr>
<td>A.2.18 TN 2004-56</td>
<td>x</td>
</tr>
<tr>
<td>A.2.21 TN 2005-25</td>
<td>(x) (x)</td>
</tr>
<tr>
<td>A.2.22 TN 2005-29</td>
<td>x x</td>
</tr>
<tr>
<td>A.2.23 TN 2005-34</td>
<td>x x x (x)</td>
</tr>
<tr>
<td>A.2.24 TN 2005-57</td>
<td>x x (x)</td>
</tr>
<tr>
<td>A.2.26 TN 2006-37</td>
<td>x x</td>
</tr>
<tr>
<td>A.2.27 TN 2006-50</td>
<td>(x)</td>
</tr>
<tr>
<td>A.2.29 TN 2006-72</td>
<td>(x)</td>
</tr>
<tr>
<td>A.2.30 TN 2007-30</td>
<td>x (x) x x x</td>
</tr>
<tr>
<td>A.3.2 Gens et al.</td>
<td>(x)</td>
</tr>
<tr>
<td>A.3.3 Marschall et al.</td>
<td>(x)</td>
</tr>
<tr>
<td>A.3.4 Corkum &amp; Martin</td>
<td></td>
</tr>
<tr>
<td>A.3.6 Naumann et al.</td>
<td>x x x</td>
</tr>
<tr>
<td>A.3.7 Lux et al.</td>
<td>x x x x</td>
</tr>
</tbody>
</table>

Tab. 4-1. New information on rock mechanics design parameters since TR 2000-02.

Legend: x Substantial information and accepted for parameter value evaluation
(x) Marginal or general information, or rejected for parameter value evaluation

21 Deformation moduli E
22 Poisson’s ratio ν
23 Bedding plane stiffness
24 Creep parameters
25 Unc.compr.strength UCS
26 Tensile strength UTS
27 Shear strength material
28 Shear strength bedding
29 Dilatation δ and angle i
30 Permeability k / hydr.conductivity
31 Swelling and shrinking
32 Linear thermal expansion
33 Heat capacity
34 Thermal conductivity
The pertinent design parameters may be classified as follows:

1. **Deformation Parameters (Section 4.1)**
   - General deformation parameters of material:
     - First-loading modulus \( E_{\text{init}} \)
     - Unloading modulus \( E_u \)
     -Reloading modulus \( E_r \)
   - Linear elastic parameters of material:
     - Young’s modulus \( E \)
     - Poisson’s ratio \( \nu \)
   - Deformation parameters of discontinuities:
     - Normal stiffness
     - Shear stiffness
   - Long-term deformation (creep) parameters:
     - Viscosity

2. **Strength Parameters (Section 4.2)**
   - Uniaxial strength parameters:
     - Compressive: UCS
     - Tensile: UTS
   - Mohr-Coulomb failure parameters (for both material + bedding planes):
     - Cohesion \( c \)
     - Friction angle \( \phi \)
     - Dilatation (bulk \( \delta \) and angle \( i \))
   - Hoek and Brown parameters \( m \) and \( s \)

3. **Permeability Parameters (Section 4.3)**
   - Intrinsic permeability \( k \)
   - Hydraulic conductivity \( K \)

4. **Hydro-mechanically coupled parameters (Section 4.4)**
   - Coefficient of consolidation \( C_v \)
   - Swelling parameters:
     - Swelling pressure \( p_s \)
     - Swelling strain index \( S_e \)
   - Deformability and strength as a function of the water content \( w \)

5. **Thermal parameters (Section 4.5)**
- Linear thermal expansion solid grains coefficient $\alpha \ [K^{-1}]$
- Specific heat capacity (solids) $c_s \ [J / kg^{-1} K^{-1}]$
- Thermal conductivity $\lambda \ [W / m^{-1} K^{-1}]$

The Opalinus Clay is a distinctively bedded material. Its mechanical behaviour can best be described in a *transverse isotropic model* (Fig. 4-1 top). The mechanical importance of the bedding planes has been increasingly accepted within the review period 2000 to 2008. The test samples are commonly specified and termed with respect to the orientation of the sample axis towards bedding (Fig. 4-1; bottom). The degree of structural and mechanical anisotropy of the bedding of the Opalinus Clay is considered in Section 5.1.

![Fig. 4-1. Convention of reference axis for a transverse isotropic rock (top) and designation of test samples (bottom)](image)

Where the elementary *isotropic model* is used, it is most often used for simplicity and/or as a first approach to the problem. It can be generally stated that the higher the refinement of a mechanical model, the more
considerable the accompanying efforts will be for both for determination of the design parameters and for carrying out the computation itself. In any specific application, engineering judgement is required to decide which of the alternative models, isotropic or transverse isotropic, is the most appropriate one and whether the chosen model is sufficient.

4.1 Deformation parameters

4.1.1 Short-term deformation parameters of the Opalinus Clay material

Isotropic elastic rock: Two (2) material constants as follows:

- \( E \): Young’s modulus
- \( \nu \): Poisson’s ratio

Transverse isotropic elastic rock: Five (5) material constants as follows:

- \( E_1 \): \((E \perp_{ss})\)
- \( \nu_{12} = \nu_{13} \): \((\nu //_{ss})\)
- \( E_2 = E_3 \): \((E //_{ss})\)
- \( \nu_{23} \): \((\nu \perp_{ss})\)
- \( G_{12} = G_{13} \): Shear modulus

Definition: Refer to Figs. 4-2 and 4-3 and the following comments.

Comments:

C 4.1 The elastic parameters \( E \) and \( \nu \) are defined with reference to a uniaxial test of an elastically behaving homogeneous isotropic body (Hooke’s Law; Timoshenko and Goodier, 1970). The slope of the axial stress versus axial strain curve is a measure solely of \( E \), that of the lateral strain versus axial strain curve solely of \( \nu \).

C 4.2 Uniaxial testing of Opalinus Clay samples reveals a somehow non-linear stress-strain response similar to that schematically shown in Fig. 4-2. The parameter \( E \) may be taken at the 50% strength level and denoted \( E_{t,50} \). The parameter \( \nu \) is commonly taken at the early portions of the axial strain- lateral strain curve. That portion, which corresponds to a relatively low stress level, is often reasonably linear.
C 4.3 Within the review period, several triaxial laboratory tests with deformation measurements have been carried out (e.g. Appendix A, numbers A.1.1; A.1.3; A.1.4; A.2.12; A.2.30; A.3.6). In triaxial testing (which can be seen as superposing three uniaxial tests) the slope of a stress-strain curve can only be used for the determination of E, if the confining pressure is held constant in the relevant test phase, i.e. if $\sigma_2 = \sigma_3 = \text{constant}$.

C 4.4 Certain Standards (e.g. ASTM, 2004; DIN 18141; DGGT Suggested Method, Mutschler, 2004) propose to determine the Young’s Modulus E from the unloading/reloading curves of cyclic tests. Within engineering accuracy both the unloading modulus $E_u$ and the reloading modulus $E_R$ are equal to E (i.e. $E_u \approx E_R \approx E$). Whenever possible, the Young’s Modulus E specified within this Report was determined from about the 30% to 70% range of the secant modulus of the unloading curve as indicated in Fig. 4-3.
Fig. 4-3. Definition of short-term deformation parameters in a cyclic test with:

- **UCS** = Unconfined Compressive Strength \( ISRM (*) \)
- **\( E_{\text{init}} \)** = First-loading Modulus \( DIN 18141 \)
- **\( E_u \)** = Unloading Modulus \( DIN 18141 \)
- **\( E_R \)** = Reloading Modulus \( DIN 18141 \)
- **\( E \)** = Young’s Modulus \( DIN 18141 \)
- **\( v \)** = Poisson’s ratio see Fig. 4-2

\( (*) \) Ulusay and Hudson, 2007; pp. 153 -154

C 4.5 As shown in Figs. 4-2 and 4-3, the stress-strain behaviour of the Opalinus Clay is often highly non-linear. With regard to the initial loading phase, which often shows a comparatively low initial modulus \( E_{\text{init}} \), Corkum and Martin (2007) anticipate that this feature is inherent to the Opalinus Clay. However, it should be kept in mind that deformability testing at low stress levels is sensitively dependent on a complex mix of conditions and factors such as sample retrieval, sample preparation, testing set-up and accuracy of the sample dimensions. Figure B-3 in Appendix B shows a test example where there is virtually no reduced initial...
stiffness at all. Against that background it seems to be of limited 

purpose to specify a dedicated modulus for low stress levels.

C 4.6 With regard to the non-linearity of the testing curve at higher stress 

levels towards reaching failure (100% UCS in Fig. 4-3), it is 

commonly agreed that this feature is physically caused by the 
ocurrence of micro cracks. Such cracks are pervasive features 

which, in line with their progressive formation, alter the structural 
and mechanical properties of the material. That alteration is more 
distinctly reflected in the lateral vs. axial strain curve (i.e. in the 
apparent Poisson’s ratio $\nu$ which might even exceed the value of 
0.5 ) than in the axial stress vs. axial strain curve (i.e. in the 
apparent $E$).

Results of short-term laboratory tests

P- and S-samples (Fig. 4-1) were used for the evaluation of the deformation 

parameters. Z-samples were not considered for this purpose.

For results of the isotropic elastic moduli, refer to Table 4-2. For the 
transverse isotropic moduli and the assessment of the influence of $\sigma_3$, refer 
to Table 4-3. For the Poisson’s ratio $\nu$ and the shear modulus $G$, refer to 
Table 4-4.

<table>
<thead>
<tr>
<th>new results</th>
<th>Comparison: TR 2000-02</th>
<th>References for new results</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E$ = 7.8 ± 5.1</td>
<td>$E = 12.7 ± 4.4$</td>
<td>ref. to Tab. 4-1 Column 21</td>
</tr>
<tr>
<td>$E_{\text{t-50}}$ = 2.7 ± 1.5</td>
<td>$E_{\text{t-50}} = 5.2 ± 2.7$</td>
<td>56</td>
</tr>
</tbody>
</table>

Tab. 4-2. Elastic moduli for an isotropic model of Opalinus Clay as deduced from 
short-term laboratory tests without consideration of the confining 
pressure $\sigma_3$. New results (left) and comparison with data base of 2000 
(right)
<table>
<thead>
<tr>
<th>Range</th>
<th>new results</th>
<th>Comparison: TR 2000-02</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\sigma_3$ [MPa]</td>
<td>Value [GPa]</td>
<td>n</td>
</tr>
<tr>
<td>0</td>
<td>$E_{//} = 11.1 \pm 5.6$</td>
<td>26</td>
</tr>
<tr>
<td></td>
<td>$E_{\perp} = 4.4 \pm 2.1$</td>
<td>15</td>
</tr>
<tr>
<td></td>
<td>$E_{//t-50} = 3.8 \pm 1.3$</td>
<td>28</td>
</tr>
<tr>
<td></td>
<td>$E_{\perp t=50} = 1.5 \pm 0.4$</td>
<td>18</td>
</tr>
<tr>
<td>2-4</td>
<td>$E_{//} = 7.1 \pm 2.3$</td>
<td>8</td>
</tr>
<tr>
<td></td>
<td>$E_{\perp} = 3.5 \pm 1.3$</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>$E_{//t-50} = 3.5 \pm 0.6$</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>$E_{\perp t=50} = 0.7 \pm 0.2$</td>
<td>2</td>
</tr>
<tr>
<td>5-6</td>
<td>$E_{//} = 8.0 \pm 2.5$</td>
<td>6</td>
</tr>
<tr>
<td></td>
<td>$E_{\perp} = 3.3 \pm 1.3$</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>$E_{//t-50} = 3.8 \pm 0.9$</td>
<td>8</td>
</tr>
<tr>
<td></td>
<td>$E_{\perp t=50} = 1.5 \pm 0.2$</td>
<td>6</td>
</tr>
<tr>
<td>10</td>
<td>$E_{//} = 11.5 \pm 1.6$</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>$E_{\perp} = 4.1 \pm 2.0$</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>$E_{//t-50} = 2.2 \pm 0.5$</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>$E_{\perp t=50} = 0.7 \pm 0.2$</td>
<td>6</td>
</tr>
<tr>
<td>15</td>
<td>$E_{//} = 14.9$</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>$E_{\perp} = 8.0$</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>$E_{//t-50} = 4.2 \pm 0.7$</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>$E_{\perp t=50} = 1.8 \pm 0.7$</td>
<td>4</td>
</tr>
<tr>
<td>Average $0 \leq \sigma_3 \leq 15$ [MPa]</td>
<td>$E_{//} = E_2 = 10.1 \pm 4.9$</td>
<td>44</td>
</tr>
<tr>
<td></td>
<td>$E_{\perp} = E_1 = 4.1 \pm 2.0$</td>
<td>31</td>
</tr>
<tr>
<td></td>
<td>$E_{//t-50} = 3.8 \pm 1.5$</td>
<td>49</td>
</tr>
<tr>
<td></td>
<td>$E_{\perp t=50} = 1.3 \pm 0.7$</td>
<td>36</td>
</tr>
</tbody>
</table>

Tab. 4-3. Elastic moduli for a transverse isotropic rock model of Opalinus Clay as deduced from short-term laboratory tests with consideration of confining pressure $\sigma_3$. New results (left) and comparison with data base of 2000 (right). The very limited values for $\sigma_3 > 15$ MPa were discarded for the evaluation.

Legend: $AF = \text{Anisotropy Factor (ref. to Section 5.1)}$
<table>
<thead>
<tr>
<th>Type of isotropy</th>
<th>Type of testing</th>
<th>new results</th>
<th>Comparison: TR 2000-02</th>
</tr>
</thead>
<tbody>
<tr>
<td>isotropic</td>
<td>P + S</td>
<td>( \nu = 0.29 \pm 0.09 )</td>
<td>( \nu = 0.27 \pm 0.08 )</td>
</tr>
<tr>
<td></td>
<td></td>
<td>( G = 3.0 ) [GPa] * *)</td>
<td>[GPa]</td>
</tr>
<tr>
<td>transverse isotropic</td>
<td>S</td>
<td>( \nu_{12} = \nu_{13} = 0.25 \pm 0.09 )</td>
<td>( \nu_{12} = \nu_{13} = 0.24 \pm 0.08 )</td>
</tr>
<tr>
<td></td>
<td>P</td>
<td>( \nu_{23} = 0.35 \pm 0.04 )</td>
<td>( \nu_{23} = 0.33 \pm 0.05 )</td>
</tr>
<tr>
<td></td>
<td>-</td>
<td>( G_{12} = G_{13} = 3.7 ) [GPa] ** *)</td>
<td>( G_{12} / G_{13} = 1.2 \pm 0.4 ) [GPa]</td>
</tr>
</tbody>
</table>

Tab. 4-4. Poisson’s ratio \( \nu \) and shear modulus \( G \) for an isotropic and transverse isotropic rock model of Opalinus Clay as deduced from short-term laboratory tests without consideration of the confining pressure \( \sigma_3 \). New results (left) and comparison with data base of 2000 (right)

Legend:

*) Computed from isotropic mean values by \( G = E / [2(1+\nu)] \)

**) No new data. Computed from transverse isotropic mean values by the following approximation (Naumann and Plischke, 2005):

\[ \frac{1}{G_{12}} = \frac{1}{E_1} + \frac{1}{E_2} + 2 \frac{\nu_{23}}{E_2} \]

Comments:

C 4.7 E was deduced solely from cyclic tests (secant modulus of the unloading curve; for definition refer to Fig. 4-3). \( E_{t-50} \) was deduced from both cyclic and non-cyclic tests (tangent modulus of the first-loading curve; for definition refer to Fig. 4-2). The isotropic moduli \( E \) were computed as the average of the transversal isotropic moduli \( E_{\perp ss} \) (\( E_1 \)) and \( E_{// ss} \) (\( E_2 \) and \( E_3 \)) (refer to Table 4-4 and Fig. 4-1).

C 4.8 Analogous procedures were employed for the determination of the Poisson’s ratio \( \nu \).

C 4.9 Part of the assessment of the deformation tests was also related to the strain measuring techniques employed in the tests. Consistent with Fairhurst and Hudson (1999), all types of deformation or strain measurements taken directly at the sample surface were considered to be most reliable, e.g. strain gauges (Auvray, TN 2006-37), “3-star” configuration in lateral strain.
measurements (Rummel and Weber, TN 2005-57) and mechanical axial and radial length measuring techniques employed by Jahns (TN 2007-30).

C 4.10 The data base accumulated since 2000 (and which provides the basis of this Report) is significantly more substantial than that presented in TR 2000-02. This is evidenced in the generally higher numbers of tests from which the various new parameter values were deduced (ref. to columns “n” in Tabs. 4-2 and 4-3).

C 4.11 In the tests, carried out within the review period, a generally higher degree of attention has been paid to the anisotropy of the Opalinus Clay due to bedding.

C 4.12 On inspection of Tabs. 4-2 and 4-3 it becomes evident that the new moduli values are generally lower than those of TR 2000-02. It is anticipated that these new values represent the deformation behaviour of the Opalinus Clay at Mont Terri more reliably than the 2000 values. Amongst the reasons for this assumption are the broader data base, the explicit attention to the various moduli in the respective first loading, unloading and reloading testing phases and the increased care which has been employed in sampling, sample preparation and sample storage procedures.

C 4.13 There is an ongoing debate surrounding the dependency of the confining pressure onto the deformation parameters. Amongst the authors who assume that such a dependency exists are Naumann and Plischke (2005), Corkum and Martin (2007) and Zhang et al. (TR 2007-02) (ref. to Figures B-4 and B-5 in Appendix B). Figure 4-4 compiles the new moduli data from all relevant tests conducted during the review period. From inspection it becomes evident that for those stress levels relevant for Mont Terri (i.e. of the order of about 2 up to 10 MPa) the new test data do not necessarily support the assumption of a significant dependency between $E$ and the confining pressure $\sigma_3$. 
C.4.14 As already noted in C 4.2, testing of Opalinus Clay samples regularly reveals a non-linear stress-strain response. When selecting short-term deformation parameters for numerical modelling calculations, there is an ongoing debate as to how far any such non-linearity should be accommodated within the model. Thus far, the focus of the debate has been on a non-linear modulus $E$, either in the range of low stress (ref. to C 4.5) or that of high stress close to the strength limit (ref. to C 4.6). Another non-linearity, which may also be considered, is that of the Poisson’s ratio $\nu$ (thus implicitly also of bulk modulus $K$). At higher stress levels (say at a stress state of some 50% below failure) and at low confinement (say at $0 \leq \sigma_3 < 2$ MPa) the Poisson’s ratio $\nu$ typically departs from its initial linear behaviour to significantly higher values, often to apparent values of $\nu >> 0.5$ (i.e. significant volume increase). Accommodation of such an effect in numerical modelling appears to be a possible strategy to simulate the rock deformations at and near the surface of underground excavations (convergence). So far, numerical modelling of the Mont Terri

Fig. 4-4. Deformation moduli of all relevant tests of the review period 2000 – 2008 as a function of the confining pressure $\sigma_3$. 
underground excavations often fell short of reproducing the observed convergence to an acceptable degree.

4.1.2 Short-term deformation parameters of Opalinus Clay bedding planes

Relevant parameters are the normal stiffness \( (k_n) \) and shear stiffness \( (k_s) \).

Definitions:

\[
\begin{align*}
    k_n &= \frac{\Delta\sigma_n}{\Delta u_n} \quad \text{[GPa/m]} \quad (13a) \\
    k_s &= \frac{\Delta\tau_s}{\Delta u_s} \quad \text{[GPa/m]} \quad (13b)
\end{align*}
\]

with:

\[
\begin{align*}
    \Delta\sigma_n &= \text{normal stress component acting normal to bedding plane} \\
    \Delta u_n &= \text{displacement component normal to bedding plane} \\
    \Delta\tau_s &= \text{shear stress component acting in direction of bedding plane} \\
    \Delta u_s &= \text{displacement component parallel to bedding plane}
\end{align*}
\]

Closely related with the normal and shear stiffnesses is the dilatation angle \( i \) of the bedding plane (ref. to Section 4.2.3)

Definition:

\[
    i = \tan^{-1} \left( \frac{\Delta u_n}{\Delta u_s} \right) \quad [^\circ] \quad (22)
\]

with:

\[
\begin{align*}
    \Delta u_n &= \text{normal displacement component of discontinuity when subject to shear (+ opening / - closure)} \\
    \Delta u_s &= \text{shear displacement of discontinuity}
\end{align*}
\]


A test example is presented in Fig. B-6 of Appendix B.

For results refer to Table 4-5.

<table>
<thead>
<tr>
<th>new results</th>
<th>Comparison: TR 2000-02</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>( k_n = 8 )</td>
<td>( 6 )</td>
<td>( no tests available )</td>
</tr>
<tr>
<td>( k_s = 10 )</td>
<td>( 6 )</td>
<td>( no tests available )</td>
</tr>
</tbody>
</table>

Tab. 4-5. Normal and shear stiffnesses of Opalinus Clay bedding planes
4.1.3 Long-term deformation parameters (creep)

Various creep laws have been proposed for the Opalinus Clay, amongst them the visco-plastic Lemaitre creep model for transient creep (Horseman et al., TN 2003-03) and a simple Norton power law, considering steady-state creep only (Zhang et al., TR 2007-02). TR 2000-02 noted the following creep law for a uniaxially loaded sample:

\[
\varepsilon_1(t) = \frac{2\sigma_1}{9K} + \frac{\sigma_1}{3G_2} + \frac{\sigma_1}{3G_1}[1 - e^{-\left(\frac{G_1}{\eta_1} t\right)}] + \frac{\sigma_1}{3\eta_2} t
\]

with:
- \(\varepsilon_1(t)\) = axial strain of uniaxial test
- \(\sigma_1\) = deviatoric stress
- \(K\) = bulk modulus with \(K = \frac{E}{3(1-2\nu)}\)
- \(G_1, G_2\) = shear moduli
- \(\eta_1, \eta_2\) = viscosity parameters

An alternative notation for Equation (14a) is:

\[
\varepsilon_1(t) = \varepsilon_e + \varepsilon_f [1 - \exp (-\frac{t}{T_1})] + \frac{\Delta e_1}{\Delta t} t
\]

with:
- 1st term = instantaneous (or elastic) strain \(\varepsilon_e\)
- 2nd term = primary consolidation
- 3rd term = secondary consolidation (creep proper)

The above law represents a material which shows an instantaneous (elastic) response together with a transient (primary) creep and steady-state (secondary) creep. The accelerated (tertiary) creep is not considered within this model.

Table 4-6 compiles some of the results on the steady-state (secondary) creep which have been found within the 2000 – 2008 source documents.

<table>
<thead>
<tr>
<th>Test conditions</th>
<th>Stress deviator (\Delta\sigma) [MPa]</th>
<th>Temp. [°C]</th>
<th>n</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>((3.2 \pm 1.4) \times 10^{-12})</td>
<td>13</td>
<td>30</td>
<td>3</td>
<td>TR 2007-05, Tab.2</td>
</tr>
<tr>
<td>((620 \pm 250) \times 10^{-12})</td>
<td>2.3 - 5.6</td>
<td>58</td>
<td>3</td>
<td>TR 2007-02, Fig. 3-27</td>
</tr>
</tbody>
</table>

Tab. 4-6. Selection of some new results on strain rate of steady-state (secondary) creep of the Opalinus Clay material
A comprehensive investigation on steady-state creep parameters of the Opalinus Clay material at Mont Terri was carried out by Lux et al. (2007). In general they found that a “wide scattering” of the steady-state creep parameter values obviously is “a typical feature” of claystone and that there is a relatively low dependency on the level of confining stresses (p. 42). Their investigations concluded in the diagram of Fig. 4-5 in which the strain rate $\varepsilon_1/\Delta t$ is graphed as function of the deviatoric stress.

Fig. 4-5. Range of the steady-state creep parameter values of the Opalinus Clay at Mont Terri as extrapolated from 8 creep tests MT-7 ... MT-16 under the assumption of the validity of the Hou/Lux creep law (modified after Lux et al. 2007)
Nagra (2002) has adopted a modified Salzer creep law (Salzer et al. 1998) for the Opalinus Clay. It writes as follows:

\[ \dot{\varepsilon}_\text{eff} = \alpha \cdot \frac{\beta \varepsilon_v \sigma_{\text{eff}}}{(\varepsilon_{\text{eff}}^v)^\mu} - \frac{\varepsilon_{\text{eff}}^v}{t_0} \] ....................................................... (15a)

or

\[ \varepsilon_v = \int_0^t \left( \alpha \cdot \frac{\beta \sigma_{\text{eff}}}{(\varepsilon_{\text{eff}}^v)^\mu} - \frac{\varepsilon_{\text{eff}}^v}{t_0} \right) \, dt \] ....................................................... (15b)

with:

- \( \varepsilon_v \) = creep rate [s\(^{-1}\)]
- \( \varepsilon_{\text{eff}} \) = creep strain [-]
- \( \sigma_{\text{eff}} \) = \( I_2 \equiv 2^{nd} \) invariant of stress tensor [Pa]
- \( \varepsilon_{\text{eff}}^v \) = \( 2^{nd} \) invariant of creep strain tensor [-]
- \( \mu \) = hardening parameter [-]
- \( t_0 \) = recovery parameter [s]
- \( \alpha \) = creep factor [Pa\(^{-1}\) s\(^{-1}\)]
- \( \beta \) = stress exponent [-]

If there is neither a hardening (i.e. \( \mu = 0 \)) nor a recovery (i.e. \( t_0 = \infty \)), the Salzer creep law reduces to:

\[ \varepsilon_v = \alpha \sigma_{\text{eff}}^\beta \] ....................................................... (15c)

Table 4-7 presents the parameter values of Nagra (2002).

<table>
<thead>
<tr>
<th>Creep parameter</th>
<th>( \beta ) [-]</th>
<th>( \alpha ) [Pa(^{-1}) s(^{-1})]</th>
<th>( t_0 ) [s]</th>
<th>( \mu ) [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Value</td>
<td>7</td>
<td>( 1 \times 10^{-35} )</td>
<td>( 5 \times 10^9 )</td>
<td>5.6</td>
</tr>
</tbody>
</table>

*Tab. 4-7. Creep parameter values of the Salzer model for the Opalinus Clay (out of Nagra, 2002)*

Comments on new test results which are presented in Tables 4-6 to 4-7:

C 4.15 The data base on Opalinus Clay creep parameters remains rather limited. Within the source documents there are only very few new tests on the long-term deformation behaviour of the Opalinus Clay. This is despite the fact that “due to its
importance for long-term safety aspects special attention ... [should be] paid to investigations concerning ... the reasons for viscoplastic deformation processes (primary, secondary and tertiary phase) to get a sound understanding of the physical mechanisms prior to constitutive modelling” (Jobmann et al., TN 2006-72, p. 14).

C 4.16 The derivation of the visco-elastic parameters $G_1$, $G_2$, $\eta_1$ and $\eta_2$ according to Equations (14 a) from the laboratory test curves requires an effort which is beyond the scope of this Technical Report.

4.2 Strength parameters

4.2.1 Unconfined (i.e. uniaxial) material strength parameters

4.2.1.1 Unconfined Compressive Strength UCS

**Definition:**

$$\text{UCS} = \frac{P_{\text{max}}}{A_0} = \frac{\sigma_{1\text{max}}}{\sigma_3 = \sigma_2 = 0} \quad \ldots \quad \text{(15)}$$

with:
- $P_{\text{max}} = \text{maximal compressive load on sample}$
- $A_0 = \text{initial cross-sectional area}$
- $\sigma_3 = \text{confining pressure}$

**Reference:** ISRM (Ulusay and Hudson, 2007, p. 151 –156).

For results ref. to Table 4-8.

<table>
<thead>
<tr>
<th>new results</th>
<th><strong>Comparison: TR 2000-02</strong></th>
<th>References for new results</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>UCS value [MPa]</strong></td>
<td><strong>UCS value [GPa]</strong></td>
<td>n</td>
</tr>
<tr>
<td>$\text{UCS}_{\parallel ss} = 11.6 \pm 3.9$</td>
<td>$\text{UCS}_{\parallel ss} = 13.4 \pm 4.3$</td>
<td>35</td>
</tr>
<tr>
<td>$\text{UCS}_{\perp ss} = 14.9 \pm 5.1$</td>
<td>$\text{UCS}_{\perp ss} = 25.6 \pm 2.5$</td>
<td>15</td>
</tr>
</tbody>
</table>

Tab. 4-8. *Unconfined Compressive Strength UCS as deduced from short-term laboratory tests. New results (left) and comparison with data base of 2000 (right).

*) The original $\text{UCS}_{\parallel ss}$ value of $10.5 \pm 6.5$ MPa, as specified in TR 2000-02, is erroneous due to a printing error.
Comments:

C 4.17 As in the 2000 review, UCS//ss turns out to be smaller than UCS ⊥ ss. This effect seems to be related to different failure mechanisms (ref. to fig. 4-4 of TR 2000-02).

C 4.18 The test result of UCS//ss < UCS ⊥ ss is in marked contrast to that situation where samples are subjected to confining pressures (Popp and Salzer, TR 2007-04). Under confined conditions the strength of Opalinus Clay P-samples tends to be higher than for S-samples.

4.2.1.2 Indirect Tensile (“Brazilian”) Strength UTS

Definition: \[ UTS = 0.636 \cdot \frac{P_f}{D \cdot l} \text{ [MPa]} \] .......................... (16)

with:  
\[ P_f \] = Load at failure [N]
\[ D \] = diameter of the test specimen [mm]
\[ l \] = length of the sample axis [mm]


For results ref. to Table 4-9.

<table>
<thead>
<tr>
<th>new results</th>
<th>( UTS_{// ss} = 1.3 \pm 0.2 )</th>
<th>TR 2003-04</th>
</tr>
</thead>
<tbody>
<tr>
<td>( UTS_{// ss} ) value [MPa]</td>
<td>n = 5</td>
<td>( UTS_{// ss} ) = ~ 2</td>
</tr>
<tr>
<td>( UTS_{\perp ss} ) = 0.67</td>
<td>n = 2</td>
<td>( UCS_{\perp ss} ) ~ 1</td>
</tr>
</tbody>
</table>

Tab. 4-9. Unconfined Tensile Strength UTS as deduced from short-term indirect (“Brazilian”) tensile tests. New results (left) and comparison with data base of 2000 (right).

Comment:

C 4.19 The UTS-values of TR 2000-02 were estimated from hydraulic fracture initiation pressure tests of mini core samples with a central 3 mm \( \varnothing \) injection borehole.
4.2.2 Strength parameters in general stress conditions

4.2.2.1 Mohr-Coulomb parameters for intact material

Definition: \[ \tau = c' + \sigma_n' \cdot \tan \phi' \] \hspace{1cm} (17a)

with:
- \( \tau \): shear strength
- \( c' \): effective cohesion
- \( \phi' \): effective angle of internal friction [°] (for material)
- \( \sigma_n' \): effective normal stress

The Mohr-Coulomb criterion can be considered to be a linear approximation of the limiting conditions of all possible stress states at which the strength of the material is exceeded. Such conditions can be graphically represented in the Mohr- \((\tau-\sigma)\) diagram (Fig. 4-6, left), or alternatively in the principal stress \((\sigma_1-\sigma_3)\) diagram (Fig. 4-6, right).

\[
\sin \phi' = \frac{(\tan \beta - 1)}{(\tan \beta + 1)} \hspace{1cm} (18a)
\]
\[
c' = \text{UCS} \cdot \frac{(1 - \sin \phi')}{(2 \cdot \cos \phi')} \hspace{1cm} (18b)
\]

or
\[
\tan \beta = \frac{(1 + \sin \phi')}{(1 - \sin \phi')} \hspace{1cm} (19a)
\]
\[
\text{UCS} = 2 \cdot c \cdot \cos \phi' / (1 - \sin \phi') \hspace{1cm} (19b)
\]

Fig. 4-6. Alternative graphical representation of the Mohr-Coulomb strength criterion: (a) Left: Mohr diagram (b) Right: Principal stress diagram
with:  
\[ \phi' = \text{angle of inclination of the strength line in the } \tau - \sigma \text{ diagram} \]  
\[ c' = \text{intercept of the strength line with the } \tau \text{-axis in the } \tau - \sigma \text{ diagram} \]  
\[ \beta = \text{angle of inclination of the strength line in the } \sigma_1 / \sigma_3 \text{ diagram} \]  
UCS = \text{intercept of the strength line with the } \sigma_1 \text{-axis in the } \sigma_1 / \sigma_3 \text{ diagram}  
\[ (= \text{unconfined compressive strength}) \]

Brady and Brown (1985, p.105 ff)

For results refer to Table 4-10 and Fig. 4-7.

<table>
<thead>
<tr>
<th>Sample type</th>
<th>new results</th>
<th>Comparison: TR 2000-02</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Mohr-Coulomb failure parameters for Opalinus Clay material</td>
<td>Failure parameters for Opalinus Clay material</td>
</tr>
<tr>
<td></td>
<td>n</td>
<td>ref.</td>
</tr>
<tr>
<td>P</td>
<td>( c_{//ss'} = 5.4 \text{ [MPa]} )</td>
<td>41</td>
</tr>
<tr>
<td></td>
<td>( \phi_{//ss'} = 23 \text{ [°]} )</td>
<td>Tab 4-1, Column 27</td>
</tr>
<tr>
<td>S</td>
<td>( c_{\perp ss'} = 3.7 \text{ [MPa]} )</td>
<td>39</td>
</tr>
<tr>
<td></td>
<td>( \phi_{\perp ss'} = 22 \text{ [°]} )</td>
<td>Tab. 4-10</td>
</tr>
</tbody>
</table>

Tab. 4-10. Mohr-Coulomb strength parameters of the Opalinus Clay material as deduced from laboratory tests. New results (left) and comparison with data base of 2000 (right).

Comments:

C 4.20 Naumann and Plütschke (2005) found that specimens subjected to a temperature of 80°C had a slightly higher strength than those tested at ambient temperature. Schnier and Stührenberg (TR 2003-04) found the contrary (about 20 to 25% lower strength values at T= 80°C compared with room temperature). Generally, however, there is agreement that the influence of temperature on strength is not as significant as the influence from bedding (P- and S-samples) or from confining pressure \( \sigma_3 \).
The regression of the laboratory test results in the stress diagram Fig. 4-7 yielded an $\sigma_1$-intercept of 16.2 MPa for the P-samples and of 11.0 MPa for S-samples. The comparative UCS-values are $11.6 \pm 3.9$ MPa (P) and $14.9 \pm 5.1$ MPa (S), respectively (Tab. 4-8). This observation supports the assumption that linear Mohr-Coulomb failure lines might not adequately describe the strength properties of the Opalinus Clay material. Commonly, the inadequacy of such linear lines is physically explained by different failure modes occurring at distinct levels of confinement: At low confining pressures axial fractures in combination with local shear fractures and/or spalling are characteristic of the failure mode (see fig. 4-4 of TR 2000-02), whereas at higher confining pressures shear fractures prevail in the failure mode.

Against this background, non-linear Mohr failure envelopes might be considered. A first approximation in this regard is the bi-linear failure curve. Kaiser and Kim (2008) considered such a bi-linear curve a minimum requirement for the characterisation of the strength properties of brittle rocks.

Table 4-11 provides the bi-linear strength parameters for the Opalinus Clay at Mont Terri. In line with TN 2005-25 (p. 34) the transition point between the two linear sections was taken at $\sigma_3 = 5$ MPa.
### Tab. 4-11. Bi-linear approximation of the Mohr-Coulomb strength parameters of the Opalinus Clay material as deduced from laboratory tests.

<table>
<thead>
<tr>
<th>Sample type</th>
<th>Parameter</th>
<th>Range $0 \leq \sigma_3 \leq 5$ MPa</th>
<th>Range $5 &lt; \sigma_3 \leq 30$ MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>P</td>
<td>$c_{//ss'}$ [MPa]</td>
<td>3.1 [MPa]</td>
<td>6.3 [MPa]</td>
</tr>
<tr>
<td></td>
<td>$\phi_{//ss'}$</td>
<td>34°</td>
<td>21°</td>
</tr>
<tr>
<td>S</td>
<td>$c_{\perp ss'}$ [MPa]</td>
<td>4.0 [MPa]</td>
<td>5.5 [MPa]</td>
</tr>
<tr>
<td></td>
<td>$\phi_{\perp ss'}$ [°]</td>
<td>25°</td>
<td>18°</td>
</tr>
</tbody>
</table>

### 4.2.2.2 Mohr-Coulomb parameters for bedding planes

**Definition:**

$$\tau = c' + \sigma_n' \cdot \tan \phi' \quad \text{.........................} \ (17b)$$

with:

- $\tau$ = shear strength of bedding plane
- $c'$ = effective cohesion of bedding plane
- $\phi'$ = effective angle of friction [°] of bedding plane
- $\sigma_n'$ = effective normal stress

For results refer to Table 4-12 and Fig. B-7 of Appendix B.

### Tab. 4-12. Mohr-Coulomb strength parameters of the Opalinus clay material as deduced from laboratory tests. New results (left) and comparison with database of 2000 (right).

<table>
<thead>
<tr>
<th>Sample and test type</th>
<th>new results</th>
<th>Comparison: TR 2000-02</th>
</tr>
</thead>
<tbody>
<tr>
<td>Z</td>
<td></td>
<td><strong>Failure parameters of Opalinus Clay bedding planes</strong></td>
</tr>
<tr>
<td>Direct shear</td>
<td></td>
<td>$n$ ref.</td>
</tr>
<tr>
<td></td>
<td>$c' = 0.94$ [MPa]</td>
<td>$n$</td>
</tr>
<tr>
<td></td>
<td>$\phi' = 21$ [°]</td>
<td>$c_{z'} = 1$ [MPa]</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$\phi_{z'} = 23$ [°]</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Tab. 4-1, Column 28</td>
</tr>
</tbody>
</table>

Tab. 4-11. Bi-linear approximation of the Mohr-Coulomb strength parameters of the Opalinus Clay material as deduced from laboratory tests.

Tab. 4-12. Mohr-Coulomb strength parameters of the Opalinus clay material as deduced from laboratory tests. New results (left) and comparison with data base of 2000 (right).
Comments:

C 4.21 The evaluation of the bedding plane shear strength is based on a number of direct shear tests carried out by Popp and Salzer (TR 2007-04).

C 4.22 Z-samples were not included in the evaluation of the bedding plane strength parameters. Within the documents reviewed, there was neither accurate enough information available on the inclination of bedding towards the sample axis nor was there clarity as to the correctness of the parameter transformation from the principal stress diagram to the Mohr-diagram.

C 4.23 A bi-linear approximation of the bedding plane strength may be deduced from Fig. B-7 in Appendix B.

4.2.2.3 Hoek-Brown material strength parameters

Definition:

\[
\frac{\sigma_1'}{UCS} = \frac{\sigma_3'}{UCS} + \left[ m \cdot \frac{\sigma_3'}{UCS} + s \right]^{\frac{1}{2}} \quad \ldots \quad (20a)
\]

\[
T_0 = \frac{UCS}{2} \left[ m - (m^2 + 4s)^{\frac{1}{2}} \right] \quad \ldots \quad (20b)
\]

with:

UCS = unconfined compressive strength of the intact rock

T_0 = unconfined tensile strength of the rock mass

m and s = material constants (s = 1 for intact rock)

References: Hoek and Brown (1980)
Hoek and Marinos (2007)

For results refer to Fig. 4-8.

There is a considerable scatter of the test data in the Hoek-Brown stress diagram. In these circumstances an empirical fitting procedure, as shown in Fig. 4-8, seemed to be more appropriate than an apparently precise parameter evaluation. It can be depicted that the parameter “m” lies between about 1 and 5 for S-samples and between 5 and 10 for P-samples.

Comments:

C 4.24 The Hoek and Brown (1980) strength criterion is an empirical criterion for the design of underground excavations in rocks. It
was originally developed to overcome one of the shortcomings of the linear Mohr-Coulomb criterion by introducing a higher-term strength criterion. However, the underpinning argument is now of reduced validity as contemporary computer codes can cope with almost any form of the Mohr-Colulomb strength curve by considering a set of multiple linear segments.

C 4.25 For comparison, Zhang et al. (2004) deduced for the Callovo-Oxfordian argillite of Bure the following Hoek and Brown strength parameters: \( m = 2.5 \), and \( s = 1 \). Hoek et al. (1995) recommend for “claystone” in general \( m = 4 \) and \( s = 1 \).

![Graph of laboratory test results](image)

**Fig. 4-8.** Laboratory test results graphed in the dimensionless stress diagram used for the delineation of the Hoek-Brown strength parameters with lines for selected values of material parameter \( m \) (\( m = 1, 5, 10, 20 \) and 30) and \( s = 1 \).

4.2.3 Dilatation

- Dilatation of the material (volumetric strain \( \delta \))
Definition: \[ \delta = \frac{\Delta V}{V} = \varepsilon_{\text{ax}} + 2 \cdot \varepsilon_{\text{lat}} = \varepsilon_{\text{ax}} (1 - 2 \nu) \] ........................ (21)

with: \( \Delta V \) = volume increase (+) of an element
\( V \) = original volume
\( \varepsilon_{\text{ax}} \) = axial strain
\( \varepsilon_{\text{lat}} \) = lateral strain
\( \nu \) = Poisson’s ratio

- Dilatation of discontinuities (dilatation angle \( i \))

Definition \[ i = \tan^{-1} \left( \frac{\Delta n}{\Delta s} \right) \ [^\circ] \] ................................. (22)

with: \( \Delta n \) = normal displacement of discontinuity when subject to shear (+ opening / - closure)
\( \Delta s \) = shear displacement of discontinuity

References: ISRM (Ulusay and Hudson, 2007, p. 165 –176)

For results of the volume dilatancy \( \delta \) of the Opalinus Clay material refer to Table 4-13 and Fig. B-8 of Appendix B. For results of the dilation angle \( i \) of the bedding planes refer to Table 4-14.

<table>
<thead>
<tr>
<th>( \sigma_3 ) [MPa]</th>
<th>new results</th>
<th>Comparison: TR 2000-02</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Volume dilatancy ( \delta = \frac{\Delta V}{V} [10^{-3}] )</td>
<td>( \delta = \frac{\Delta V}{V} [10^{-3}] )</td>
</tr>
<tr>
<td></td>
<td>n ref.</td>
<td>P</td>
</tr>
<tr>
<td>----------------------</td>
<td>-------</td>
<td>---</td>
</tr>
<tr>
<td>0</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>2</td>
<td>-1,1</td>
<td>-3,2</td>
</tr>
<tr>
<td>3</td>
<td>-1,8</td>
<td>-2,9</td>
</tr>
<tr>
<td>5</td>
<td>-2,4</td>
<td>-</td>
</tr>
<tr>
<td>10</td>
<td>-3,9</td>
<td>-2,9</td>
</tr>
</tbody>
</table>

Tab. 4-13. Volume dilatancy at peak strength of the Opalinus Clay material as deduced from laboratory tests. New results (left) and comparison with data base of 2000 (right). Note: negative values = volume decrease
new results

<table>
<thead>
<tr>
<th>( \sigma_3 ) [MPa]</th>
<th>Dilatancy angle ( i ) [°]</th>
<th>( n )</th>
<th>Comparison: TR 2000-02</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0 ± 2 [°] (at ( \sigma_3 = 10 ) MPa)</td>
<td>10</td>
<td>Table 4.1, Column 29</td>
</tr>
<tr>
<td>6</td>
<td>-5 to 35 [°]</td>
<td>6</td>
<td></td>
</tr>
</tbody>
</table>

Tab. 4-14. Dilatancy angle \( i \) of Opalinus Clay bedding planes as deduced from direct shear laboratory tests of Popp and Salzer (TR 2007-04). New results (left) and comparison with data base of 2000 (right). Note: negative angle = down gliding (= compaction of shear plane)

4.3 Permeability parameters

- Hydraulic conductivity \( K \)

  Definition \[ K = \frac{Q \cdot \Delta x}{(\Delta h \cdot A)} \] [m / s] \hspace{1cm} (23)

  with:
  - \( Q \) = flow rate [m³ / s]
  - \( \Delta x \) = length increment [m]
  - \( \Delta h \) = hydraulic head increment [m]
  - \( \Delta h / \Delta x = \) “hydraulic gradient” [°]
  - \( A \) = cross-sectional area of the flow path [m²]

- (Specific or Intrinsic) Permeability \( k \)

  Definition \[ k = \frac{(Q \cdot \mu \cdot \Delta x)}{(\Delta p \cdot A)} \] [m²] \hspace{1cm} (24)

  with:
  - \( Q \) = flow rate [m³ / s]
  - \( \mu \) = dynamic viscosity of the fluid [N · s / m²]
  - \( \Delta x \) = length increment [m]
\[ \Delta p = \text{fluid pressure increment} \ [\text{N} / \text{m}^2] \]

\[ A = \text{cross-sectional area of the flow path} \ [\text{m}^2] \]

The parameters \( K \) and \( k \) are interrelated as follows:

\[ K = k \cdot \rho \cdot g / \mu \] ............................................... (25)

with:

\[ \rho = \text{density of fluid} \ [\text{kg} / \text{m}^3] \]

\[ g = \text{gravitational acceleration} \ [\text{N} / \text{kg}] \text{ or } [\text{m} / \text{s}^2] \]

For water at an ambient temperature of 20° C (\( \mu = 10^{-3} \text{ Pa} \cdot \text{s} \)), it is:

\[ K = 10^7 \cdot k \] ............................................... (26)


For results of the hydraulic conductivity \( K \) refer to Table 4-15.

<table>
<thead>
<tr>
<th>Orientation</th>
<th>Hydraulic conductivity ( K ) [m/s]</th>
<th>( n )</th>
<th>ref.</th>
<th>Hydraulic conductivity ( K ) [m/s]</th>
<th>( n )</th>
</tr>
</thead>
<tbody>
<tr>
<td>isotropic</td>
<td>( 2 \cdot 10^{-13} )</td>
<td>19</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( \perp ) (S-sample)</td>
<td>( 0.6 - 0.7 \cdot 10^{-13} )</td>
<td>2</td>
<td></td>
<td>( 0.6 \cdot 10^{-13} )</td>
<td>1</td>
</tr>
<tr>
<td>( \parallel ) (P-sample)</td>
<td>( 1.3 - 2 \cdot 10^{-13} )</td>
<td>2</td>
<td>Table 4.1 Column 30</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>( \angle ) (Z-sample)</td>
<td>-</td>
<td>0</td>
<td></td>
<td>( 2 \cdot 10^{-13} )</td>
<td>2</td>
</tr>
<tr>
<td>isotropic</td>
<td>( 0.2 - 9 \cdot 10^{-13} ) (sandy facies)</td>
<td>-</td>
<td>&quot;numerous&quot;</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>( 0.2 - 20 \cdot 10^{-13} ) (clayey facies)</td>
<td>-</td>
<td></td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>( \perp )</td>
<td>( 0.6 - 1.2 \cdot 10^{-13} )</td>
<td>-</td>
<td>Nagra (2002)</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>( \angle )</td>
<td>( 1 - 2 \cdot 10^{-13} )</td>
<td>-</td>
<td></td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Tab. 4-15. Hydraulic conductivity \( K \) of the Opalinus Clay material as deduced from laboratory and in-situ tests and numerical back analyses. New results (left) and comparison with data base of 2000 (right). Note: Permeability \( k \) values were transferred to \( K \) according to Equation (26).
Comments:

C 4.26 There is not much change in the values compared to the TR 2000-02 review. This may be taken as a sign of a matured knowledge of the hydraulic permeability parameters of the undisturbed (intrinsic) Opalinus Clay material.

C 4.27 Hydraulic conductivity is substantially higher and somehow irregularly distributed within the EDZ (e.g. TN 2004-56; TN 2006-72). Figure B-9 in Appendix B gives an indication on the distribution of the hydraulic conductivity in the sidewall rock of a tunnel and the change of K with time.

C 4.28 Permeability is not significantly affected by elevated temperatures (HE-D Experiment; TR 2006-01 and TR 2007-02).

4.4 Hydro-mechanically coupled parameters

4.4.1 Consolidation parameter (one-dimensional)

Definition \[ e = e_0 - C_c \cdot \ln \left( \frac{\sigma'}{\sigma_0'} \right) \] ................................. (27)

with: \[ e = \text{void ratio [-]} \]
\[ C_c = \text{Compression index (primary consolidation) [-]} \]
\[ \sigma' = \text{effective stress acting on specimen [MPa]} \]
\[ e_0 = \text{void ratio intercept at } \sigma_0' = 1 \text{ MPa [-]} \]

Reference: Terzaghi (2005)
Oedometer test: DIN 18 135

For results refer to Table 4-16.

<table>
<thead>
<tr>
<th>Range of $\sigma'$ [MPa]</th>
<th>new results</th>
<th>TR 2000-02</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$C_c$ [-]</td>
<td>n</td>
</tr>
<tr>
<td>7.3 – 29.3</td>
<td>0.017 ± 0.01</td>
<td>11</td>
</tr>
</tbody>
</table>

Tab. 4-16. Compression index $C_c$ of the Opalinus Clay material as deduced from a laboratory test. Type of sample not specified (?) S-sample.)
4.4.2 Swelling parameters

- **Swelling Strain Index $S_\varepsilon$**

  **Definition**
  
  $S_\varepsilon = \left( \frac{d}{L} \right) \cdot 100 \quad [\%] \quad \ldots \quad (28)$

  at a surcharge pressure $\sigma_n = \text{constant}$

  with:
  
  $d = \text{maximum swelling displacement} \quad [\text{m}]$

  $L = \text{initial specimen thickness before swelling} \quad [\text{m}]$

  $\sigma_n = \sigma_0 + \sigma_s \quad [\text{MPa}]$

  with

  $\sigma_0 = \text{pre-loading pressure}$

  $\sigma_s = \text{swelling pressure}$


  $S_\varepsilon$ is defined for a specific (= constant) surcharge pressure $\sigma_n$ at zero lateral strain (uniaxial strain) conditions.

  No new results are available to enhance the state of knowledge documented in TR 2000-02 (Table 4-17).

<table>
<thead>
<tr>
<th>$\sigma_0$ [MPa]</th>
<th>$\sigma_s$ [MPa]</th>
<th>$n$</th>
<th>Sample direction</th>
<th>Swelling strain index (*)</th>
<th>Opalinus clay material</th>
<th>References</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.05</td>
<td>$\sim 0.45$</td>
<td>5</td>
<td>$\perp \text{ss}$</td>
<td>$S_{\varepsilon \perp \text{ss}} = 7 \pm 2 \quad [%]$</td>
<td></td>
<td>TN 96-22</td>
</tr>
<tr>
<td>0.05</td>
<td>$\sim 0.12$</td>
<td>4</td>
<td>$// \text{ss}$</td>
<td>$S_{\varepsilon // \text{ss}} = \sim 1 \quad [%]$</td>
<td></td>
<td>TN 97-06</td>
</tr>
</tbody>
</table>

**Tab. 4-17** Swelling strain index $S_\varepsilon$ of the silty-shaly facies of the Opalinus clay at a specific swelling pressure $\sigma_s$ as deduced from laboratory tests (out of TR 2000-02). (*) estimated value only.

- **Swelling Pressure $\sigma_s$**

  **Definition**
  
  $\sigma_s = \sigma_{\text{max}} \quad [\text{MPa}] \quad \ldots \quad (29)$

  with:

  $\sigma_{\text{max}} = \text{maximum swelling pressure for a defined amount of expansion (swelling strain)}$

  $\sigma_s$ is defined for a specific (= constant) amount of swelling strain at zero lateral strain conditions (uniaxial strain).

No new results are available to enhance the state of knowledge documented in TR 2000-02 (Table 4-18).

<table>
<thead>
<tr>
<th>$p_0$ [MPa]</th>
<th>n</th>
<th>Sample direction</th>
<th>Swelling pressure $p_s$ (*)</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.4</td>
<td>4</td>
<td>⊥ ss</td>
<td>$p_s \perp ss = 1.2$ [MPa]</td>
<td>TN 97-06</td>
</tr>
<tr>
<td>5.5</td>
<td>4</td>
<td>// ss</td>
<td>$p_s // ss = 0.6$ [MPa]</td>
<td></td>
</tr>
</tbody>
</table>

*Tab. 4-18 Swelling pressure $p_s$ of the silty-shaly facies of the Opalinus as deduced from laboratory tests. $p_o = \text{pre-loading pressure (out of TR 2000-02).}$ (*) estimated value only.*

Comments:

C 4.29 Wileveau und Rothfuchs (TR 2006-01) and Zhang et al. (TR 2007-02) document some swelling and shrinking tests under triaxial stress conditions. Due to different boundary conditions ($\varepsilon_{\text{lateral}} \neq 0$) their tests results are not directly comparable to those listed in Tabs. 4-17 and 4-18. For an effective confining pressure of $p' = \sigma_3 = 1$MPa, they found a volumetric decrease of 1.6% (over a time span of 4 months) for the shrinking phase, and a volumetric increase of 2.4% (over a time span of 6 months) for the swelling phase.

C 4.30 For unconfined conditions ($\sigma_3 = 0; \varepsilon_{\text{lateral}} \neq 0$) Wolter (TN 2002-46) found an extremely anisotropic swelling and shrinking behaviour (ref. to Fig. B-11 in Appendix B and to Comment C 5.3 of Section 5.1).

4.4.3 Dependency of parameter values on water content $w$

In TR 2000-02 it was stated that “the deformational and strength parameters are sensitively dependent on the moisture content $w$ of the Opalinus Clay”. That statement is fully reconfirmed by the current review.

Moreover, that dependency can be generalised across virtually all relevant geotechnical parameters. Figure B-12 in Appendix B gives an example for the thermal conductivity $\lambda$ (see Section 4.5.3 below). Figure B-13 provides an update on the dependency of the strength parameters.
4.5 Thermal parameters

4.5.1 Linear thermal expansion coefficient $\alpha$

Definition $\alpha = \varepsilon / \Delta T \quad [K^{-1}] \quad \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots (30)$

with: $\varepsilon$ = axial strain [-]
$\Delta T$ = temperature difference [K]

Reference: Jobmann et al. (TN 2006-72)

For results refer to Table 4-19.

<table>
<thead>
<tr>
<th>Heating path</th>
<th>New results</th>
<th>TR 2000-02</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\alpha \quad [10^{-6} \text{ K}^{-1}]$</td>
<td>n</td>
</tr>
<tr>
<td>$P_\parallel$-Sample</td>
<td>$1.4 \pm 0.4$</td>
<td>28</td>
</tr>
<tr>
<td>$S_\perp$-Sample</td>
<td>$1.0 \pm 0.2$</td>
<td>4</td>
</tr>
<tr>
<td>$P_\parallel$</td>
<td>not considered</td>
<td></td>
</tr>
<tr>
<td>$S_\perp$</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Tab. 4-19. Linear thermal coefficient $\alpha$ of the Opalinus Clay material as deduced from laboratory tests and numerical back analyses. New results (left) and comparison with data base of 2000 (right).

Comment:

C 4.31 Jobmann et al. (TN 2006-72) emphasised that for the Opalinus Clay the assumption of a constant thermal expansion coefficient is only valid for temperatures below $\sim 40^\circ C$ (ref. to Fig. B-10 in Appendix B).

4.5.2 Specific heat capacity $c_p$

Definition $c_p = \Delta H / (m \cdot \Delta T) \quad [J \cdot \text{kg}^{-1} \cdot \text{K}^{-1}] \quad \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots (31)$

with: $\Delta H$ = heat change [J]
$m$ = mass of body subject to heat change [kg]
$\Delta T$ = temperature difference [K$^{-1}$]

Reference: Jobmann et al. (TN 2006-72)
For results refer to Table 4-20.

<table>
<thead>
<tr>
<th>new results</th>
<th>TR 2000-02</th>
</tr>
</thead>
<tbody>
<tr>
<td>at T</td>
<td>References</td>
</tr>
<tr>
<td>20°C</td>
<td>Tab. 4-1, Column 33</td>
</tr>
<tr>
<td>80°C</td>
<td>not considered</td>
</tr>
<tr>
<td>n.sp.</td>
<td>Nagra (2002)</td>
</tr>
</tbody>
</table>

Tab. 4-20. Specific heat capacity $c_p$ of the Opalinus Clay material as deduced from laboratory tests. New results (left) and comparison with data base of 2000 (right). n. sp. = not specified

4.5.3 Thermal conductivity $\lambda$.

Definition

$$\lambda = \frac{Q}{(\Delta T / \Delta x)} \quad [W \cdot m^{-1} \cdot K^{-1}] \quad ................. \ (30a)$$

with:

- $Q$ = heat flow density $[W/m^2]$ or $[J/(m^2 \cdot s)]$
- $\Delta T$ = temperature difference $[K]$
- $\Delta x$ = distance $[m]$

Reference: Jobmann et al. (TN 2006-72)

For results refer to Table 4-21.

<table>
<thead>
<tr>
<th>new results</th>
<th>TR 2000-02</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\lambda$</td>
<td>References</td>
</tr>
<tr>
<td>$\lambda_{P/\perp}$</td>
<td>$\lambda_{S/\perp}$</td>
</tr>
<tr>
<td>$P/\perp$-Sample</td>
<td>$S/\perp$-Sample</td>
</tr>
<tr>
<td>2.25 ± 0.40</td>
<td>9</td>
</tr>
<tr>
<td>2.04 ± 0.23</td>
<td>3</td>
</tr>
</tbody>
</table>

Tab. 4-21. Thermal conductivity $\lambda$ of the Opalinus Clay material as deduced from laboratory tests and numerical back analyses. New results (left) and comparison with data base of 2000 (right).
5 Degree of Anisotropy, Rock Mass Classification and Scale Effect

In recent years increased attention has been paid to bedding anisotropy of the Opalinus Clay. Also some thought has been given to common rock mass classification systems and to their potential applicability to Mont Terri. Furthermore, there is an ongoing debate on possible scale effects. All three aspects will be briefly reviewed in the following Sections 5.1 to 5.3.

5.1 Bedding anisotropy

Table 5-1 lists the source documents which contain some new information on the degree of anisotropy of the Opalinus Clay at Mont Terri.

<table>
<thead>
<tr>
<th>Source Document (new)</th>
<th>Information on the degree of anisotropy</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1 2 3 4 5 6 7 8 9 10 11 12</td>
</tr>
<tr>
<td>A.1.1 TR 2003-04</td>
<td>x x x x x</td>
</tr>
<tr>
<td>A.1.2 TR 2006-01</td>
<td>x</td>
</tr>
<tr>
<td>A.1.3 TR 2007-02</td>
<td></td>
</tr>
<tr>
<td>A.1.4 TR 2007-04</td>
<td></td>
</tr>
<tr>
<td>A.2.3 TN 2000-11</td>
<td>x</td>
</tr>
<tr>
<td>A.2.10 TN 2002-46</td>
<td>x x x x</td>
</tr>
<tr>
<td>A.2.12 TN 2002-50</td>
<td>x</td>
</tr>
<tr>
<td>A.2.16 TN 2004-37</td>
<td></td>
</tr>
<tr>
<td>A.2.17 TN 2004-38</td>
<td></td>
</tr>
<tr>
<td>A.2.19 TN 2004-86</td>
<td>x</td>
</tr>
<tr>
<td>A.2.21 TN 2005-25</td>
<td>x x x x x</td>
</tr>
<tr>
<td>A.2.26 TN 2006-37</td>
<td></td>
</tr>
<tr>
<td>A.2.28 TN 2006-72</td>
<td></td>
</tr>
<tr>
<td>A.2.30 TN 2007-30</td>
<td></td>
</tr>
<tr>
<td>A.3.1 Wenk et al.</td>
<td></td>
</tr>
<tr>
<td>A.3.2 Gens et al.</td>
<td></td>
</tr>
<tr>
<td>A.3.6 Naumann et al.</td>
<td></td>
</tr>
</tbody>
</table>

Tab. 5-1. Information on degree of bedding anisotropy.

Legend

- x Accepted for parameter value evaluation
- (x) Rejected for parameter value evaluation

1 Textural (mineralogy)  
2 Structural (rock mechanics)  
3 Seismic velocity \( v_p \)
4 Mechanic: \( E \) and \( \nu \)  
5 Mechanic: UCS  
6 Mechanic: UTS  
7 Mechanic: 3-ax strength  
8 Hydraulic: \( K \)  
9 Thermal: \( \alpha \) and/or \( \lambda \)  
10 De-/ resaturation, swelling  
11 Strain relaxation  
12 In-situ stress state

As a consistency check Table 5-2 compiles the various anisotropy factors from the actual data presented in the respective tables of this Report.
<table>
<thead>
<tr>
<th>Parameter</th>
<th>Parameter Value</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Seismic velocity</td>
<td>$v_p$ 1.76</td>
<td>Tab. 3-7</td>
</tr>
<tr>
<td></td>
<td>$v_s$ 1.72</td>
<td></td>
</tr>
<tr>
<td>Secant Modulus $E_{50}$</td>
<td>2.9</td>
<td>Tab. 4.3</td>
</tr>
<tr>
<td>Young's Modulus $E$</td>
<td>2.5</td>
<td></td>
</tr>
<tr>
<td>Poisson's ratio $\nu$</td>
<td>1.4</td>
<td>Tab. 4.4</td>
</tr>
<tr>
<td>Joint stiffness $K_d/K_n$</td>
<td>1.25</td>
<td>Tab. 4-5</td>
</tr>
<tr>
<td>UCS</td>
<td>0.78</td>
<td>Tab. 4-8</td>
</tr>
<tr>
<td>UTS</td>
<td>1.9</td>
<td>Tab. 4-9</td>
</tr>
<tr>
<td>Cohesion $c'$</td>
<td>1.5</td>
<td>Tab. 4-10</td>
</tr>
<tr>
<td>Friction angle $\phi'$</td>
<td>1.0</td>
<td></td>
</tr>
<tr>
<td>Volume dilatancy $\Delta V/V$</td>
<td>$\sigma_3=2$ MPa 0.3</td>
<td>Tab. 4-13</td>
</tr>
<tr>
<td></td>
<td>$\sigma_3=3$ MPa 0.6</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$\sigma_3=10$ MPa 1.3</td>
<td></td>
</tr>
<tr>
<td>Hydraulic conductivity $K$</td>
<td>~ 2.5</td>
<td>Tab. 4-15</td>
</tr>
<tr>
<td>Swelling strain index $S_e$</td>
<td>~ 7</td>
<td>Tab. 4-17</td>
</tr>
<tr>
<td>Swelling pressure</td>
<td>2.0</td>
<td>Tab. 4-18</td>
</tr>
<tr>
<td>Strain relaxation desaturation</td>
<td>5.4</td>
<td>TN 2002-46</td>
</tr>
<tr>
<td></td>
<td>7.0</td>
<td></td>
</tr>
<tr>
<td>Linear thermal coefficient $\alpha$</td>
<td>heating 1.4</td>
<td>Tab. 4-19</td>
</tr>
<tr>
<td></td>
<td>cooling 2.5</td>
<td></td>
</tr>
<tr>
<td>Thermal conductivity $\lambda$</td>
<td>1.9</td>
<td>Tab. 4-21</td>
</tr>
<tr>
<td>In-situ stress measurement</td>
<td>~ 3.6</td>
<td>TN 2004-86</td>
</tr>
<tr>
<td></td>
<td>&quot;mean&quot;: 2.4 (\pm 1.8)</td>
<td></td>
</tr>
</tbody>
</table>

*Tab. 5-2  Bedding anisotropy factor AF of the H-T-M parameters of the Opalinus Clay*
Comments:

C 5.1 The anisotropy factor AF is, with some major exemptions, generally at a value of about 2.5. This means that in direction of bedding there is generally a higher stiffness, higher strength, higher hydraulic and thermal conductivity and a lower linear thermal expansion coefficient than across bedding.

C 5-2 Non-typically low AF values, even below 1, are associated with the UCS and the material dilatancy (= volume change) at relatively low confining stresses. This phenomenon, however, is fully plausible. In rock mechanics it is well established that loading of samples in direction of bedding (= “P-sample”) is much more sensitive in producing axial cracks and opening of the pre-existing bedding planes (thus producing a marked volume increase) than loading across bedding.

C 5.3 Other non-typical AF values, however now on the high side, are associated with swelling and de- and resaturation processes. An AF number of up to 7 underlines the particular sensitivity of these processes towards the existence of bedding plane structures in the Opalinus Clay.

C 5-4 The AF value of ~3.6 of the in-situ stresses (last line in Tab. 5-2), as evaluated by CSIRO strain cell measurements, is possibly due to the specific strain-stress transformation procedures employed in the stress evaluation which did not account for the transverse isotropic character of the Opalinus Clay rock.

5.2 Rock mass classification systems

One of the explicitly stated mission topics of this review is to “estimate the rock mass behaviour based on alternative rock mass classification systems (e.g. Q-System after Barton, RMR after Bieniawski)”.

The principal idea of any rock mass classification system is to classify experience so that it may be extrapolated from one site to another (“processed experience”). Classification schemes seek to assign
numerical values to those properties or features of the rock mass considered likely to influence its behaviour, and to combine these individual values into one overall classification rating for the rock mass. Rating values for the rock masses associated with a number of real rock engineering projects are then determined and correlated with observed rock mass behaviour. In the cases of both Barton’s et al. (1974) Q- and Bieniawski’s (1973) RMR-Systems, a total of 6 rock parameters or features make up the overall rating number of each system.

Although the use of this approach is superficially attractive it has a number of serious shortcomings which, amongst others, extend to:

- a classification scheme not always fully evaluating important aspects of the problem. If blindly applied without any supportive analysis of the mechanics of the problem, it can lead to disastrous results;
- a classification scheme giving reliable results only for those rock masses and circumstances for which they were originally developed;
- an intrinsic constraint whereby any classification scheme which might succeed in “normal” applications and for “average” problems may fail once unusual factors become dominant.

For these reasons and keeping in mind that an underground repository in Opalinus Clay represents a unique prototype structure, it is obvious that an application of a rock classification system is not indicated.

5.3 Scale effect

In TR 2000-02 various evidence was collected that, with regard to the Opalinus Clay at Mont Terri and it’s deformational and strength parameters, the scale effect factor between laboratory and prototype scales is less significant than in other rock mechanics applications and, in fact, close to 1.

Within the review period that evidence has been further consolidated. For H-T-M experiments for example, Zhang et al. (TR 2007-02; p. 179) state that “large-scale consolidation tests provided similar results to those
obtained on the normal samples. This indicates that the scale-effect is negligible for the clay rock within the test range.”

6 Recommendation for the Selection of the Rock Mechanics Parameters

Based on the data sets of Sections 3 and 4, and furthermore on guidelines of Eurocode 7 (EN 1997-1) for a “cautious estimate” of the characteristic values, it is recommended that the rock mechanical parameters as compiled in Tables 6-1 and 6.2 are considered to be representative for the Mont Terri region (ambient conditions with an overburden of 250 m).

<table>
<thead>
<tr>
<th>Index / State Parameter</th>
<th>Section</th>
<th>Symbol Value [Unit]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bulk Density</td>
<td>3.1</td>
<td>$\rho = 2 430 \pm 20$ [kg/m³]</td>
</tr>
<tr>
<td>Bulk Density (dry)</td>
<td>3.2</td>
<td>$\rho_d = 2 330 \pm 50$ [kg/m³]</td>
</tr>
<tr>
<td>Grain Density</td>
<td>3.3</td>
<td>$\rho_s = 2 700 \pm 20$ [kg/m³]</td>
</tr>
<tr>
<td>Water content</td>
<td>3.4</td>
<td>$w = 6.4 \pm 1.0$ [%]</td>
</tr>
<tr>
<td>Porosity</td>
<td>3.5</td>
<td>$n = 13.7 \pm 2.5$ [%]</td>
</tr>
<tr>
<td>P-wave velocity</td>
<td>3.6</td>
<td>$v_p//ss = 3 350 \pm 150$ [m/s]</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$v_p\perp ss = 2 620 \pm 150$ [m/s]</td>
</tr>
<tr>
<td>S-wave velocity</td>
<td>3.6</td>
<td>$v_s//ss = 1 920 \pm 100$ [m/s]</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$v_s\perp ss = 1 510 \pm 50$ [m/s]</td>
</tr>
<tr>
<td>Dynamic Young’s modulus</td>
<td>3.6</td>
<td>$E_{dyn//ss} = 22$ [GPa]</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$E_{dyn\perp ss} = 14$ [GPa]</td>
</tr>
<tr>
<td>Dynamic Poisson’s ratio</td>
<td>3.6</td>
<td>$\nu_{dyn//ss} = 0.26$ [-]</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$\nu_{dyn\perp ss} = 0.25$ [-]</td>
</tr>
<tr>
<td>Atterberg Limits</td>
<td>3.7</td>
<td>$w_i = 38 \pm 5$ [%]</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$w_p = 23 \pm 2$ [%]</td>
</tr>
<tr>
<td></td>
<td></td>
<td>P.I. = 15 \pm 3 [%]</td>
</tr>
<tr>
<td>Carbonate content</td>
<td>3.8</td>
<td>$C_{RCO3} = 11 \pm 4$ [%]</td>
</tr>
<tr>
<td>CaSO₄ content</td>
<td>3.9</td>
<td>$C_{CaSO_4} = 0.26 \pm 0.05$ [%]</td>
</tr>
<tr>
<td>Fracture toughness</td>
<td>3.10</td>
<td>$K_{IC//ss} = 0.53 \pm 0.09$ [MN/m¹.⁵]</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$K_{IC\perp ss} = 0.12 \pm 0.03$ [MN/m¹.⁵]</td>
</tr>
<tr>
<td>Bridgman pinch-off</td>
<td>3.11</td>
<td>$p_{mco//ss} = 13 \pm 9$ [MPa]</td>
</tr>
</tbody>
</table>

Table 6-1. Recommended index and state parameters of the Mont Terri region
<table>
<thead>
<tr>
<th>Design Parameter</th>
<th>Section</th>
<th>Recommended value</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Deformation parameters of isotropic rock model</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tangent modulus</td>
<td>4.1.1</td>
<td>$E_{t-50} = 3$ [GPa]</td>
<td></td>
</tr>
<tr>
<td>Unloading, reloading and Young's modulus</td>
<td></td>
<td>$E_u = E_f = E = 7$ [GPa]</td>
<td></td>
</tr>
<tr>
<td>Poisson's ratio</td>
<td></td>
<td>$\nu = 0.29$ [-]</td>
<td></td>
</tr>
<tr>
<td><strong>Deformation parameters of transverse isotropic rock model</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tangent modulus</td>
<td>4.1.1</td>
<td>$E_{t-50\perp} = 3$ [GPa]</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>$E_{t-50\parallel} = 4$ [GPa]</td>
<td></td>
</tr>
<tr>
<td>Unloading, reloading and Young's modulus</td>
<td></td>
<td>$E_{\perp} = 4$ [GPa]</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>$E_{\parallel} = 10$ [GPa]</td>
<td></td>
</tr>
<tr>
<td>Poisson's ratio</td>
<td></td>
<td>$\nu_{23} = 0.35$ [-]</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>$\nu_{12} = \nu_{13} = 0.25$ [-]</td>
<td></td>
</tr>
<tr>
<td>Shear modulus</td>
<td></td>
<td>$G_{12} = G_{13} = 3.5$ [GPa]</td>
<td></td>
</tr>
<tr>
<td>Bedding plane stiffness</td>
<td>4.1.2</td>
<td>$k_n = 8$ [GPa/m]</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>$k_s = 10$ [GPa/m]</td>
<td></td>
</tr>
<tr>
<td>Steady-state creep</td>
<td>4.1.3</td>
<td>$\Delta \varepsilon / \Delta t = 500 \cdot 10^{-12}$ [s^{-1}]</td>
<td></td>
</tr>
<tr>
<td><strong>Strength parameters</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Uniaxial compressive strength</td>
<td>4.2.1</td>
<td>$UCS_{\perp ss} = 15$ [MPa]</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>$UCS_{\parallel ss} = 11$ [MPa]</td>
<td></td>
</tr>
<tr>
<td>Uniaxial tensile strength</td>
<td></td>
<td>$UTS_{\perp ss} = 0.6$ [MPa]</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>$UTS_{\parallel ss} = 1.2$ [MPa]</td>
<td></td>
</tr>
<tr>
<td>Mohr Coulomb shear strength of material</td>
<td>4.2.2</td>
<td>$c'_{\perp ss} = 4.0$ [MPa]</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>$c'_{\parallel ss} = 3.0$ [MPa]</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>$\phi'_{\perp ss} = 23^\circ$</td>
<td>$\phi'_{\parallel ss} = 22^\circ$</td>
</tr>
<tr>
<td>Shear strength of bedding planes</td>
<td></td>
<td>$c'_{bedding} = 0.8$ [MPa]</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>$\phi'_{bedding} = 21^\circ$</td>
<td></td>
</tr>
<tr>
<td>Hoek &amp; Brown material strength parameters</td>
<td>4.2.3</td>
<td>$m_0 = 5 - 10$</td>
<td>$m_\perp = 1 - 5$</td>
</tr>
</tbody>
</table>
• Strength parameters (continued)

\[
\begin{align*}
\delta_{\perp ss} &= -2 \times 10^{-6} \\
\delta_{\parallel ss} &= -4 \times 10^{-6}
\end{align*}
\]

Dilatation of material

Dilatation angle

\[i = 0^\circ \pm 2^\circ \quad (4)\]

• Permeability parameters

\[
\begin{align*}
K_{\text{isotropic}} &= 2 \times 10^{-13} \text{ [m/s]} \\
K_{\perp ss} &= 0.7 \times 10^{-13} \text{ [m/s]} \\
K_{\parallel} &= 2 \times 10^{-13} \text{ [m/s]}
\end{align*}
\]

Hydraulic conductivity

• Hydro-mechanically coupled parameters

\[
\begin{align*}
S_{\varepsilon\perp ss} &= 7 \% \\
S_{\varepsilon\parallel ss} &= 1 \%
\end{align*}
\]

Swelling strain index

\[
\begin{align*}
\rho_{s\perp ss} &= 1.2 \text{ [MPa]} \\
\rho_{s\parallel ss} &= 0.6 \text{ [MPa]}
\end{align*}
\]

Swelling pressure

• Thermal parameters

\[
\begin{align*}
\alpha_{\parallel} &= 1.5 \times 10^{-5} \text{ [K}^{-1}] \\
\alpha_{\perp} &= 2 \times 10^{-5} \text{ [K}^{-1}]
\end{align*}
\]

Linear thermal expansion coefficient \(\alpha\)

\[
\begin{align*}
\alpha_{\parallel} &= 1.0 \times 10^{-5} \text{ [K}^{-1}] \\
\alpha_{\perp} &= 2.5 \times 10^{-5} \text{ [K}^{-1}]
\end{align*}
\]

Specific heat capacity \(c_p\)

\[
\begin{align*}
\lambda_{\parallel} &= 2.2 \text{ [W m}^{-1} \text{ K}^{-1}] \\
\lambda_{\perp} &= 1.2 \text{ [W m}^{-1} \text{ K}^{-1}]\end{align*}
\]

Thermal conductivity \(\lambda\)

\[
\begin{align*}
(1) \text{ Evaluated as the mean of all relevant laboratory test values, irrespectively of the influence of the confinement (}\sigma_3). \\
(2) \text{ Refer also to Comment C 4.14 on p. 26} \\
(3) \text{ For bi-linear parameters, refer to Tab. 4-11.} \\
(4) \text{ Refer to Tab. 4-14.} \\
(5) \text{ Preliminary values only.}
\end{align*}
\]

Table 6-2. Recommended design parameters of the Mont Terri region (under ambient conditions with an overburden depth of 250 m).

Remarks:
List of References


Acknowledgement

The input provided by Mr. Christophe Nussbaum (Mont Terri Project Consortium) and Dr. Tim Vietor (NAGRA) in the review work is highly appreciated. Detailed comments of an anonymous reviewer assisted in improving the draft report of 15th December, 2008.

Respectfully submitted, 30th June, 2009

Q+S CONSULT, Bad Bentheim, Germany

Prof. Dr.-Ing. Helmut Bock
Publicly accredited and sworn-in Expert
in Geomechanics and Geomonitoring Systems
Appendix A: Source Documents

In co-operation with Dr. Tim Vietor, the following documents were selected for the review and provided the source documents for the Technical Report at hand.

A.1 Mont Terri Project Technical Reports (TRs)
A.1.1 TR 2003-04 Schnier & Stührenberg (BGR) Lab tests, Phases 8 and 9
A.1.2 TR 2006-01 Wileveau & Rothfuchs HE-D: Thermal effects (Synthesis)
A.1.3 TR 2007-02 Zhang et al. (GRS) HE-D: Thermal effects
A.1.4 TR 2007-04 Popp & Salzer (IfG) Lab tests on bedding planes
A.1.5 TR 2007-05 Gräsle & Plischke (BGR) Strength & deformation lab tests

A.2 Mont Terri Project Technical Notes (TNs)
A.2.1 TN 2000-08 Enachescu et al. (Golder) In situ: long-term hydraulic test
A.2.2 TN 2000-10 Enachescu et al. (Golder) In-situ: hydraulic fracturing
A.2.3 TN 2000-11 Lagler & Siegel (BLM) In-situ: Optical borehole logging
A.2.4 TN 2000-12 Fierz & Wälchli (SolExperts) In-situ: Deformation monitoring
A.2.5 TN 2000-47 Gens (Univ. Catalonia) Lab: water retention tests
A.2.6 TN 2000-55 Fierz (SolExperts) In-situ: Deformation monitoring
A.2.7 TN 2001-01 Kamp & Konietzky (ITASCA) Conceptual modelling of stiff clay
A.2.8 TN 2001-19 Konietzky (ITASCA) Modelling of HM coupling in EDZ
A.2.9 TN 2001-28 Laue & Springman (ETHZ) Soil mechanics constitutive models
A.2.10 TN 2002-46 Wolter (DCM) Lab tests on relaxation & strain
A.2.11 TN 2002-47 Pei et al. (MIT) Disturbance in samples
A.2.12 TN 2002-50 Schnier (BGR) Lab strength tests (Phases 6 & 7)
A.2.13 TN 2003-03 Horseman et al. (BGS) Lab test: consolidation & rebound
A.2.14 TN 2003-17 Bühler (SolExperts) In-situ: long-term dilatometer
A.2.15 TN 2003-45 Pei et al. (MIT) Disturbance in samples
A.2.16 TN 2004-37 Wolter (DCM) Lab tests on relaxation & strain
A.2.17 TN 2004-38 Rummel & Weber (MeSy) Lab tests, Phase 9
A.2.18 TN 2004-56 Alheid et al. (BGR) EB-Experiment: hydrogeol. EDZ
A.2.19 TN 2004-86 Lahaye (INERIS) In-situ: overcoring CSRIO cell
A.2.20 TN 2005-21 Konietzky & Kamp (ITASCA) Gallery 04 – numerical modelling
A.2.21 TN 2005-25 Bock (Q+S) Lab tests – review & interpretation
A.2.22 TN 2005-29 Gómez & Fernández (UPV) Ventilation experiment
A.2.23 TN 2005-34 Popp & Salzer (IfG) THM behaviour; bedding planes
A.2.24 TN 2005-57 Rummel & Weber (MeSy) Lab tests, Phase 10
A.2.25 TN 2006-33 Shin (CRIEPI) AS-Experiment: In-situ test
A.2.26 TN 2006-37 Auvray (Laego) Thermo-mechanical tests
A.2.27 TN 2006-50 Konietzky & Kamp (ITASCA) EZ-A - numerical modelling
A.2.28 TN 2006-72 Jobmann et al. (DBE) Thermal expansion effects
A.2.29 TN 2007-25 Badertscher et al. (GeolInst.) Self-sealing in drillcore
A.2.30 TN 2007-30 Jahns (Gesteinslabor) Lab deformability & strength tests

The topics of TN 2005-42 and TN 2007-08 also seemed to be of particular interest for the review, however their publication was still pending at the point in time when this review was carried out.
A.3  Open Literature and other reports referring to Mont Terri Project

<table>
<thead>
<tr>
<th>A.3.1</th>
<th>Wenk et al. (Univ California)</th>
<th>The Leading Edge (June 2008)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A.3.2</td>
<td>Gens et al. (Univ. Catalonia)</td>
<td>Géotechnique, Vol. 57 (2007)</td>
</tr>
<tr>
<td>A.3.3</td>
<td>Marschall et al. (NAGRA)</td>
<td>ARMA 08-193</td>
</tr>
<tr>
<td>A.3.4</td>
<td>Blümling et al. (NAGRA)</td>
<td>Phys. Chem. of Earth, 32 (2007)</td>
</tr>
<tr>
<td>A.3.5</td>
<td>Corkum &amp; Martin (ITASCA; Univ. Alberta)</td>
<td>Int. J. Rock Mech. etc. 44 (2007)</td>
</tr>
<tr>
<td>A.3.6</td>
<td>Naumann &amp; Plischke (BGR)</td>
<td>Anisotropy T-M properties</td>
</tr>
<tr>
<td>A.3.7</td>
<td>Lux et al. (Clausthal Univ. of Technology)</td>
<td>NF-PRO, Deliv. 4.4.13 (2007)</td>
</tr>
</tbody>
</table>
Appendix B: Plots from relevant laboratory and field tests and numerical modelling assumptions

Fig. B-1: Dependency of $v_p$ on hydrostatic pressure $p$ (after Popp and Salzer, 2007). Considered are velocities $// \text{ and } \perp$ to bedding.
For $p = 0$: Data taken from TR 2000-02.
Shaded: Confidence interval of measurements
Dotted line: Hypothetical pressure effect on $v_p$ measured $\perp$ to bedding.
Fig. B-2. Contractor’s performance as monitored by means of the excavation rate \([\text{in } \text{m}^3/\text{h}]\) of Mont Terri Niche 2 of Gallery 08 by a road header. Note that the performance in the sandy facies is significantly lower than in the shaly facies (out of Vietor, 2008).

Fig. B-3. Series of axial stress-axial strain curves at different confinements. Note the absence of a distinct non-linearity at low stress levels and the significant influence of bedding (“foliation”) (out of Popp and Salzer, TR 2007-04; ref. also to Fig. B-8).
Fig. B-4. Young’s Modulus of P-samples as a function of mean stress ($\sigma_{\text{mean}}$) and temperature (out of Naumann and Plischke, 2005)

Fig. B-5. Dependency between elastic parameters $E$, $K$ and $G$ and mean stress $[1/3 (\sigma_1 + \sigma_2 + \sigma_3)]$ as assumed by Zhang et al. (TR 2007-02)
Fig. B-6. Set of shear displacement laboratory experiments of Opalinus Clay samples at various levels of normal stress $\sigma_n$ (out of Popp and Salzer, TR 2007-04)

Fig. B-7. Shear strength (friction) properties of Opalinus Clay bedding planes (out of Popp and Salzer, TR 2007-04)
Fig. B-8. Summary of triaxial tests carried out by Popp and Salzer (TR 2007-04)
Top: Stress deviator – axial strain curves (as in Fig. B-3)
Middle: Dilatancy (volume change) – axial strain curves
Bottom: Permeability – axial strain curves
Fig. B-9. Hydraulic conductivity over the depth from the tunnel sidewall as deduced from in-situ constant head injection tests. Note the tendency for lower K-values with time ("self-sealing") (from Alheid et al. TN 2004-56).

Fig. B-10 Thermal expansion and shrinkage experiment of an Opalinus Clay sample. Note that the $\Delta L/L (=\alpha)$-curve is ~ linearly proportional with $\Delta T$ only within the range of $20^\circ\text{C} < T < -40^\circ\text{C}$ (from Jobmann et al. TN 2006-72).
Fig. B-11  Saturation (top) and de-saturation (bottom) curves of an unconfined Opalinus Clay sample (out of Wolter, TN 2002-46). Note the extremely different strains measured in the three principal sample directions $e_v$, $e_1$, and $e_2$. 
Fig. B-12  Thermal conductivity $\lambda$ of Opalinus Clay core samples (BHE D-26) in dependency of the water content $w$ (from Jobmann et al. TN 2006-72)

Fig. B-13  Triaxial strength ($\sigma_3 = 10$ MPa) of clay rocks in dependency of the water content $w$ (from Zhang et al. TR 2007-02).