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Mont Terri Project

TECHNICAL REPORT 2000-02

July 2001

RA Experiment
Rock Mechanics Analyses and Synthesis: Data Report on Rock Mechanics

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Executive Summary

This Report presents rock mechanical parameters of the Opalinus Clay of the Mont Terri Underground Rock Laboratory in form of a data base. The data were extracted from project-specific documents comprising 49 Technical Notes and from documents of the public domain.

The project-specific documents were critically reviewed on their technical and methodological consistency (Bock, 2000) prior to any acceptance in the data base.

The documents include notes and reports on laboratory and in-situ tests as well as on numerical modelling studies and prototype excavations carried out in the period of 1996 to 2000.

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1. Introduction

On 22th May, 2000 under its Project Number 132.0015.RA3_PH5, the Mont Terri Project, Berne / Switzerland issued an order to *Prof. Dr.-Ing. H. Bock* of *Q+S CONSULT,* Bad Bentheim / Germany for a data report on rock mechanical parameters of the Opalinus Clay of the Mont Terri Underground Rock Laboratory.

The order relates to the ***RA***-Experiment '**R**ock Mechanical **A**nalysis' of the Opalinus Clay at Mont Terri, Phase 5, which was established in July 1999. Amongst others, the objective of this experiment is to analyse and predict the mechanical behaviour of the underground structures at Mont Terri in a step-by-step procedure as follows:

- Critical review of the existing documents on laboratory and in-situ tests as well as on EDZ-prototype experiments.
  This review was carried out by the author and is documented in Technical Note 2000-28 (Bock, 2000).

- Establishment of a comprehensive data base of the rock mechanical parameters of the Opalinus Clay at Mont Terri.
  This is the scope of the Technical Report on hand.

- Establishment of a conceptual model and constitutive law for the Opalinus Clay

- Analysis of the existing and future underground structures and facilities on their mechanical behaviour, e.g. by means of numerical modelling studies and/or prototype experiments.

The establishment of a comprehensive rock mechanical data base is a major prerequisite in achieving the overall project objective.

Partners of the RA-Experiment in Phase 5 are Andra (France), BGR (Germany), IPSN (France), NAGRA (Switzerland) and Obayashi (Japan).

The rock mechanical data base to be established was extracted from documents which are termed *"source documents"*. The most important source documents were project-specific documents comprising of 49

Technical Notes (TN) as listed in Appendix A. In contrast to previous work with similar objectives (e.g. TN 98-49 and Technical Report 99-02) all source documents were critically reviewed on their technical and methodological consistency prior to any acceptance in the rock mechanical data base. The source documents include laboratory and in-situ tests as well as numerical modelling studies and monitoring of prototype excavations carried out between 1996 and 2000.

After identification of the source documents which are relevant for this Technical Report (Section 2), a comprehensive data base of the rock mechanical parameters of the Opalinus Clay is developed. In line with ISRM (1981) it is distinguished between "index" or state parameters (Section 3) and "design parameters" (Section 4). The report concludes with some general remarks on the deformation mechanisms of the Opalinus Clay (Section 5) and with recommendations for the selection of the rock mechanical parameters (Section 6).

2. Source Documents

The source documents of the rock mechanical parameters of the Opalinus Clay at Mont Terri relate to laboratory tests, in-situ (field) tests as well as monitoring and observations in association with the EDZ experiment around the New Gallery.

2.1 Laboratory Tests

A total of 15 source documents, as listed in Table 2-1, covers laboratory tests (for details ref. to Appendix A). There is a large variety of types of laboratory tests as outlined in Table 2-2.

For results of the laboratory tests refer to Sections 3 and 4.

Source Document		Author(s)	General characteristics
A.2.5	TN 97-13	Homand, F. et al.	IS-A: Results of laboratory tests
A.3.4	TN 98-57	Olalla, C. et al.	Laboratory tests
A.4.1/5.1	TN 96-22	Homand, F. and Pepa, S.	Laboratory tests
A.4.2	TN 98-18	Rummel, F. et al.	Gas-frac self-healing (Phase 4)
A.4.3	TN 99-36	Rummel, F. and Weber, U.	Gas-frac self-healing experiment
A.4.4	TN 98-35	Rummel, F. et al.	Laboratory tests
A.4.5	TN 98-55	Rummel, F. et al.	Laboratory tests
A.4.6	TN 99-35	Rummel, F. et al.	Laboratory tests
A.4.7	TN 98-47	Suzuki, K. and Maruyama, M.	DM: Laboratory tests
A.5.2	TN 97-06	Vögtli, B. and Bossart, P.	Swelling experiment
A.5.3	TN 97-06rev.	Vögtli, B. and Bossart, P.	Swelling experiment
A.5.4	TN 98-36	Chiffoleau, S. and Robinet, J.C.	Hydro-mechanical characteristics
A.5.7	TN 99-78	N. N. (ANTEA)	Creep tests
A.5.8	TN 97-26	Horseman, St.	Osmotic pressure experiment
A.5.9	TN 98-15	Ortiz, L.	Laboratory tests (GP experiment)

Table 2-1. Source documents of laboratory tests

Source Document		Test for State / Index Parameters											Test for Design Parameters									
		1	2	3	4	5	6	7	8	9	10	11	21	22	23	24	25	26	27	28	29	30
A.2.5	TN 97-13	x	x	x		x	x						x	x		x		x				
A.3.4	TN 98-57	x	x	x	x		x	x	x	x			x			x		x				
A.4.1/5.1	TN 96-22	x	x	x	x	x			x				x	x		x		x	x			x
A.4.2	TN 98-18	x			x		x				x	x	x	x			x					
A.4.3	TN 99-36	x					x							x				x				
A.4.4	TN 98-35	x					x						x	x				x	x			
A.4.5	TN 98-55	x			x		x						x	x				x	x	x		
A.4.6	TN 99-35	x			x		x						x	x				x	x	x		
A.4.7	TN 98-47	x			x		x						x	x				x	x			
A.5.2	TN 97-06				x				x													x
A.5.3	TN 97-06rev.				x				x													x
A.5.4	TN 98-36	x			x								x									
A.5.7	TN 99-78	x			x		x		x						x							
A.5.8	TN 97-26																				x	
A.5.9	TN 98-15																				x	

Legend of type of test (indicated by " x ") for the determination of ...

... State / Index Parameters

1	Bulk density	(ISRM, 1981)
2	Bulk density (dry)	(ISRM, 1981)
3	Grain density	(ISRM, 1981)
4	Water content	(ISRM, 1981)
5	Porosity n	(ISRM, 1981)
6	Ultrasonic velocity v_p	(ISRM, 1981)
7	Atterberg limits	(DIN 18 122; Part 1)
8	Carbonate content	(DIN 19 684; Part 5)
9	Sulphate content	-
10	Fracture toughness	(ISRM, 1988)
11	Bridgman pinch-off	(Jaeger & Cook, 1963)

... Geotechnical Design Parameters

21 Deformation moduli E
22 Poisson's ratio ν
23 Viscoelastic creep parameter
24 Unconfined compression strength UCS
25 Hydraulic tensile strength HTS
26 Mohr-Coulomb shear parameters c ϕ
27 Dilatation δ
28 Dilatation angle i
29 Permeability k and hydr. conductivity K
30 Swelling strain index S_ε and swelling pressure p_s

Table 2-2. Type of laboratory tests carried out

2.2 In-situ Tests

A total of three (3) source documents, as listed in Table 2-3, relates to in-situ tests (for details ref. to Appendix A).

For results of the field tests refer to Section 4.

Source Documents		Author(s)	Type of experiment / test
A. 2.8	TN 97-16	Bühler	Borehole dilatometer
A. 5.5	TN 97-41	Bühler	Long-term dilatometer
A. 5.6	TN 99-03	Bühler	Long-term dilatometer

Table 2-3. Source documents on in-situ tests at Mont Terri

2.3 EDZ Experiment around New Gallery

Table 2-4 lists the key source documents of the EDZ-experiment around the New Gallery. The documents include monitoring of both rock and lining in response to the excavation of the gallery and numerical analysis prior to and after the excavation.

For results of the field tests refer to Sections 4 and 5.

Source Documents		Author(s)	Type of experiment / test
A. 3.1	TN 97-31	Velasco and Pedraza	Numerical modelling ahead of excavation
A. 3.6	TN 98-58	Fierz	Monitoring of rock displacements
A. 3.7	TN 99-25	Fierz	Monitoring of rock displacements
A. 3.8	TN 99-68	Fierz	Monitoring of rock displacements
A. 3.9	TN 98-52	Fierz	Monitoring of tunnel lining stresses
A. 3.10	TN 98-12	Mathier et al.	Monitoring of rock displacements
A. 3.12	TN 99-37a	Mathier et al.	Monitoring of tunnel convergence
A. 3.14	TN 98-09	Bigarré	Monitoring of rock stresses
A. 3.15	TN 98-30	Velasco and Ruiz	Numerical backanalysis
A. 6.4	TN 98-10	Forney et al.	Tomography of the EDZ rock
A. 7.3	TN 99-62	Schuster et al.	Monitoring of the ultrasonic velocity in rock
A. 8.3	TN 99-53	te Kamp and Konietzky	Numerical modelling of the EDZ
A. 8.4	TN 99-69	Konietzky	Numerical modelling of Opalinus clay
A. 9.1	TN 99-34	Velasco and Pedraza	Numerical modelling ahead of RB-experm.

Table 2-4. Key source documents of the EDZ-Experiment

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.

For results refer to Tables 3-1 to 3-11.

3.1 Bulk Density ρ (in natural conditions)

Definition: $\rho = M / V = (M_s + M_W) / V$ .... (1)

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

Reference: ISRM (1981; p. 81-89)

Parameter	Number of samples n	Bulk density mean ± standard deviation	References
Bulk Density	239	$\rho = 2\,450 \pm 30$ [kg/m³]	ref. to Table 2-2

Table 3-1. Bulk Density of the Opalinus Clay at Mont Terri

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

3.2 Bulk Density (dry) ρ_d

Definition: $\rho_d = M_s / V$ .... (2)

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

Reference: ISRM (1981; p. 81-89)

Parameter	Number of samples n	Dry bulk density mean ± standard deviation	References
Bulk Density (dry)	27	$\rho_d = 2\ 340 \pm 60$ [kg/m³]	ref. to Table 2-2

Table 3-2. Bulk Density (dry) of the Opalinus Clay at Mont Terri

Note: Distinguishing of the dry bulk density for the various geological facies units (e.g. sandy, carbonate-rich, shaly facies) does not yield any significant differences.

3.3 Grain Density ρ_s

Definition: $\rho_s = M_s / V_s$ .... (3)

with: M_s = mass of solids
V_s = volume of solids

Reference: ISRM (1981; p. 81-89)

Parameter	Number of samples n	value mean ± standard deviation	References
Grain Density	42	$\rho_s = 2\ 710 \pm 30$ [kg/m³]	ref. to Table 2-2

Table 3-3. Grain Density of the Opalinus Clay at Mont Terri

Note: Distinguishing of the grain density for the various geological facies units (e.g. sandy, carbonate-rich, shaly facies) does not yield any significant differences.

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 and not to adsorbed or structural water (Horseman et al., 1996). Investigations carried out by Chiffoleau and Robinet (in TN 98-36) indicate that, for the Opalinus Clay, the quantity of "free water" (~ 40%) is less than that of the adsorbed water (~ 60%).

Definition: $w = (M_w / M_s) \cdot 100$ [%] .... (4)

with: M_w = mass of "free" water
M_s = mass of solids (including adsorbed water)

Reference: ISRM (1981; p. 81-83)

Parameter	Number of samples n	Natural water content mean ± standard deviation	References
Water content	116	$w = 6.1 \pm 1.9$ [%]	ref. to Table 2-2

Table 3-4. Water content of the Opalinus Clay at Mont Terri in natural conditions

Note: All test results, considered in this Report, are based on the oven drying method with temperatures between 105 and 115°C over a duration of 1 to 7 days.

3.5 Porosity n

Definition: $n = (V_v / V) \cdot 100$ [%] .... (5 a)
$n = (1 - \rho_d / \rho_s) \cdot 100$ [%] .... (5 b)

with: V_v = volume of voids
V = bulk sample volume
ρ_d = dry density
ρ_s = grain density

Reference: ISRM (1981; p. 81-89)

Parameter	Number of samples n	Porosity mean ± standard deviation	References
Porosity	17	$n = 12.1 \pm 1.4$ [%]	ref. to Table 2-2

Table 3-5. Porosity of the Opalinus Clay at Mont Terri as determined by mercury injection and helium pycnometry

Note: The determination of the porosity was carried out by mercury injection technique (TN 96-22) and by helium pycnometry (TN 97-13). As mentioned by Bath (2001) these techniques tend to underestimate the porosity, indicating that a proportion of voids have sizes smaller than that required for mercury entry.

This tendency is confirmed when substituting the values ρ_d = 2340 ± 60 [kg/m³] and ρ_s = 2710 ± 30 [kg/m³], as determined in Sections 3.2 and 3.3, into Equation (5 b). It is

$$n = 13.7 \pm 3.1 \, [\%]$$

Against this background and because of the much wider data base which is represented by the substituted values, $n = 13.7 \pm 3.1$ [%] is considered to be the relevant porosity value for the Opalinus Clay.

3.6 Ultrasonic Velocity v_p and v_s and Dynamic Elastic Constants E_{Dyn} and ν_{dyn}

Definition:

$$E_{dyn} = 2 * v_s^2 * \rho \, (1 + \nu_{dyn}) \quad \text{............ (6 a)}$$

$$\nu_{dyn} = 0.5 * [2 - (v_p / v_s)^2] \, / \, [1 - (v_p / v_s)^2] \quad \text{............ (6 b)}$$

with:
- E_{dyn} = dynamic Young's Modulus
- ν_{dyn} = dynamic Poisson's ratio
- ρ = density
- v_p = compressional wave velocity
- v_s = shear wave velocity
- $//$ ss = index for P-samples (in direction of bedding)
- $\perp$ ss = index for S-samples (normal to bedding)

Reference: ISRM (1981; p. 105-110)

Parameter	Number n of samples	Value mean ± standard deviation	Reference Remarks
$v_{p\ //\ ss}$	111	3 410 ± 240 [m/s]	ref. to Table 2-2
$v_{s\ //\ ss}$	111	1 960 ± 120 [m/s]	ref. to Table 2-2
$v_p / v_{s\ //\ ss}$	-	*1.74*	
$v_{p\ \perp\ ss}$	48	2 620 ± 400 [m/s]	ref. to Table 2-2
$v_{s\ \perp\ ss}$	33	1 510 ± 250 [m/s]	ref. to Table 2-2
$v_p / v_{s\ \perp\ ss}$	-	*1.73*	
$E_{dyn\ //\ ss}$	111 *111*	23.7 ± 3.2 [GPa] *23.5 [GPa]*	(*) *(**)*
$\nu_{dyn\ //\ ss}$	108 *111*	0.24 ± 0.03 *0.25*	(*) *(**)*
$E_{dyn\ \perp\ ss}$	29 *33*	11.9 ± 1.6 [GPa] *14.0 [GPa]*	(*) *(**)*
$\nu_{dyn\ \perp\ ss}$	29 *33*	0.28 ± 0.02 *0.25*	(*) *(**)*

Table 3-6. Ultrasonic velocity and dynamic elastic parameters

Note: The determination of E_{dyn} and ν_{dyn} was carried out in two ways:

(*) Referring to individual E_{dyn} and ν_{dyn} values as documented in the source documents (ref. to Table 2-2).

(**) By substitution of the relevant mean values of v_p and v_s and of ρ = 2 450 [kg/m³] (Section 3.1) into Equs. (6a) and (6b).

Generally, there is a good agreement between the two data sets. Due to its more direct data base, the values as determined by the first method (*) are preferred.

3.7 Atterberg Limits

Definition: $P.I. = w_l - w_p$ .... (7)

with: P.I. = plasticity index
w_l = water content at liquid limit
w_p = water content at plastic limit

Reference: DIN 18 122 Part 1

Parameter	Number of samples n	Value mean ± standard deviation	Reference
Liquid limit	33	$w_l = 38 \pm 5$ [%]	(*) TN 98-57
Plastic limit		$w_p = 23 \pm 2$ [%]	
Plasticity Index		$P.I. = 15 \pm 3$ [%]	

Table 3-7. Atterberg Limits of the Opalinus Clay at Mont Terri

Note: (*) Atterberg limit tests are carried out on samples which are entirely remoulded. For the Opalinus Clay such tests are of limited value as the test samples lack characteristic features such as the natural fabric and original cementation.

3.8 Carbonate Content C_{RCO_3}

Definition: $C_{RCO_3} = (M_R / M_s) \cdot 100$ [%] .... (8)

with: M_R = mass of Ca-, Mg-, Fe-, Sr-, Ba-Ions
M_s = mass of solids

Reference: DIN 18 129 and DIN 19 684 (Part 5)

Parameter	Number of samples n	Carbonate content mean ± standard deviation	Reference
RCO_3 content	33	$C_{ROO3} = 9.4 \pm 5.9$ [%] (*)	ref. to Table 2-2

Table 3-8. Carbonate content of the Opalinus Clay at Mont Terri

Note: (*) The comparatively very high standard deviation (± 5.9 %) is indicative of an inhomogeneous distribution of the RCO_3 content within the Opalinus Clay.

3.9 Sulphate Content C_{CaSO4}

Definition: $C_{CaSO4} = (M_{CaSO4} / M_s) * 100$ [%] .... (9)

with: M_{CaSO4} = mass of Ca-sulphates (anhydrite and gypsum)
M_s = mass of solids

Parameter	Number of samples n	Sulphate content mean ± standard deviation	Reference
$CaSO_4$ content	5	$C_{CaSO4} = 0.26 \pm 0.05$ [%] (*)	TN 98-57

Table 3-9. Sulphate content of the Opalinus Clay at Mont Terri

Note: (*) This value must be considered a poor representation due to the insufficient number of test samples. There is also evidence of a significant variation of $CaSO_4$ content depending on type of rock facies.

3.10 Fracture Toughness K_{IC}

Definition: $K_{IC} = A_{min} * F_{max} / D^{1.5}$ .... (10)

with: F_{max} = failure load
D = diameter of the specimen
A_{min} = dimensionless factor

$$A_{min} = 3.33\,[1.835 + 7.15\,a_o/D + 9.85\,(a_o/D)^2]$$

with a_o = initial crack length

References: ISRM (1988)
Ouchterlony, F. (1989)

Parameter	Number of samples n	Fracture toughness mean ± standard deviation	References
$K_{IC\ //\ ss}$	8	0.53 ± 0.09 [$MN/m^{1.5}$]	TN 98-18
$K_{IC\ \perp\ ss}$	6	0.12 ± 0.03 [$MN/m^{1.5}$]	

Table 3-10. Fracture toughness K_{IC} of the Opalinus Clay at Mont Terri

3.11 Bridgman Pinch-Off Strength

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

with: p_{mco} = hydraulic tensile strength of the specimen
p_{mc} = confining pressure at tensile failure
σ_1 = axial stress

Reference: Jaeger, J.C. and Cook, N. G. W. (1963).

Parameter	No. of samples	Hydraulic tensile strength mean ± standard deviation	Reference
$p_{mco\ //\ ss}$	4	13 ± 9 [MPa]	TN 98-18

Table 3-11. Hydraulic tensile strength p_{mco} of the Opalinus Clay at Mont Terri

4. Rock Mechanical Design Parameters

Design parameters provide quantitative materials characteristics for calculations. 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 mechanical design parameters of the Opalinus Clay at Mont Terri were determined by laboratory and/or field tests and by numerical backanalyses of prototype experiments.

The pertinent design parameters may be classified as follows:

1. Deformation Parameters (Section 4.1)

• General deformation parameters:	First-loading modulus E_{init} Unloading modulus E_u Reloading modulus E_r
• Linear elastic parameters:	Young's modulus E Poisson's ratio ν
• Long-term deformation (creep) parameter:	Viscosity η

2. Strength Parameters (Section 4.2)

• Uniaxial strength parameters:	compressive: UCS tensile: HTS and UTS
• Mohr-Coulomb failure parameters (for both material and bedding planes):	cohesion c friction angle ϕ dilatation (bulk δ and angle i)

3. Permeability Parameters (Section 4.3)

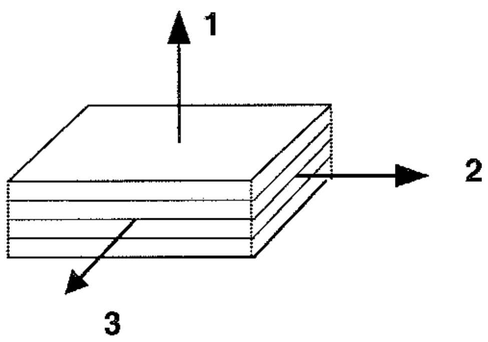
	Intrinsic permeability k Hydraulic conductivity K Coefficient of consolidation C_v

4. Hydro-mechanically coupled parameters (Section 4.4)

- Swelling — Swelling pressure p_s; Swelling strain index S_ε

- Deformability and strength as a function of the water content w

The Opalinus Clay is a distinctively bedded material. Its mechanical behaviour can best be described in a *transverse isotropic model* (Fig. 4-1 top). Accordingly, the test samples are termed with respect to the orientation of the sample axis towards bedding (Fig. 4-1; bottom).



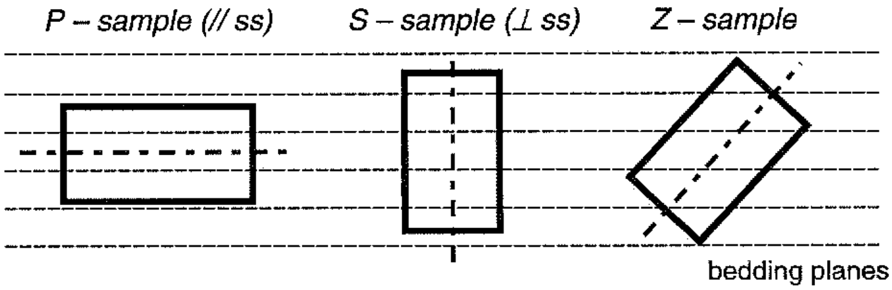


Fig. 4-1. Convention of reference axes for a transverse isotropic rock (top) and designation of test samples (bottom; after TN 98-47)

A more elementary approach uses an *isotropic model.* This model is commonly employed, often as a first approximation, despite the fact that it neglects the influence of the bedding planes on the mechanical characteristics of the rock. Generally, it can be stated that the higher the refinement of a mechanical model the more considerable the accompanying efforts both for the determination of the design parameters and for carrying out of the computations. In any specific application, engineering judgement is required to decide which of the alternative models is most appropriate and sufficient.

4.1 Deformation Parameters

4.1.1 Short-term deformation parameters

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

E	Young's modulus
ν	Poisson's *ratio*

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

E_1	($E \perp_{ss}$)
$E_2 = E_3$	($E_{//ss}$)
$\nu_{12} = \nu_{13}$	($\nu_{//ss}$)
ν_{23}	($\nu \perp_{ss}$)
$G_{12} = G_{13}$	Shear modulus

Definition: Refer to Figs. 4-2 and 4-3.

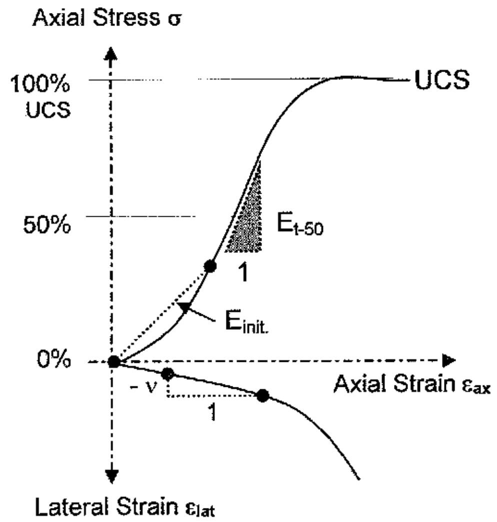


Fig. 4-2. Definition of short-term deformation parameters in a non-cyclic test

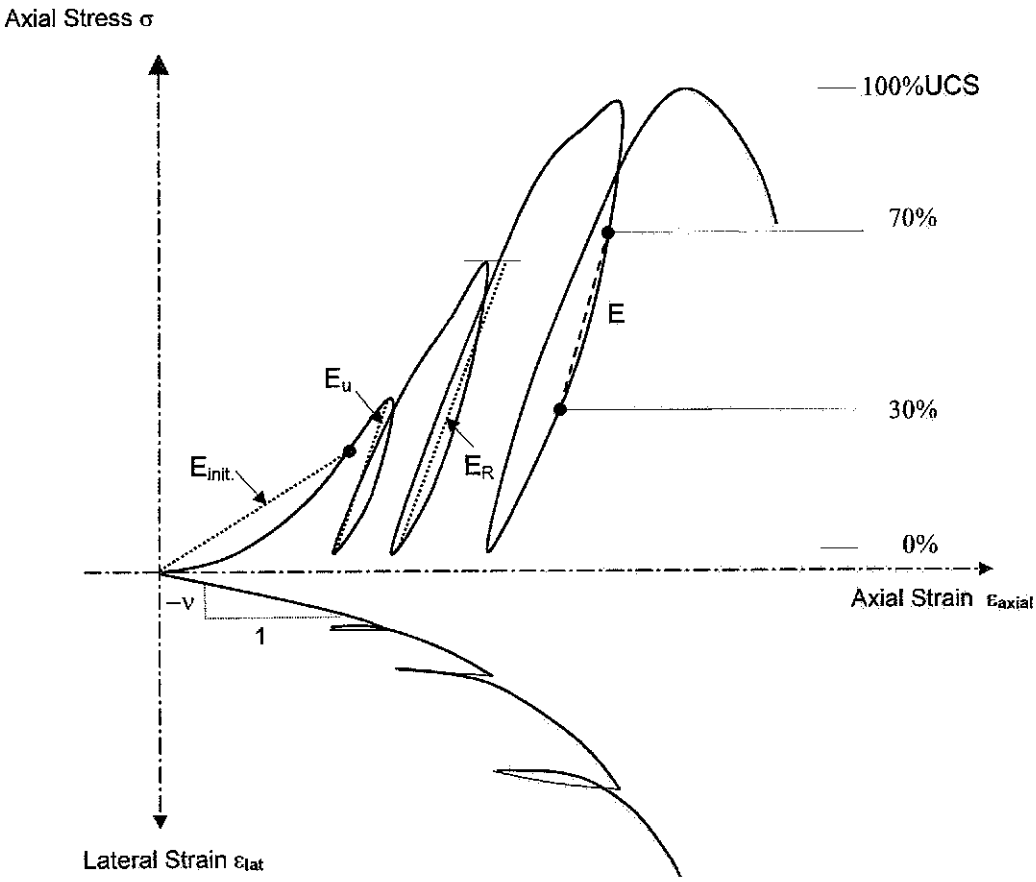


Fig. 4-3. Definition of short-term deformation parameters in a cyclic test

with:

UCS	=	Unconfined Compressive Strength	ISRM (1981, p. 113)
$E_{init.}$	=	First-loading Modulus	DIN 4094-5
E_u	=	Unloading Modulus	DIN 4094-5
E_R	=	Reloading Modulus	DIN 4094-5
E_{t-50}	=	Tangent Modulus at 50% UCS	ISRM (1981, p. 115)
E	=	Young's Modulus	DIN 4094-5
ν	=	Poisson's ratio	ISRM (1981, p. 116)

The first-loading modulus E_{init} is sensitively dependent on a complex mix of conditions and factors such as sample preparation, water content, load set-up, load level and degree of cracking of the sample. It is therefore not always meaningful to specify a general first-loading modulus E_{init}.

With regard to their magnitudes both the unloading and reloading moduli E_u and E_R are very close to the Young's Modulus E proper, particularly in relation to the central sections of the unloading and reloading curves (Fig. 4-3).

Within engineering accuracy it is therefore $E_u \approx E_R \approx E$.

Consequently, no distinction is made within this report between the unloading, reloading and Young's moduli. An example of a laboratory test for the determination of E is given in Fig. B-1 of Appendix B.

- Results of Laboratory Tests

Isotropic elastic parameters: For results refer to Table 4-1.

σ_3 [MPa]	No. of samples	Modulus [GPa]	Poisson's ratio ν	References
0	13	$E = 13.2 \pm 4.5$	0.27 ± 0.08	TN 96-22 TN 97-13 TN 98-18 TN 98-35 TN 98-47 TN 98-55 TN 99-35
2	2	$E = 10.2 \pm 3.6$		
10	2	$E = 11.8 \pm 7.2$		
0	19	$E_{t-50} = 6.4 \pm 3.8$		
2	2	$E_{t-50} = 3.6 \pm 1.4$		
10	35	$E_{t-50} = 4.6 \pm 1.8$		

Table 4-1. Elastic parameters for an isotropic rock model as deduced from short-term laboratory tests

Note:
- 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 4-2).
- Analogous procedures were employed for the determination of the Poisson's ratio ν.

Transversal isotropic elastic parameters: For results refer to Table 4-2.

σ_3 [MPa]	n	Modulus [GPa] mean ± standard deviation	Poisson's ratio ν	$G_{12} = G_{13}$ [GPa]	Reference
0	8	$E_2 = E_3$ = 15.5 ± 4.0	$\nu_{12} = \nu_{13}$ = 0.24 ± 0.08	1.2 ± 0.4 (n = 3)	TN 96-22 TN 97-13 TN 98-18 TN 98-35 TN 98-47 TN 98-55 TN 99-35
	5	E_1 = 9.5 ± 2.4			
2	0	$E_2 = E_3$ -			
	2	E_1 = 10.2 ± 3.6			
10	0	$E_2 = E_3$ -			
	2	E_1 = 11.8 ± 7.2			
0	11	$E_{t\text{-}50\,2} = E_{t\text{-}50\,3}$ = 8.5 ± 3.7	ν_{23} = 0.33 ± 0.05		
	8	$E_{t\text{-}50\,1}$ = 3.6 ± 1.0			
2	0	$E_2 = E_3$ -			
	2	$E_{t\text{-}50\,1}$ = 3.6 ± 1.4			
10	20	$E_{t\text{-}50\,2} = E_{t\text{-}50\,3}$ = 5.9 ± 0.7			
	15	$E_{t\text{-}50\,1}$ = 2.9 ± 1.5			

Table 4-2. Elastic parameters for a transversal isotropic rock model as deduced from short-term laboratory tests

Note: The parameters E and $E_{t\text{-}50}$ derived in Tables 4-1 and 4-2 are not representative for the deformation behaviour of the Opalinus Clay *at very low stress levels*. As indicated in Figs. 4-2 and 4-3 the stress-strain curve near the origin is characterised by a comparatively shallow slope. This phenomenon may be associated with the micro structure of the Opalinus Clay and the effects of porewater pressures. For the representation of the deformational behaviour at very low stress levels (say for $0 \leq \sigma \leq 2$ MPa) a further modulus E_V may therefore be introduced with $E_V << E_{t\text{-}50} < E$. It is difficult to reliably deduce a specific value of E_V from laboratory or field tests due to systematic imperfections in the relevant testing set-ups (refer to earlier remarks in connection with the first-loading modulus $E_{init.}$). E_V may best be determined by numerical backanalysis of the EDZ experiment, especially with regard to the deformations (convergence) of the excavated surface of the New Gallery.

- Results of Field Tests

Definition: $E_d = (1+\nu)\,[\,D / \Delta D\,] * \Delta p_i$ .... (12)

with: E_d = dilatometric modulus
Δp_i = pressure increment
ΔD = average change of drillhole diameter D due to Δp_i
ν = Poisson's ratio

Reference: ISRM (1987); p. 132.

An example of a dilatometer test curve is given in Fig. B-2 of Appendix B.

The first-loading curve is considered to be non-representative as the intrinsic deformability of the tested rock is superimposed by various mechanisms of unknown magnitudes, in particular consolidation effects and squeeze of the disturbed borehole wall. In contrast, the unloading and reloading curves are widely free of such disturbances.

The following unloading moduli were determined (Table 4-3):

n	Unloading Modulus [GPa]		Reference
	$\nu = 0.27$	$\nu = 0.35$	
9	$E_2 = E_3 =$ 6.8 ± 0.9	$E_2 = E_3 =$ 7.2 ± 0.9	TN 97-16 TN 99-03
30	$E_1 =$ 2.4 ± 0.6	$E_1 =$ 2.5 ± 0.6	

Table 4-3. Moduli for transversal isotropic rock as deduced from the unloading curves of short-term field dilatometer tests.

4.1.2 Long-term deformation parameters (creep)

Creep tests were carried out in the laboratory in form of triaxial load tests (TN 99-78) and in the field by use of borehole dilatometers (TN 97-41; TN 99-03).

Assuming a linear visco-elastic material behaviour (Burgers body) it is:

A. For laboratory testing

$$\varepsilon_1(t) = 2\,\sigma_1 / (9\,K) + \sigma_1 / (3\,G_2) + \sigma_1 / (3\,G_1) \cdot [1 - e^{-(G1\,t/\eta 1)}] + \sigma_1 / (3\,\eta_2)\,t$$

with: $\varepsilon_1(t)$ = axial strain of uniaxial test .... (13 a)
σ_1 = deviatoric stress
K = bulk modulus
with $K = E / [3\,(1-2\nu)]$
G_1, G_2 = shear moduli
η_1, η_2 = viscosity parameters

An alternative notation for Equation (13 a) is (ref. to TN 99-78):

$$\varepsilon_1(t) = \varepsilon_e + \varepsilon_f\,[1 - \exp(-t / T_1)] + [\Delta\varepsilon_1 / \Delta t] \cdot t \quad \text{........................ (13 b)}$$

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

B. For borehole dilatometer testing

$$u_r(t) = p \cdot a / (2\,G_2) + p \cdot a / (2 \cdot G_1) \cdot [1 - e^{-(G1\,t/\eta 1)}] + p \cdot a / (2\,\eta_2)\,t$$

with: $u_r(t)$ = widening of the borehole wall .... (14)
p = sleeve pressure of the dilatometer
G_1, G_2, η_1, η_2 as above

References: ISRM (1987)
Goodman (1980)

From laboratory tests of re-saturated samples the following strain parameters of Equation (13 b) were determined (Table 4-4):

n	Type	Visco-elastic strain parameters				Reference
		ε_e	ε_f	T_1 [s]	$\Delta\varepsilon_1 / \Delta t$ [s^{-1}]	
1	// ss	0.035 (*)	0.20	200	0.000 25	TN 99-78
1	$\perp$ ss	0.040 (*)	0.11	90	0.001 5	

Table 4-4. Visco-elastic strain parameters as deduced from laboratory creep tests

Note: (*) - The instantaneous elastic strain evaluated is considered to be non-representative. Reasons: (1) Poorly defined intercept of the ε-axis; (2) Very limited number of tests.

- The derivation of the visco-elastic parameters G_1, G_2, η_1 and η_2 according to Equations (13 a) and (14) from the laboratory and field test curves needs an effort which is beyond the scope of this Technical Report.

4.2 Strength Parameters

4.2.1 Uniaxial Strength Parameters

- Uniaxial Compressive Strength UCS

Definition: $UCS = P_{max} / A_0 = \sigma_{1\,max}$ at $\sigma_3 = \sigma_2 = 0$ .... (15)

with: P_{max} = maximal compressive load on sample
A_0 = initial cross-sectional area
σ_3 = confining pressure

Reference: ISRM (1981); p. 113-116.

Unconfined compressive strength: For results ref. to Table 4-5.

Parameter	Number of samples n	Value mean ± standard deviation	References
UCS $_{//ss}$	22	$UCS_{//ss} = 10.5 \pm 6.5$ [MPa]	TN 96-22 TN 97-13 TN 98-18 TN 98-57
UCS $_{\perp ss}$	4	$UCS_{\perp ss} = 25.6 \pm 2.5$ [MPa]	

Table 4-5. Unconfined Compressive Strength as deduced from short-term laboratory tests

Note: The fact that $UCS_{//ss}$ turned out to be significantly smaller than $UCS_{\perp ss}$ is most likely related to different failure mechanisms. Fig. 4-4 depicts some failure mechanisms which are common in UCS testing. Mechanism 1 seems to be relevant in $UCS_{//ss}$ tests whereas Mechanism 3 prevails in $UCS_{\perp ss}$ tests.

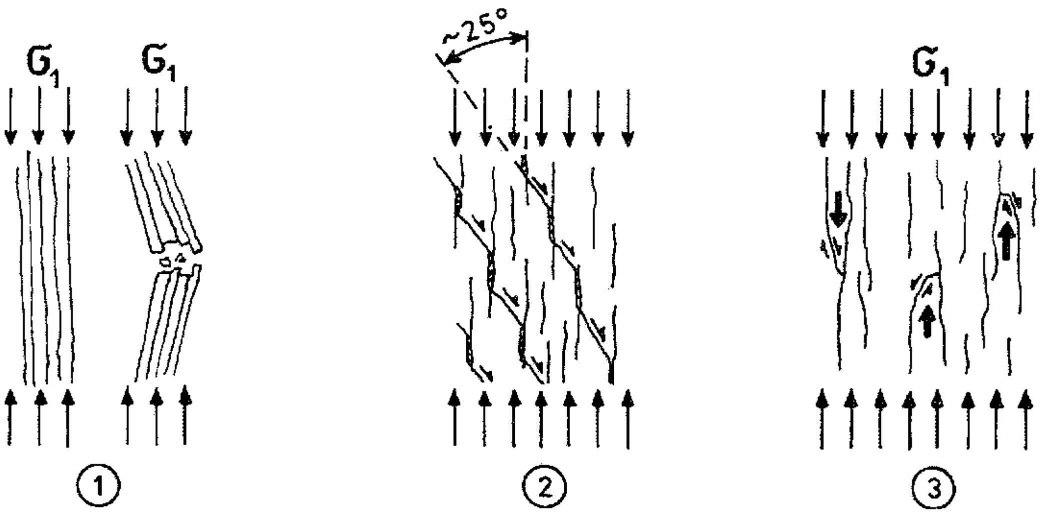


Fig. 4-4. Alternative failure mechanisms in unconfined compression testing (1) Kinking (2) Composite shear failures (3) Local wedge failures (out of Bock, 1980)

- Hydraulic Tensile Strength HTS

Definition: $HTS = p_c - k * p_1$ .... (16 a)

with: p_c = hydraulic fracture initiation pressure
p_1 = confining pressure
k = frac coefficient (k = 2 denotes a material without any pre-existing micro fractures)

From experience (e. g. at Benken / Switzerland) it is known that

HTS >> UTS (Unconfined Tensile Strength)

$UTS = P_{min} / A_0 = \sigma_{3\,min}$ at $\sigma_1 = \sigma_2 = 0$ .... (16 b)

with: P_{min} = Minimal (= max. tensile) load on sample
A_0 = Initial cross-sectional area

Reference: ISRM (1981); p. 117-121
Jaeger and Cook (1963)
Dr. P. Blümling (2001; pers. communication)

Hydraulic and unconfined tensile strength: For results ref. to Table 4-6.

Parameter	Number of samples n	Values [MPa] (*)		Reference
		HTS	UTS (estimated)	
$TS_{//ss\ material}$	14	$HTS_{//ss} = 9$	$UTS_{//ss} \cong 2$	TN 98-18
$TS_{\perp\ bedding}$	14	$HTS_{\perp ss} = 2$	$UTS_{\perp ss} \cong 1$	

Table 4-6. Hydraulic Tensile Strength and estimated Unconfined Tensile Strength

(*) Note: No direct or indirect (Brazilian) tensile tests were carried out at Mont Terri. The HTS was determined in hydraulic fracturing tests on mini cores with 3 mm diameter injection boreholes. By means of empirical relationships (e.g. Jaeger & Cook, 1963) and from experience rough estimates can be given for the UTS.

4.2.2 Mohr-Coulomb Shear Strength Parameters

Definition: $\tau = c' + \sigma_n' \cdot \tan \phi'$ .... (17)

with: τ = shear strength
c' = effective cohesion
ϕ' = effective angle of internal friction [°] (for material)
effective angle of friction [°] (for discontinuities)
σ_n' = effective normal stress

Reference: ISRM (1981); p. 123-127
DIN 18 137; Part 2

- Material strength

For results refer to Table 4-7 and Fig. B-3 in Appendix B.

n	Type of sample	Failure parameter Opalinus Clay material	References
10	P	$c_{//\,ss}'$ = 2.2 MPa $\phi_{//\,ss}'$ = 25°	TN 98-57 TN 96-22 TN 97-13 TN 98-35 TN 98-47 TN 98-55 TN 99-35
34	S	$c_{\perp\,ss}'$ = 5.5 MPa $\phi_{\perp\,ss}'$ = 25°	

Table 4-7. Mohr-Coulomb strength parameters of the Opalinus clay material as deduced from laboratory triaxial tests

Note: The strength of the Opalinus Clay sensitively depends on the moisture content w as is shown e.g. in TN 99-35 (ref. to Fig. B-9). The parameters c' and ϕ', specified in Table 4-7, were derived from those strength data of the source documents for which the moisture content of the rock was specified to be within the limits of the natural moisture content, i. e. within the limits of $4.2 \leq w \leq 8.0$ % (ref. to Table 3-4).

- Bedding plane strength

For results refer to Table 4-8 and Fig. B-4 in Appendix B.

n	Type of sample	Failure parameter of bedding planes	References
22	Z	c_z' = 1 MPa ϕ_z' = 23°	TN 98-57 TN 98-55 TN 99-35

Table 4-8. Mohr-Coulomb strength parameters of bedding planes as deduced from laboratory triaxial and direct shear tests

4.2.3 Dilatation

- Dilatation of the material (volumetric strain δ)

Definition: $\delta = \Delta V / V = \varepsilon_{ax} + 2 \cdot \varepsilon_{lat} = \varepsilon_{ax}(1 - 2\nu)$ .... (18)

with: ΔV = volume increase (+) of an element
V = original volume
ε_{ax} = axial strain
ε_{lat} = lateral strain
ν = Poisson's ratio

- Dilatation of discontinuities (dilatation angle i)

Definition $i = \tan^{-1} (\Delta n / \Delta s)$ [°] .... (19)

with: Δn = normal displacement of discontinuity when subject to shear (+ opening / - closure)
Δs = shear displacement of discontinuity

References: ISRM (1975) and (1981)
Brady & Brown (1985).

For results of the dilatancy δ of material refer to Table 4-9 and Fig. B-5 of Appendix B.

σ_3	n	Type of sample	Dilatancy δ at UCS of the Opalinus clay material	References
0 MPa	1	insufficient data base		TN 96-22 TN 98-35 TN 98-47 TN 98-55 TN 99-35
10 MPa	21	P	$\delta_P = -2100 \pm 1800 \cdot 10^{-6}$	
	16	S	$\delta_S = -4100 \pm 4100 \cdot 10^{-6}$	

Table 4-9. Dilatancy δ of the Opalinus clay material at failure as deduced from laboratory triaxial tests. Note: negative values = volume decrease

For results of the dilation angle i of the bedding planes refer to Table 4-10.

σ_3	n	Type of sample	Dilatance angle i of bedding planes	References
10 MPa	11	Z	$i \approx 0 \pm 2$ [°]	TN 96-35 TN 98-47 TN 98-55 TN 99-35

Table 4-10. Dilatance angle i of the bedding planes at shear as deduced from laboratory triaxial tests

4.3 Permeability Parameters

- Hydraulic conductivity K

Definition $K = Q \cdot \Delta x / (\Delta h \cdot A)$ [m / s] .... (20)

with:

Q	=	flow rate	[m³ / s]
Δx	=	length increment	[m]
Δh	=	hydraulic head increment	[m]
		$\Delta h / \Delta x$ = "hydraulic gradient" i	[-]
A	=	cross-sectional area of the flow path	[m²]

- (Specific or Intrinsic) Permeability k

Definition $k = (Q * \mu * \Delta x) / (\Delta p * A)$ $[m^2]$ .... (21)

with:

Q =	flow rate	$[m^3 / s]$
μ =	dynamic viscosity of the fluid	$[N * s / m^2]$
Δx =	length increment	$[m]$
Δp =	fluid pressure increment	$[N / m^2]$
A =	cross-sectional area of the flow path	$[m^2]$

The parameters K and k are interrelated as follows:

$K = k * \rho * g / \mu$ .... (22)

with:

ρ =	density of fluid	$[kg / m^3]$
g =	gravitational acceleration	$[N / kg]$ or $[m / s^2]$

Reference: DIN 18 130 (1989)

For results of the permeability k refer to Table 4-11.

σ_c [MPa]	n	Type of sample	Permeability Opalinus clay material	References
4	2	Z	$k_Z = 2 * 10^{-20}$ $[m^2]$	TN 97-26 TN 98-15
6	1	S	$k_{\perp ss} = 8 * 10^{-21}$ $[m^2]$	

Table 4-11. Permeability k of the Opalinus clay as deduced from laboratory isostatic cell tests.
σ_c = confinement pressure

Substituting into Equation (22) of the following values

$\rho = 1000$ $[kg / m^3]$ and

$\mu = 1330$ $[N * s / m^2]$ (for 10°C)

yields orders of magnitude of the hydraulic conductivity K as shown in Table 4-12.

σ_c [MPa]	n	Type of sample	Hydraulic conductivity (for 10°C) Opalinus clay material	References
4	2	Z	$K_Z = 2 \cdot 10^{-13}$ [m/s]	TN 97-26 TN 98-15
6	1	S	$K_{\perp ss} = 6 \cdot 10^{-14}$ [m/s]	

Table 4-12. Orders of magnitude of the hydraulic conductivity K of the Opalinus clay as deduced from laboratory isostatic cell tests.

Note:

- The permeability parameters presented in Tables 4-11 and 4-12 are based on laboratory test samples only.
- A recent compilation of the hydraulic parameters of the Opalinus Clay at Mont Terri (TN 2000-22B), which is based on both laboratory and field tests, resulted in the following ranges of permeability k and hydraulic conductivity K:

$$10^{-21} \leq k \leq 0.5 \cdot 10^{-19} \quad [m^2]$$
$$10^{-14} \leq K \leq 0.5 \cdot 10^{-12} \quad [m/s]$$

- A systematic anisotropy of the hydraulic parameters could not be confirmed in TN 2000-22B due to the lack of a reliable data base.

• Coefficient of primary consolidation (one-dimensional)

Definition $\partial^2 u / \partial z^2 = (1 / C_v) \cdot \partial u / \partial t$ .... (23)

with: C_v = coefficient of primary consolidation
u = excess porewater pressure
z = reference direction of compaction movement

C_v is dependent on the permeability k of the ground, the void ratio e and the thickness and structure of the stratum which is subject to consolidation.
The laboratory experiments (e. g. TN 98-36; TN 99-78) were inconclusive with regard to C_v. However, they gave clear indications that C_v is a parameter which is relevant for the Opalinus Clay. This

is also supported by a long-term dilatometer field test (TN 99-03). The test yielded time-versus-deformation curves which are characteristic for a consolidation-controlled behaviour the tested ground (Fig. B-6 in Appendix B). For the evaluation of the relevant C_v a separate analysis considering the particular boundary conditions of the dilatometer test has to be carried out.

4.4 Hydro-mechanically coupled parameters

4.4.1 Swelling Parameters

- Swelling Strain Index S_ε

Definition $S_\varepsilon = (d / L) \cdot 100 \quad [\%]$ .... (24)

at a surcharge pressure σ_n = constant

with: d = maximum swelling displacement
L = initial thickness of the specimen before swelling
$\sigma_n = p_0 + p_s$
with p_0 = pre-loading pressure
p_s = swelling pressure

Reference: ISRM (1981), p. 90-91
Paul (1986)

S_ε is defined for a specific (= constant) surcharge pressure σ_n and for zero lateral strain (uniaxial strain) conditions.

For results of the swelling strain index S_ε refer to Table 4-13 and Fig. B-7.

p_0 [MPa]	p_s [MPa]	n	Sample direction	Swelling strain index (*) Opalinus clay material	References
0.05	~ 0.45	5	$\perp$ ss	$S_{\varepsilon \perp ss} = 7 \pm 2$ [%]	TN 96-22 TN 97-06
0.05	~ 0.12	4	// ss	$S_{\varepsilon // ss} = \sim 1$ [%]	

Table 4-13. Swelling strain index S_ε of the silty-shaly facies of the Opalinus clay at a specific swelling pressure p_s as deduced from laboratory tests.

Note: (*) Estimated value only as there are an insufficient data base and incomplete specifications of the swelling test boundary conditions (ref. to TN 2000-28).

- Swelling Pressure p_s [MPa]

Definition $p_s = \sigma_{max}$ .... (25)

with: σ_{max} = maximum swelling pressure for a defined amount of expansion (swelling strain)

Reference: ISRM (1981), p. 90-91
Paul (1986)

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

For results of the swelling pressure p_s refer to Table 4-14 and Fig. B-8.

p_0 [MPa]	n	Sample direction	Swelling pressure p_s (*) Opalinus clay material	Reference
5.4	4	$\perp$ ss	$p_{s \perp ss}$ = 1.2 [MPa]	TN 97-06
5.5	4	// ss	$p_{s // ss}$ = 0.6 [MPa]	

Table 4-14. Swelling pressure p_s of the silty-shaly facies of the Opalinus as deduced from laboratory tests. p_o = pre-loading pressure

(*) Note: Estimated value only as there are an insufficient data base and incomplete specifications of the swelling test boundary conditions (ref. to TN 2000-28).

4.4.2 Deformability and strength as a function of the water content w

The deformational and strength parameters are sensitively dependent on the moisture content w of the Opalinus clay. The parameters determined in Sections 4.1 and 4.2 are valid for the natural water content of w = 6.1 ± 1.9 % (ref. to Section 3.4). Higher water contents are generally associated with lower deformation moduli E, decreased strength and a tendency for a more ductile material behaviour. An example of the dependency of the strength from the water content is given in Fig. B-9 of Appendix B.

5. Deformation and Failure Mechanisms

The term “deformation and failure mechanisms” is coined for the processes which are relevant at large deformations including the point of failure and beyond. There might be alternative mechanisms for the one material, depending on its state, the conditions of loading and on the type of problem in question.

5.1 Undisturbed samples

The deformation mechanism relevant for the conditions at Mont Terri is “cata-clastic flow” (Nüesch, 1991). At a macro-scopic scale, this mechanism is manifested by either *ductile or brittle material behaviour* of undisturbed Opalinus Clay samples. The behaviour is controlled by a number of factors, amongst them:

- structure of clay particles
- water content
- chemistry of both clay particles and water
- environmental conditions (e.g. confinement; temperature; pre-loading)
- load velocity and consolidation conditions.

In natural conditions and to a depth of about 500 m, the Opalinus Clay is just at the point of brittle-ductile transition. Small changes (e.g. of the water content; confinement; temperature; loading conditions) can trigger a change of the deformation mechanism, e.g. from ductile to brittle and *vice versa*.

At a micro scale, the mechanism of undisturbed samples of Opalinus Clay may be characterised by the type of contact between the clay particles, e.g. edge-to-plate, plate-to-plate or edge-to-edge contact. The amount of water bound to the particles is also of importance (e.g. swelling and shrinkage).

5.2 Mechanisms in the EDZ of the New Gallery

The EDZ-experiment of the New Gallery was carried out with extensive monitoring of both the lining of the tunnel and the surrounding rock (ref. to Points A.3, A.6, A.7 and A.8 of Appendix A). Monitoring was complemented by numerical modelling both ahead and after excavation. The type of monitoring included deformation monitoring of the excavated tunnel (by convergence tapes) and of the rock (by extensometer and inclinometer); stress monitoring of the lining (by pressure cells) and of the rock (by borehole strain cells); groundwater pressure monitoring (by piezometer) and structural monitoring of the rock (by acoustic emission, seismic borehole logging and linewise deformation observation along boreholes).

With regard to deformation and failure mechanisms, the main results of the EDZ-experiment of the New Gallery were as follows:

- Overall, the rock surrounding the New Gallery is stable and needs only minor support if any. The rock pressure exerted onto the shotcrete lining was up to 0.1 MPa which is insignificant by all tunnelling standards. The main function of the lining is to prevent the excavated rock from slaking and gradual erosion due to the moisture and temperature changes within the ventilated air.

- The excavation of the New Gallery is associated with structural changes of the surrounding rock in the form of cracking (predominately tensile cracking). The degree of cracking is minor in comparison with other host rocks for nuclear repositories.

- The extent of the EDZ is in the order of a few metres. The extent is greatest in the crown and invert and smallest in the sidewall rocks (Fig. 5-1).

- There is a clear tendency for a gradual return of the EDZ to a more homogeneous and isotropic behaviour with time. This tendency is revealed in seismic borehole logging (Fig. 5-2), and is evidenced by a beginning recovery of the groundwater pressure (TN 98-58) and by pressure cell readings in the shotcrete lining (TN 98-52).

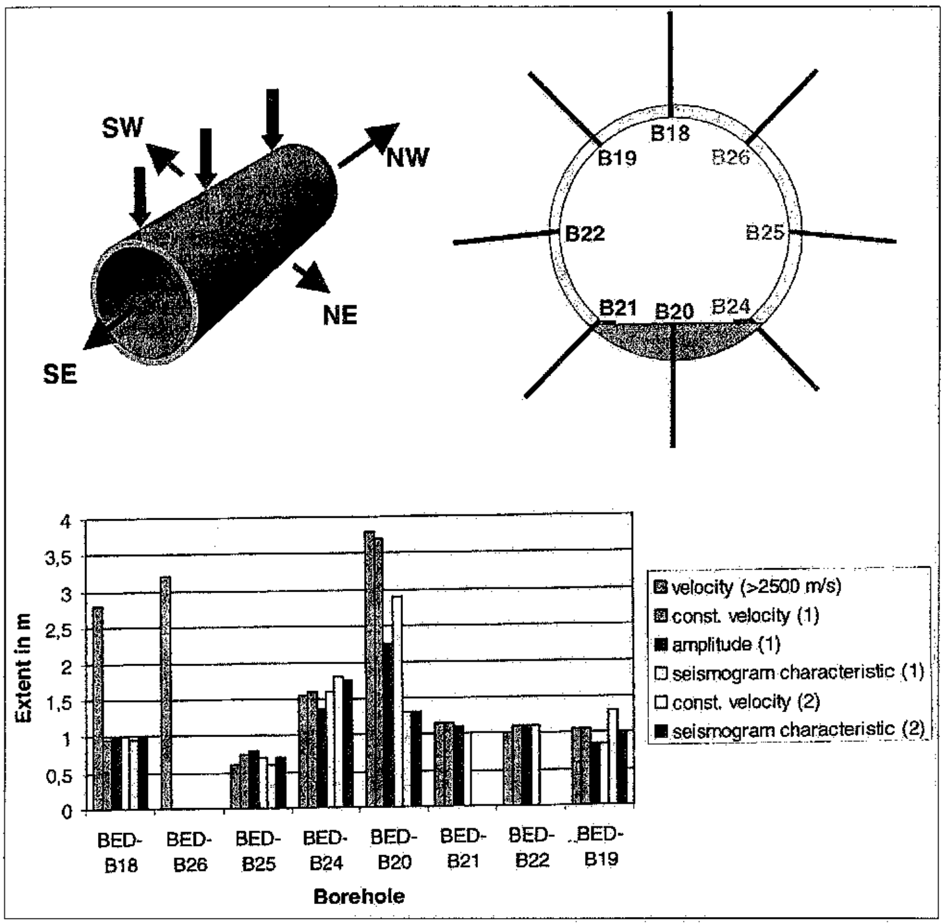


Fig. 5-1. EDZ-experiment: Location of 8 radial boreholes B 18 ... B 26 in the New Gallery (top) and extent of the disturbed zone from interval velocity measurements (bottom) (after TN 99-62).

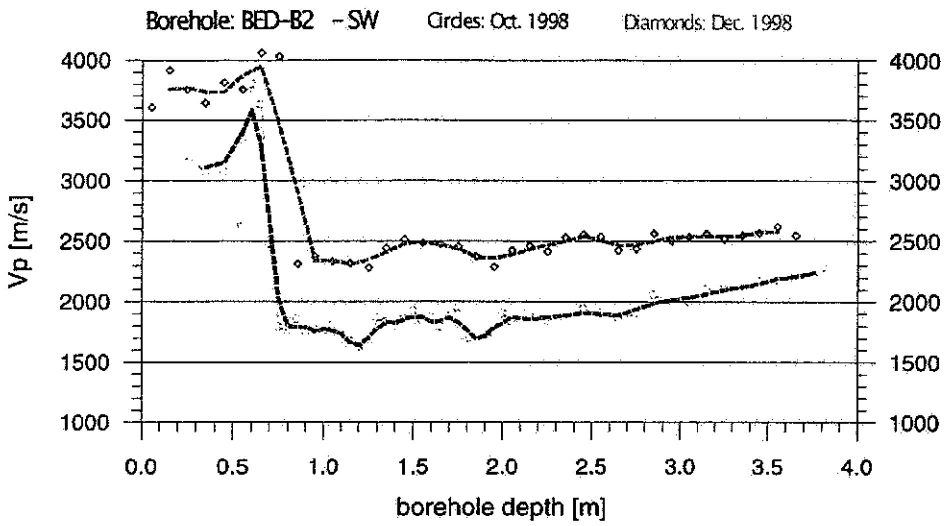


Fig. 5-2. EDZ-experiment: Recovery of the ultrasonic velocities within a time span of two months - Invert (Borehole B 20). (out of TN 99-62).

5.3 Permeability

The derived values of K and k, respectively are based on Darcy's law. This law is valid for laminar flow and for homogeneously distributed voids which remain constant throughout the flow (i.e. no erosion of particles; no swelling and shrinkage). However, in silty-clayey grounds including the Opalinus Clay, all of the above conditions will hardly prevail because of the sizeable capillary and molecule forces which are exerted between the particles and water. These forces lead to a dependency of the permeability on the hydraulic gradient in the sense that the permeability of the ground will be over-proportionally reduced at small hydraulic gradients (ref. also to DIN 18130; Part 1).

5.4 Effect of increased temperature (heat)

There is no experimental information available at this point in time on the effect of increased temperatures on the mechanical parameters of the Opalinus Clay.

5.5 Scale effect

It is generally accepted that the response of rock to an imposed load shows a pronounced effect on the size of the loaded volume (Brady and Brown, 1985). This so-called scale effect is related to the discontinuous nature of a rock mass. Joints and other geological fractures are ubiquitous features, and thus the deformational and strength properties of a rock mass are influenced by both the properties of the rock material (e.g. rock core samples) and those of the various geological fractures.

In rock mechanics often a "rule of thumb" is employed stating that with regard to the rock mechanical parameters there is a scale effect in the order of 10 between the laboratory and the prototype (Natau, 1990). In this sense the Young's modulus $E_{rock\ mass}$ of the rock mass, for instance, would be 1/10 of $E_{material}$ as determined with intact rock samples in the laboratory. For the Opalinus Clay, however, evidence suggests that with regard to the deformational and strength parameters the scale effect factor will be less significant and, in fact, close to 1. The evidence is as follows:

- By rock mechanics standards the Opalinus Clay is a poorly jointed rock with a comparatively high degree of homogeneity.
- One of its mechanically most important geological features are the bedding planes. They are pervasive structures and well represented even in comparatively small laboratory samples of the order of 10 to 100 millimetres. The results from the laboratory are therefore representative for a much larger scale.
- In the Opalinus Clay the shear strength parameters of the discontinuities are not as drastically reduced as is common with other types of rock (e.g. granite). In Section 4.2.2 it was shown that the effective cohesion c' and friction angle ϕ' of the material are not distinctively different from those of the bedding planes (and supposedly of the other joints). The dilatation angle i is close to zero (Section 4.2.3).
- Successful back analyses of the EDZ-setting (TN 98-30 and TN 99-53) were carried out without relying on scaled-down rock mechanical input parameters.

6. Recommendation for the Selection of the Rock Mechanical Parameters

Based on the data sets of Sections 3 and 4 and the considerations of Section 5, it is recommended to consider the rock mechanical parameters compiled in Tables 6-1 and 6.2 to be representative for the Mont Terri region (ambient conditions with an overburden of 250 m).

Index / State Parameter	Section	Symbol		Value		[Unit]
Bulk Density	3.1	ρ	=	2 450	± 30	[kg/m³]
Bulk Density (dry)	3.2	ρ_d	=	2 340	± 60	[kg/m³]
Grain Density	3.3	ρ_s	=	2 710	± 30	[kg/m³]
Water content	3.4	w	=	6.1	± 1.9	[%]
Porosity	3.5	n	=	13.7	± 3.1	[%]
P-wave velocity	3.6	$v_{p\ //\ ss}$	=	3 410	± 240	[m/s]
		$v_{p\ \perp\ ss}$	=	2 620	± 400	[m/s]
S-wave velocity		$v_{s\ //\ ss}$	=	1 960	± 120	[m/s]
		$v_{s\ \perp\ ss}$	=	1 510	± 250	[m/s]
Dynamic Young's modulus	3.6	$E_{dyn\ //\ ss}$	=	23.7	± 3.2	[GPa]
		$E_{dyn\ \perp\ ss}$	=	11.9	± 1.6	[GPa]
Dynamic Poisson's ratio		$\nu_{dyn\ //\ ss}$	=	0.24	± 0.03	[-]
		$\nu_{dyn\ \perp\ ss}$	=	0.28	± 0.02	[-]
Atterberg Limits	3.7	w_l	=	38	± 5	[%]
		w_p	=	23	± 2	[%]
		P.I.	=	15	± 3	[%]
Carbonate content	3.8	C_{RCO3}	=	9.4	± 5.9	[%]
$CaSO_4$ content	3.9	C_{CaSO4}	=	0.26	± 0.05	[%]
Fracture toughness	3.10	$K_{IC\ //\ ss}$	=	0.53	± 0.09	[MN/m$^{1.5}$]
		$K_{IC\ \perp\ ss}$	=	0.12	± 0.03	[MN/m$^{1.5}$]
Bridgman pinch-off	3.11	$p_{mco\ //\ ss}$	=	13	± 9	[MPa]

Table 6-1. State and index parameters of the Mont Terri region

Design Parameter	Section	Recommended value	Remarks
▪ Deformation parameters of isotropic rock model			
Tangent modulus	4.1.1	$E_{t\text{-}50}$ = 5 [GPa]	(1) (3)
Unloading, reloading and Young's modulus	4.1.1	$E_u = E_r = E$ = 6 [GPa]	(2) (3)
Poisson's ratio	4.1.1	ν = 0.27 [-]	
▪ Deformation parameters of transverse isotropic rock model			
Tangent modulus	4.1.1	$E_{t\text{-}50\ 1}$ = 3 [GPa]	(1) (3)
		$E_{t\text{-}50\ 2\ and\ 3}$ = 6 [GPa]	
Unloading, reloading and Young's modulus		E_1 = 4 [GPa]	(2) (3)
		$E_2 = E_3$ = 10 [GPa]	
Poisson's ratio		ν_{23} = 0.33 [-]	
		$\nu_{12} = \nu_{13}$ = 0.24 [-]	
Shear modulus		$G_{12} = G_{13}$ = 1.2 [GPa]	
▪ Strength parameters			
Uniaxial compressive strength	4.2.1	$UCS_{\perp ss}$ = 16 [MPa]	(4)
		$UCS_{// ss}$ = 10 [MPa]	(5)
Uniaxial tensile strength		$UTS_{\perp ss}$ = 1 [MPa]	(6)
		$UTS_{// ss}$ = 2 [MPa]	
Shear strength of material	4.2.2	$c'_{// ss}$ = 2.2 [MPa]	
		$c'_{\perp ss}$ = 5.0 [MPa]	(5)
		$\phi'_{// ss} = \phi'_{\perp ss}$ = 25°	
Shear strength of bedding planes		$c'_{bedding}$ = 1 [MPa]	
		$\phi'_{bedding}$ = 23°	
Dilatation of material	4.2.3	$\delta_{\perp ss}$ = $-2\,000 \cdot 10^{-6}$	(7)
		$\delta_{// ss}$ = $-4\,000 \cdot 10^{-6}$	
Dilatation angle		i = 0°	

Table 6-2. to be continued next page

Table 6-2. continued

▪ Permeability parameters			
Permeability	4.3	$k_{\perp ss} = 8 \cdot 10^{-21}$ [m²]	(8)
		$k_Z = 2 \cdot 10^{-20}$ [m²]	
▪ Hydro-mechanically coupled parameters			
Swelling strain index	4.4	$S_{\varepsilon \perp ss} = 7$ [%]	(9)
		$S_{\varepsilon // ss} = 1$ [%]	
Swelling pressure		$p_{s \perp ss} = 1.2$ [MPa]	(10)
		$p_{s // ss} = 0.6$ [MPa]	

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

Remarks:

(1) Evaluated as the mean of all relevant laboratory test values, irrespectively of the influence of the confinement (σ_3).
The computed value was rounded off to the next full *lower* number (to adjust for scale effects).

(2) Evaluated as the mean of all relevant laboratory and field values, irrespectively of the influence of the confinement (σ_3).
The computed value was rounded off to the next full *lower* number (to adjust for scale effects).

(3) The parameters E and $E_{t\text{-}50}$ do not represent the deformational characteristics at very low stress levels. A separate modulus
$E_v < E_{t\text{-}50} < E$ may be introduced as indicated on p. 23.

(4) Adjusted to lower values (from about 25 to 16 MPa) due to considerations in relation with the Mohr envelope.

(5) Rounded off to the next full *lower* number (to adjust for scale effects).

(6) Estimated value (ref. to p. 28).

(7) Generously rounded off because of extremely high standard deviation.

(8) Preliminary values (ref. to p. 33).

(9) Preliminary values (ref. to p. 34 f.).

(10) At a pre-loading pressure of about 5.5 MPa (= equivalent to overburden pressure).

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Appendix A: Source Documents

Technical Notes (TN) which comprise the base for the Technical Report at hand.

A.1 General setting

A.1.1	TN 98-49	Hohner, M. and Bossart, P.	Geological etc. ... parameters
A.1.2	TN 99-38	Bossart, P. and Wermeille, S.	Borehole data
A.1.3	TN 99-38b	Bossart, P. and Wermeille, S.	Borehole data (revised)

A.2 Tests in relation with the in-situ stress measuring experiments IS-A, IS-B and IS-C

A.2.3	TN 97-11	Cottour, P.	IS-A: Handling of samples
A.2.5	TN 97-13	Homand, F. et al.	IS-A: Results of laboratory tests
A.2.8	TN 97-16	Bühler, Ch.	IS-B: Dilatometer test results

A.3 Excavation Disturbed Zone experiment ED-B around New Gallery

A.3.1	TN 97-31	Velasco, M. and Pedraza, L.	Scoping calculations
A.3.2	TN 98-13	Alfrageme, F. and Cabal, P.	Geotechnical logging
A.3.3	TN 98-31	Alfrageme, F. and Cabal, P.	Geotechnical assessment
A.3.4	TN 98-57	Olalla, C. et al.	Laboratory tests
A.3.5	TN 98-08	Thut, A.	Installation of sliding micrometer
A.3.6	TN 98-58	Fierz, T.	Sliding micrometer / inclinometer
A.3.7	TN 99-25	Fierz, T.	Sliding micrometer / inclinometer
A.3.8	TN 99-68	Fierz, T.	FIM
A.3.9	TN 98-52	Fierz, T.	Pressure cells
A.3.10	TN 98-12	Mathier, J.-F. et al.	Extensometer
A.3.11	TN 99-37	Mathier, J.-F. et al.	Convergence measurements
A.3.12	TN 99-37a	Mathier, J.-F. et al.	Convergence measurements
A.3.13	TN 99-12	Dupuis, D. et al.	PAC-EX
A.3.14	TN 98-09	Bigarré, P.	Stress monitoring
A.3.15	TN 98-30	Velasco, M. and Ruiz, F.	FLAC 3D: Predictive calculations

A.4 Mechanical properties of the Opalinus Clay

A.4.1	TN 96-22	Homand, F. and Pepa, S.	Laboratory tests
A.4.2	TN 98-18	Rummel, F. et al.	Gas-frac self-healing experiment
A.4.3	TN 99-36	Rummel, F. and Weber, U.	Gas-frac self-healing (Phase 4)
A.4.4	TN 98-35	Rummel, F. et al.	Laboratory tests
A.4.5	TN 98-55	Rummel, F. et al.	Laboratory tests
A.4.6	TN 99-35	Rummel, F. et al.	Laboratory tests
A.4.7	TN 98-47	Suzuki, K. and Maruyama, M.	DM: Laboratory tests

A.5 Hydro-mechanical, swelling and creep properties of the Opalinus Clay

A.5.1	TN 96-22	Homand, F. et al.	Laboratory tests
A.5.2	TN 97-06	Vögtli, B. and Bossart, P.	Swelling experiment
A.5.3	TN 97-06rev.	Vögtli, B. and Bossart, P.	Swelling experiment
A.5.4	TN 98-36	Chiffoleau, S. & Robinet, J.C.	Hydromech. Characteristics
A.5.5	TN 97-41	Bühler, Ch.	IS-B: Long-term dilatometer
A.5.6	TN 99-03	Bühler, Ch.	DM: Long-term dilatometer
A.5.7	TN 99-78	N. N. (ANTEA)	DM: Laboratory creep tests
A.5.8	TN 97-26	Horseman, St.	Osmotic pressure experiment
A.5.9	TN 98-15	Ortiz, L.	GP: Laboratory tests

A.6 ED-B "tomography" experiment

A.6.1	TN 98-05	Forney, F. and Bigarré, P.	Site and equipment
A.6.2	TN 98-06	Forney, F. and Bigarré, P.	Field report
A.6.3	TN 98-07	Forney, F. et al.	Preliminary test data
A.6.4	TN 98-10	Forney, F. et al.	"Tomography" test results

A.7 ED-C seismic borehole measurements

A.7.1	TN 97-33	Alheid, H.-J. et al.	Feasibility study
A.7.2	TN 97-34	Alheid, H.-J. et al.	Seismic borehole measurements
A.7.3	TN 99-62	Schuster, K. et al.	Interval velocity measurements

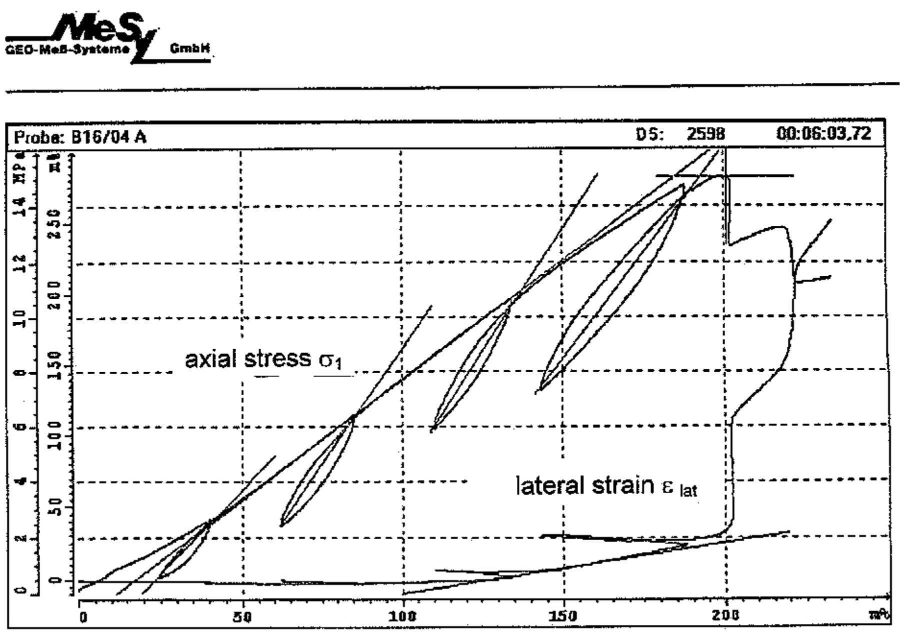
A.8 Numerical modelling

A.8.1	TN 98-54	Konietzky, H.	PFC-3D: Back calculation
A.8.2	TN 98-56	Konietzky, H. & Kamp, L. te	FLAC-2D: Back analysis
A.8.3	TN 99-53	Kamp, L. te & Konietzky, H.	PFC: Modelling of EDZ
A.8.4	TN 99-69	Konietzky, H.	DM experiment

A.9 RB-Experiment

A.9.1	TN 99-34	Velasco, M. and Pedraza, L.	Scoping calculations

Appendix B: Plots from relevant laboratory and field tests



E_{ges} = 8.7 GPa ν = 0.36 E_{EW} = 15.5 ± 2.6 GPa $\sigma_{1,max}$ = 15.0 MPa

Figure B-1. Example of a laboratory test for the determination of the elastic parameters (out of TN 98-18)

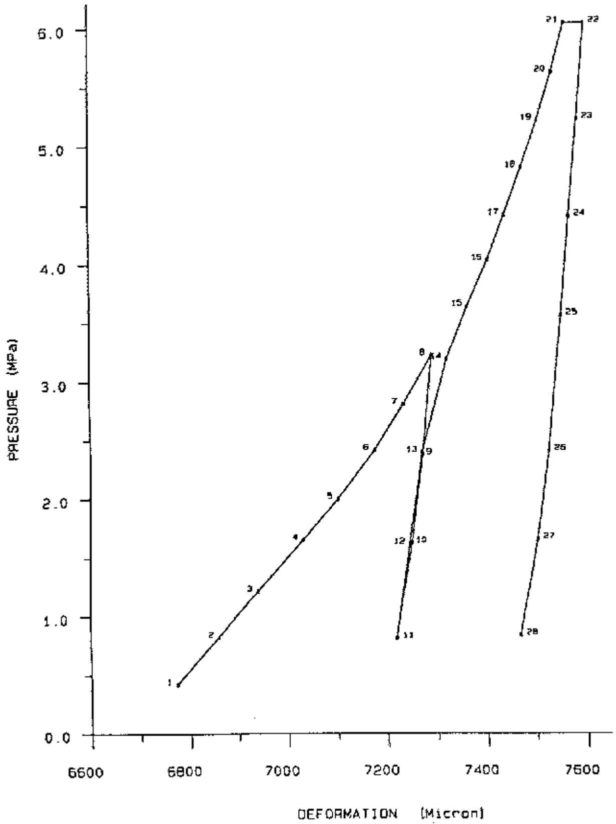


Figure B-2. Example of a field test for the determination of the deformation moduli (out of TN 97-16)

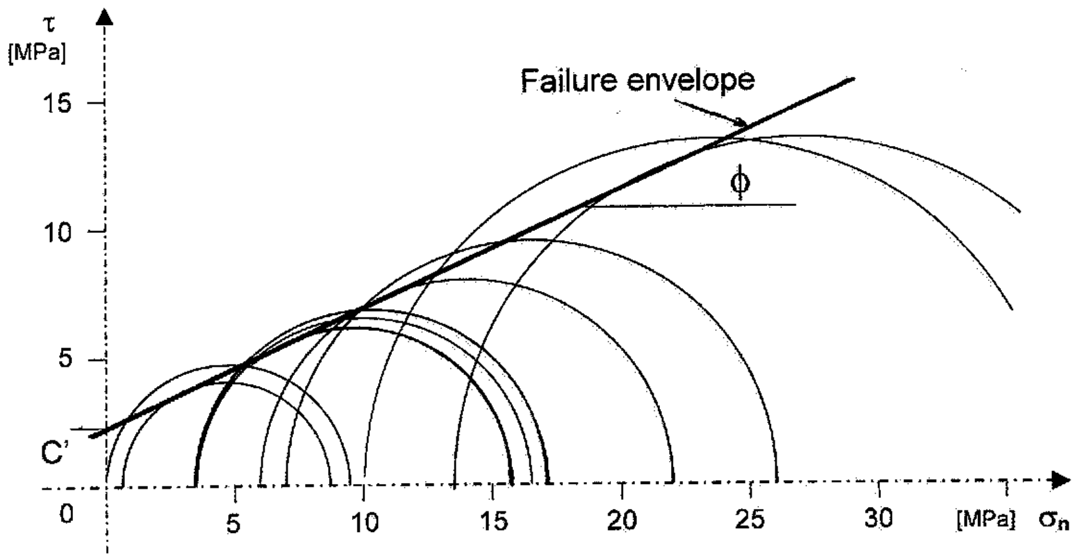


Figure B-3. *Material strength properties in a Mohr-Coulomb diagram (raw data from TN 98-57). Envelope with c' = 2.2 MPa and ϕ = 25°*

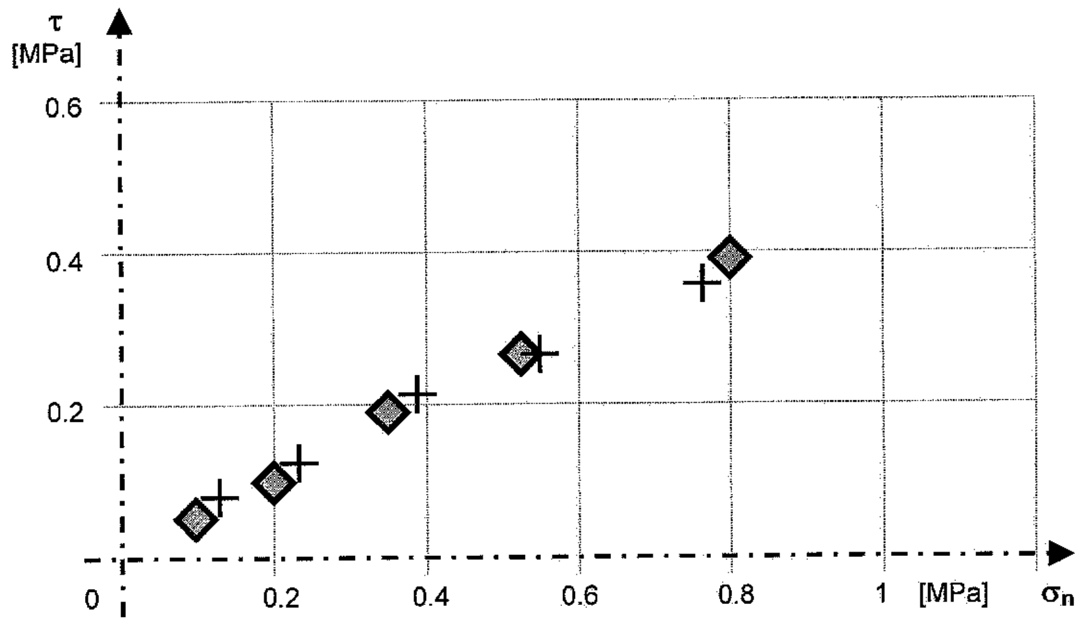


Figure B-4. *Strength of discontinuities as deduced from direct shear laboratory test (raw data from TN 98-57).*
Envelope with c = 0.03 MPa and ϕ = 22°

Legend: ◆ *Sample M 9953I* + *Sample M 9953II*

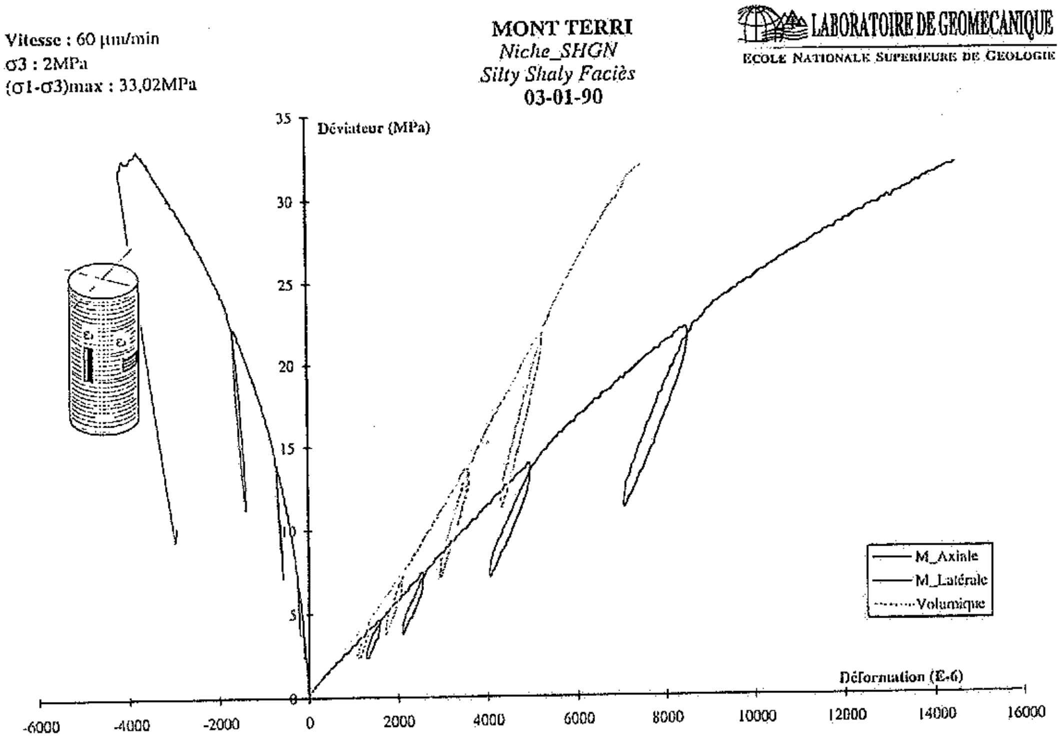


Figure B-5. *Axial, lateral and volumetric strain for the determination of the material dilatancy at failure (out of TN 96-22).*

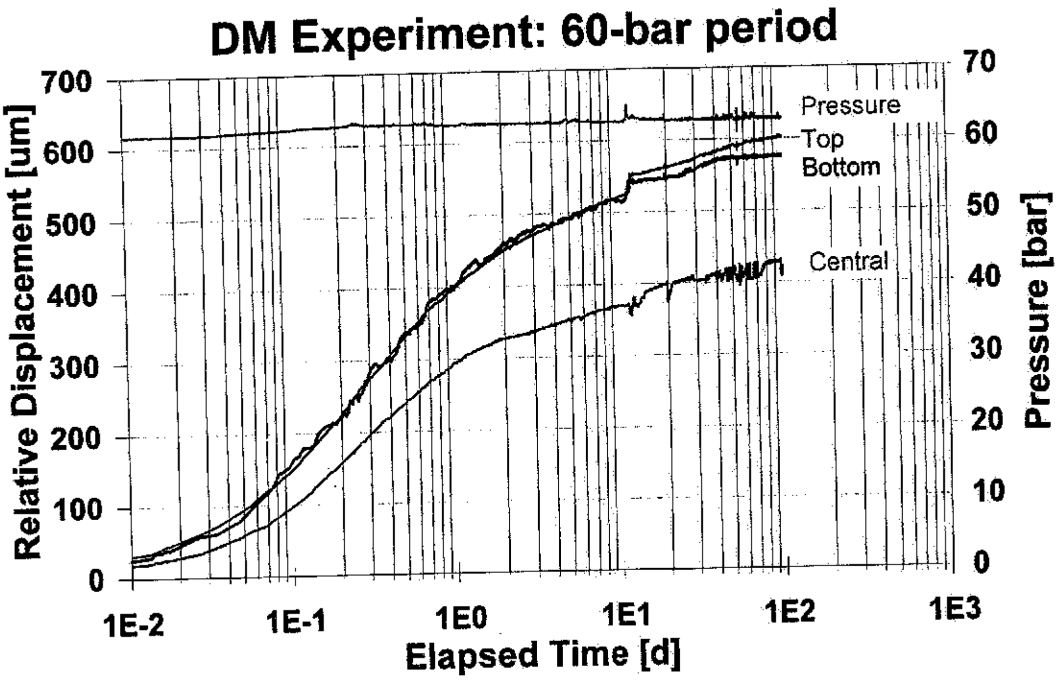


Figure B-6. *Semi-log plot of a borehole dilatometer creep test at a constant pressure of about 60 bar. Note the consolidation-controlled deformation behaviour of the Opalinus Clay as indicated by all three displacement transducers Top – Central - Bottom (out of TN 99-03).*

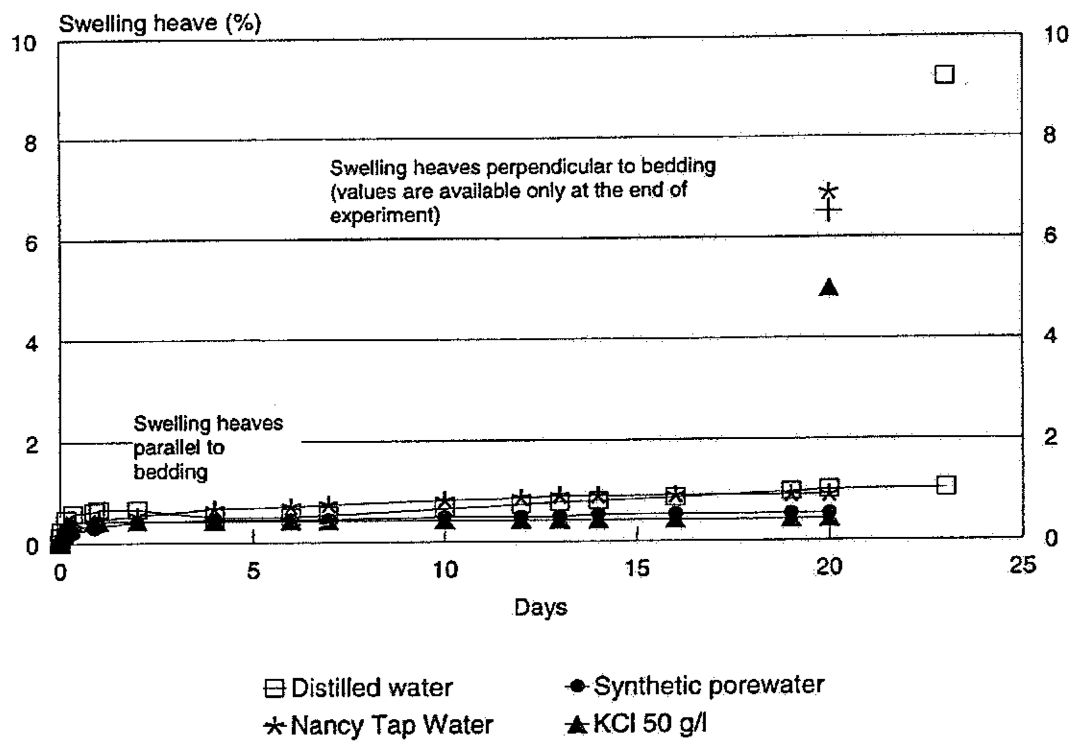


Figure B-7. Swelling heave curves of shaly samples (out of TN 97-06).

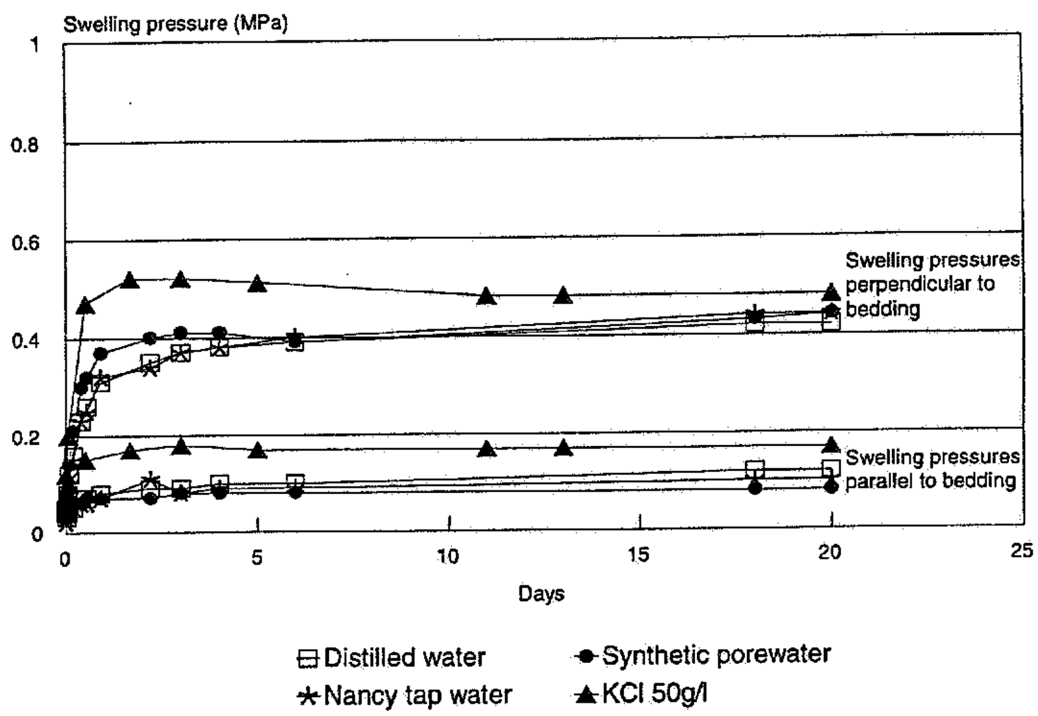


Figure B-8. Swelling pressure curves of shaly samples (out of TN 97-06).

a) **P - samples**

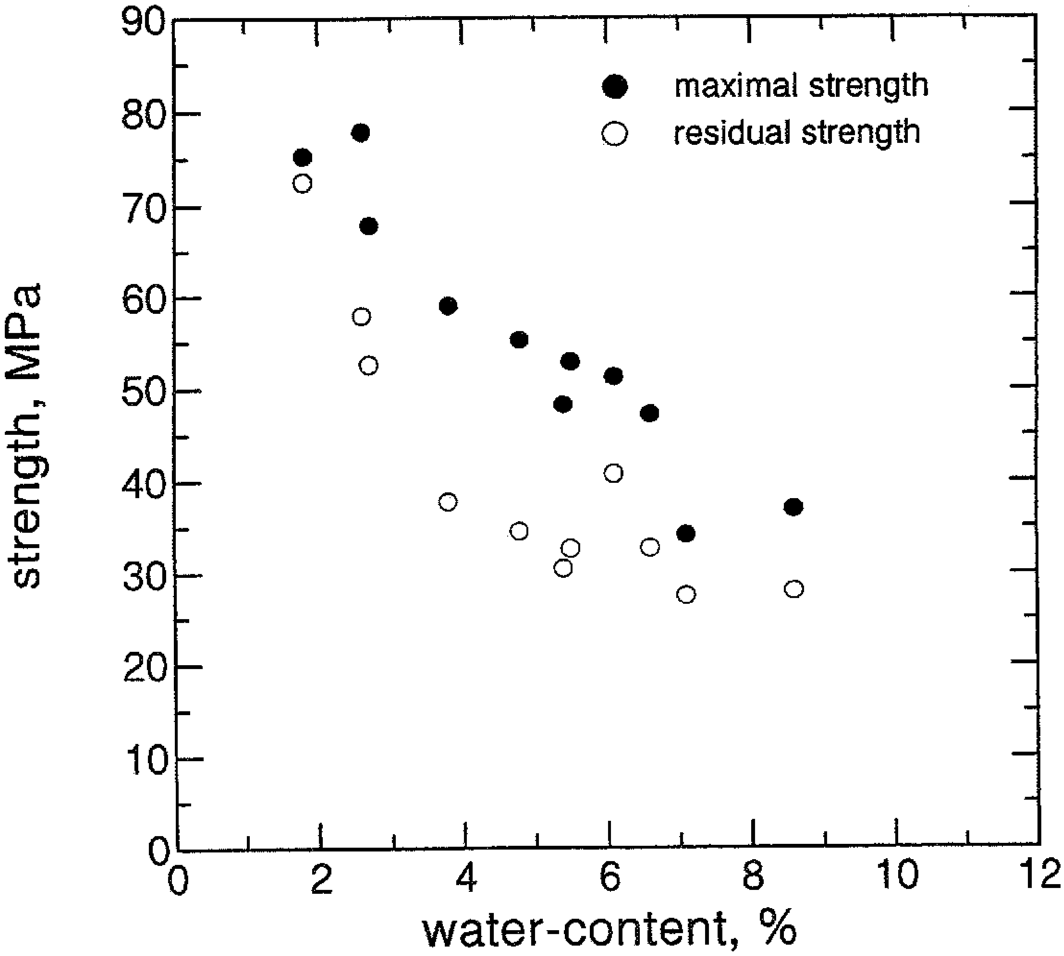


Figure B-9. Example of the dependency of material strength from the water content w (out of TN 99-35) (Confining pressure σ_3 = 10 MPa).