Deep excavation in an anisotropic argillaceous rock:
Observations and analysis

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ABSTRACT

A zone with significant irreversible deformations and significant changes in flow and transport properties (EDZ – Excavation Damage Zone) is expected to be formed around underground excavations in indurated clays as a consequence of stress redistribution. In this thesis, the influence of the stress redistribution around a 4.5m diameter gallery and a 30cm diameter borehole on excess pore pressure has been investigated on the basis of in situ measurements and numerical simulations. The relationship between the development of the EDZ and the excess pore pressure is put forward and used to improve the characterization of the EDZ. A set of parameters is determined on the basis of a literature review and further calibrated to improve the agreement between the pore water pressure measurements realised during the drilling of the borehole and the simulation results. The rigidity anisotropy in combination with a simple yield criterion is shown to provide a satisfactory explanation for the observed water pressure response. The determined parameter set is then used to simulate the larger scale opening and proved able to reproduce the observed rock response despite of the significant different observed behaviour in the two cases.

The work was structured in the following steps:

1. Critical review of the existing documentation

2. Acquaintance with the Finite Element code Code_Bright, in which a Hydro-Mechanical (HM) formulation is implemented.

3. Study of available numerical tools

4. Numerical analysis of excavation processes in argillaceous rocks and process understanding. Theoretical study of the effect of anisotropy (properties and initial stresses)
RESUMEN

Al realizar una excavación en rocas arcillosas se espera que se forme una zona con importantes deformaciones irreversibles y cambios significativos en las propiedades de flujo y el transporte (EDZ – Excavation Damage Zona). En esta tesina se ha investigado la influencia de la redistribución de la tensión en torno a una excavación de 4,5 m de diámetro de una galería y un pozo de 30 cm de diámetro el exceso de presione de poro a partir de las mediciones in situ y simulaciones numéricas. Se ha utilizado la relación entre el desarrollo de la EDZ y la presión de poros para mejorar la caracterización de la EDZ. Se ha determinado un conjunto de parámetros a partir de la revisión de la bibliografía y se ha calibrado para intentar mejorar la relación entre las mediciones de la presión de poro de agua realizadas durante la perforación del pozo de sondeo y los resultados de la simulación. La anisotropía de rigidez en combinación con un criterio simple de fluencia ha mostrado una explicación satisfactoria de la respuesta de presión de agua observada. El conjunto de parámetros determinado se ha utilizado para modelar la excavación a mayor escala y ha demostrado ser capaz de reproducir la respuesta observada roca a pesar de la importante diferencia de comportamiento entre los dos casos.

El trabajo se ha estructurado de la siguiente forma:

1. Revisión crítica de la documentación existente
2. Familiarización con el código de elementos finitos CODE_BRIGHT, con el que se lleva a cabo un análisis hidro-mecánico
3. Estudio de las herramientas numéricas disponibles
4. Análisis numérico de los procesos de excavación en rocas arcillosas y la comprensión de los diferentes procesos. Estudio teórico del efecto de la anisotropía (propiedades y las tensiones iniciales)
1. INTRODUCTION

1.1 MOTIVATION

Argillaceous formations are widely considered to be suitable as a geological barrier for radioactive waste disposal and several countries are currently investigating the suitability of various clay formations for this purpose. In Switzerland, a Mesozoic shale formation, the Opalinus Clay, is being considered as a potential repository host rock (Bossart, 2002).

During the construction of a repository in the Opalinus Clay, the host rock properties around the underground openings are expected to be considerably altered. Depending on the mechanical rock properties, initial stress field and applied excavation techniques, plastic deformations around the galleries of the facility may occur. Such an altered zone is called an excavation disturbed zone (EDZ). Due to stress redistribution during excavation and subsequent rock convergence, a fracture network consisting of unloading joints and unloading faults is formed in this EDZ (Bossart, 2002). The hydraulic conductivity of these fracture networks may be orders of magnitude higher than that in the intact Opalinus Clay which may provide a preferential path for radionuclide transport along repository galleries and shafts. It is of fundamental interest for the performance assessment of a potential repository to know the geometry and extent of such an EDZ and its fracture density, but also to have accurate estimates of parameters such as the hydraulic conductivity on the scale of the repository. Furthermore, many scientific experiments or engineering projects in the Opalinus Clay (self-healing experiment, mine-by tests during excavations) and the future sealing of the shafts after closure of a repository rely on accurate structural and hydraulic characterization of the EDZ.

1.2 OBJECTIVES

The objectives of the present work are twofold. The first one is to improve the characterization of hydro-mechanical properties of the Opalinus Clay on the basis of in situ experiments. Feedback from the laboratory is used as a starting point and some differences between laboratory characterization and field parameters are reported.

The second objective is to analyse the rock behaviour during excavation processes at different scales. Pore water pressure and the development of an EDZ are investigated with particular attention. Coupled hydro-mechanical (HM) simulations have been used to that purpose. The effect of various features such as strength, rigidity, rigidity anisotropy, time dependent deformations, initial stress state and permeability are explored in a systematic way.

1.3 THESIS LAYOUT

The Thesis is organized in five chapters reflecting the workflow presented in the abstract. The chapter content is as follow:

- Description of the Mont Terri Project and general information
- Description of the theoretical formulation implemented in Code_Bright
- Review of Numerical Methods in Geotechnical Engineering
- Sensitivity analysis, model calibration and process understanding
2. THE MONT TERRI PROJECT

In many countries, argillaceous formations are being considered as potential host rocks for repositories of radioactive waste. Therefore, in 1995 several organisations decided to start an international research project in a Mesozoic shale formation in north-western Switzerland, the Opalinus Clay (Aalenian). The typical layout for a waste disposal facility is shown in Figure 2-1. Different processes will occur during construction and operation phases of such a facility and should be investigated.

![Figure 2-1 Possible layout of a deep geological repository for high-level waste.](image)

The aim of the project is to analyse the hydrogeological, geochemical and rock mechanical properties of an argillaceous formation, the changes of these properties induced by the excavation and ventilation of galleries and to evaluate and improve appropriate investigation techniques (Thury, 1999).

A rock laboratory offers more realistic test conditions that can be achieved in conventional laboratory studies. Experiments can be carried out on a 1:1 scale and provide important information on the feasibility and safety of deep geological disposal. However, they need to be complemented by natural analogue studies, modelling and site investigation (Figure 2-2).

![Figure 2-2 Underground Rock Laboratories (URL) in the framework of performance assessment](image)

2.1 GEOGRAPHIC SITUATION AND LAYOUT

The Mont Terri Rock Underground Laboratory (URL) lies to the north of the town of St-Ursanne in Canton Jura (Figure 2-3). It is located at a depth of around 300 metres below the earth’s surface and is accessed through the security gallery of the Mont Terri
Deep excavation in an anisotropic argillaceous rock
The Mont Terri Project

Tunnel of the A16 ‘Transjurane’ motorway, which passes through the Jura Mountains. The research galleries have a total length of around 500 metres.

Figure 2-3 A) Geographic position of the Mont Terri URL. B) Aerial view of the Laboratory (Mont Terri Project, 2012)

A 3D layout of the URL is shown in Figure 2-4 and Figure 2-5. The first experiments (1996) were carried out in eight small niches along the security gallery. A separate, dedicated research gallery was then constructed in 1998 and extended in 2004 and 2008. The Rock Laboratory infrastructure is for research purposes only.

Figure 2-4 3D view of the Mont Terri Underground Laboratory (Mont Terri Project, 2012).
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The Mont Terri Project

2.2 GEOLOGY OF MONT TERRI

The Mont Terri URL is placed in the Opalinus Clay, a rock formation consisting mainly of incompetent, silty and sandy shales, deposited around 180 Ma ago (Aalenien). The Opalinus Clay formation can be characterised as an overconsolidated shale formation (present overburden 270 m, estimated overburden in the past at least 1000m) with 40-80% clay minerals (mainly kaolinite and illite) and micas, 10-40% quartz, 5-40% calcite, 1-5% siderite, 0-1.7% pyrite and 0.1-0.5% organic carbon (A.G. Corkum, 2007). At the laboratory three facies of Opalinus Clay can be distinguished: i) the shaly facies, an homogeneous, barely visible laminated clayston with low sand content; ii) sandy facies, the most layered claystone, penetrated by thin sand layers and lenses; and iii) carbonate-rich sandy facies, a sandy limestone with bioclastic material and quarts grains (Figure 2-6).

The name Opalinus Clay is derived from a particular ammonite, the Leioceras opalinum (Figure 2-7), whose shell consists of aragonite and, when backlit, shimmers with a bluish opalescent glow.

Figure 2-6 The three types of facies of Opalinus Clay (OPA) distinguished at the Mont Terri URL: (a) shaly facies, (b) sandy facies and (c) carbonate-rich sandy facies (Mont Terri Project, 2012).
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Figure 2-7 (a) The Opalinus Clay structure is highly oriented, parallel to bedding. Clay platelets are generally bent and the presence of quartz grains can be seen in the clay matrix (Corkum., 2007). (b) The Leioceras opalinum ammonite occurs frequently in the Opalinus Clay.

The Jura mountains were folded during the Late Miocene to Pliocene. They form an arcuate chain, comprising the Plateau Jura, mainly in France and, to the southeast, the Tabular and the Folded Jura. The Mont Terri Underground Rock Laboratory (URL) is located in the Folded Jura. The Jura rocks consist of competent limestones and incompetent marly, shaly formations, which were deposited in a coastal to shallow marine environment from the Triassic to the early Cretaceous.

A simplified NW-SE profile through the Mont Terri anticlines is shown in Figure 2-8. The southern limb of the Mont Terri anticline exhibits a simple geometry with gently to moderately SE-dipping strata and without major tectonic discontinuities. Because of the highly deformed northern limb and the relatively low overburden, the rock laboratory was placed in the southern limb of the anticline where the geometry is simple and the deformation weak.

Figure 2-8 Vertical section through the Mont Terri anticline along the motorway tunnel (Huggenberger, 2003).

A simplified geological map is shown in Figure 2-8. The main tectonic structure of the considered area is the Mont Terri anticline. This forms the northernmost anticline of the Folded Jura and was thrust at least 1 km over the Tabular Jura. The geometry of this almost 50-km-long anticline is variable along its strike, changing from a boxfold type in the west to an asymmetrical, dome shaped fold at Mont Terri. Finally, it is sliced up by strike and slip faulting into several pieces in the east.
Deep excavation in an anisotropic argillaceous rock

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The rock laboratory is intersected by a tectonic network composed by three different fault systems: i) SSE-dipping faults, ii) S- to SW dipping faults and iii) WNW- to W-dipping fault planes. Both first systems have been initiated during the Late Miocene Jura folding and thrusting towards NW, consequently to the Alpine push. The third system results to the sinistral transpressive reactivation of Oligocene inherited structures related to the sinistral transfer zone which connects the Rhine and Bresse grabens.

2.3 PROPERTIES AND KEY PARAMETERS OF OPALINUS CLAY

A geological repository has to ensure the long-term protection of man and the environment. The most important criteria for a potential host rock are very low permeability, high thermal conductivity, possibility for sealing, high strength, plastic/viscous behavior, lithostatic isotropic in-situ stresses, high sorption potential, high temperature reliability, low content of water and low resolution behaviour. The Opalinus clay has several properties that are favourable for the long-term safety of a geological repository. Besides its good isolation capacity, very low hydraulic permeability and predominantly diffusive transport of dissolved substances, these include a homogeneous structure, retention of radionuclides on clay mineral surfaces and the ability to close fissure and fractures due to swelling.

The Opalinus Clay is a distinctively bedded material. Its mechanical behaviour can best be described in a transverse isotropic model (Figure 2-10). The Opalinus Clay has high cohesive component of shear strength of 3.7 and 5.5 MPa (Bock, 2009) parallel and perpendicular to bedding, respectively. The high level of cohesion and brittle post-peak behaviour found in the Opalinus Clay indicates the extent to which diagenetic processes have resulted in strong physic-chemical bonds (Corkum, 2007). Opalinus Clay tends to yield in tension with little chance of healing without significant increase in water content. This is in agreement with extensional fractures observed around underground...
excavations in Opalinus Clay at Mont Terri, particularly in zones of high deviatoric stresses where shear mode of yielding might be anticipated.

The stress-strain response of geomaterials is required when analysing underground excavations. The stress-strain curve of Opalinus Clay, although it is considered a relatively weak rock, is similar to that of brittle rock (Figure 2-11). The stress-strain curve can be divided into five regions:

- Region I – closure of existing micro-cracks and crushing of asperities
- Region II – linear elastic
- Region III – onset of dilation (crack initiation)
- Region IV – unstable crack growth
- Region V – post peak

The deformation behaviour at low stress levels is characterised by a shallow sloping stress-strain curve near the origin, for stresses $0 < \sigma < 2 MPa$. This low-stress Region I is of most interest in the context of unloading around tunnels.

The long-term deformation has a rate dependent stress-strain behaviour, as clay materials behave in a viscoplasticity manner.

A comparison of the reported parameters has been made in order to determine the adequate parameters has been made (Table 2-1, 2-2, 2-3 and 2-4).
### Table 2-1 Comparison of reference values for basic properties of Opalinus Clay. The values adopted for the simulation are identified as “MB test”.

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<td>-</td>
</tr>
<tr>
<td>Plastic limit</td>
<td>w_P</td>
<td>%</td>
<td>23</td>
<td>2</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Plasticity Index</td>
<td>P.I</td>
<td>%</td>
<td>15</td>
<td>3</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Carbonate content</td>
<td>C_{KCO3}</td>
<td>%</td>
<td>9.4</td>
<td>5</td>
<td>14</td>
<td>6 to 22</td>
<td>-</td>
</tr>
<tr>
<td>CaCO3 content</td>
<td>C_{CaCO3}</td>
<td>%</td>
<td>0.26</td>
<td>0.05</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

### Table 2-2 Comparison of reference values for deformation parameters for an isotropic elastic model of Opalinus Clay.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Symbol</th>
<th>Units</th>
<th>MB test</th>
<th>Bock, 2001</th>
<th>Willeveau, 2005</th>
<th>Garitte et al., 2009</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Value</td>
<td>Value</td>
<td>Value</td>
<td>Value</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Range(±)</td>
<td>Range(±)</td>
<td>Range(±)</td>
<td>Range(±)</td>
</tr>
<tr>
<td>Tangent modulus</td>
<td>E_{50}</td>
<td>MPa</td>
<td>-</td>
<td>5000</td>
<td>2700±1500</td>
<td>700±200</td>
</tr>
<tr>
<td>Young's modulus</td>
<td>E</td>
<td>MPa</td>
<td>4000</td>
<td>6000</td>
<td>4000</td>
<td>7800±5100</td>
</tr>
<tr>
<td>Poisson's ratio</td>
<td>ν</td>
<td>-</td>
<td>0.2</td>
<td>0.27</td>
<td>0.33</td>
<td>0.29±0.08</td>
</tr>
</tbody>
</table>

### Table 2-3 Comparison of reference values for deformation parameters when considering a transverse isotropic elastic rock.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Symbol</th>
<th>Orientation to bedding</th>
<th>Units</th>
<th>MB</th>
<th>Bock, 2001</th>
<th>Willeveau, 2005</th>
<th>Garitte et al., 2009</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Value</td>
<td>Value</td>
<td>Value</td>
<td>Value</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Range(±)</td>
<td>Range(±)</td>
<td>Range(±)</td>
<td>Range(±)</td>
</tr>
<tr>
<td>Tangent modulus</td>
<td>E_{50,1}</td>
<td>Perpendicular</td>
<td>MPa</td>
<td>3000</td>
<td>4000</td>
<td>1000</td>
<td>5800</td>
</tr>
<tr>
<td></td>
<td>E_{50,2 and 3}</td>
<td>Parallel</td>
<td>MPa</td>
<td>6000</td>
<td>6000</td>
<td>10000</td>
<td>9300</td>
</tr>
<tr>
<td>Young's modulus</td>
<td>E_{1}</td>
<td>Perpendicular</td>
<td>MPa</td>
<td>3000</td>
<td>4000</td>
<td>1000</td>
<td>5800</td>
</tr>
<tr>
<td></td>
<td>E_{E}</td>
<td>Parallel</td>
<td>MPa</td>
<td>6000</td>
<td>6000</td>
<td>10000</td>
<td>9300</td>
</tr>
<tr>
<td>Poisson's ratio</td>
<td>ν_{23}</td>
<td>Perpendicular</td>
<td>-</td>
<td>0.35</td>
<td>0.33</td>
<td>0.33</td>
<td>0.295</td>
</tr>
<tr>
<td></td>
<td>ν_{12} = ν_{13}</td>
<td>Parallel</td>
<td>-</td>
<td>0.25</td>
<td>0.24</td>
<td>0.24</td>
<td>0.25</td>
</tr>
<tr>
<td>Shear modulus</td>
<td>G_{12} = G_{13}</td>
<td>-</td>
<td>MPa</td>
<td>3500</td>
<td>1200</td>
<td>-</td>
<td>3500</td>
</tr>
</tbody>
</table>

### Table 2-4 Comparison of reference values for strength parameters when considering a transverse isotropic elastic rock.
## Deep excavation in an anisotropic argillaceous rock

### The Mont Terri Project

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Symbol</th>
<th>Orientation to bedding</th>
<th>Units</th>
<th>MB test</th>
<th>Bock, 2001</th>
<th>Willeveau, 2005</th>
<th>Bock, 2009</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Value</td>
<td>Value</td>
<td>Value</td>
<td>Value</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Range(±)</td>
<td>Range(±)</td>
<td>Range(±)</td>
<td>Range(±)</td>
</tr>
<tr>
<td>Uniaxial compressive strength</td>
<td>UCS(\perp)</td>
<td>Perpendicular</td>
<td>MPa</td>
<td>16</td>
<td>16 - 6</td>
<td>16.4 - 6.5</td>
<td>14.9 - 5.1</td>
</tr>
<tr>
<td></td>
<td>UCS(\parallel)</td>
<td>Parallel</td>
<td>MPa</td>
<td>10</td>
<td>10 - 6</td>
<td>11.6 - 3.9</td>
<td></td>
</tr>
<tr>
<td>Uniaxial tensile strength</td>
<td>UTS(\perp)</td>
<td>Perpendicular</td>
<td>MPa</td>
<td>1</td>
<td>0.5 -</td>
<td>0.67 -</td>
<td></td>
</tr>
<tr>
<td></td>
<td>UTS(\parallel)</td>
<td>Parallel</td>
<td>MPa</td>
<td>2</td>
<td>1 -</td>
<td>1.3 - 0.2</td>
<td></td>
</tr>
<tr>
<td>Shear strength of material</td>
<td>(c')(\perp)</td>
<td>Perpendicular</td>
<td>MPa</td>
<td>1</td>
<td>5.5 -</td>
<td>11.6 -</td>
<td></td>
</tr>
<tr>
<td></td>
<td>(c')(\parallel)</td>
<td>Parallel</td>
<td>MPa</td>
<td>2.2</td>
<td>2.2 -</td>
<td>5.4 -</td>
<td></td>
</tr>
<tr>
<td></td>
<td>(\phi')(\perp)</td>
<td>Perpendicular</td>
<td>(\degree)</td>
<td>20</td>
<td>20 -</td>
<td>20 -</td>
<td></td>
</tr>
<tr>
<td></td>
<td>(\phi')(\parallel)</td>
<td>Parallel</td>
<td>(\degree)</td>
<td></td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Shear strength of bedding planes</td>
<td>(c')(\text{bedding})</td>
<td>-</td>
<td>MPa</td>
<td>1</td>
<td>-</td>
<td>0.94 -</td>
<td></td>
</tr>
<tr>
<td></td>
<td>(\phi')(\text{bedding})</td>
<td>-</td>
<td>(\degree)</td>
<td>23</td>
<td>-</td>
<td>21 -</td>
<td></td>
</tr>
<tr>
<td>Biot's coefficient</td>
<td>(b)</td>
<td></td>
<td></td>
<td>0.6</td>
<td>0.42-0.78</td>
<td>0.6 -</td>
<td>0.6 -</td>
</tr>
</tbody>
</table>

Table 2-5 Comparison of reference values for hydraulic parameters when considering a transverse isotropic elastic rock.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Symbol</th>
<th>Orientation to bedding</th>
<th>Units</th>
<th>MB test</th>
<th>Marschall, 2004</th>
<th>Gens et al., 2007</th>
<th>Bock, 2009</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Range</td>
<td>Value</td>
<td>Value</td>
</tr>
<tr>
<td>Intrinsic permeability</td>
<td>(k)</td>
<td>Isotropic</td>
<td>m(^2)</td>
<td>7.00E-20</td>
<td>4E-21 to 3E-19</td>
<td>5.00E-20</td>
<td>2.00E-20</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Perpendicular</td>
<td>m(^2)</td>
<td></td>
<td>5.00E-20</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Parallel</td>
<td>m(^2)</td>
<td></td>
<td>6.00E-21</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>


2.3.1 In situ stress

An accurate assessment of the in situ stress conditions is important for the analysis of underground excavations. These boundary conditions are particularly significant when the engineering objective is to understand the rockmass response to excavation. In general, it is difficult to determine the stress field in an argillaceous formation with a pronounced bedding anisotropy.

A significant number of measurements of the in situ stress have been made using different procedures, such as borehole slotter, undercoring and hydraulic fracturing (Martin, 2003). Geological observations (e.g. borehole breakouts may indicate the direction of the minor principal stress) and back-analysis of instrumented excavations provide a valuable insight of the in situ stress. In Figure 2-12a, a stereographic projection shows that the gallery is subparallel to the bedding strike direction. It appears that the most reliable data are provided by the undercoring method, yielding the estimation presented in Error! Reference source not found..

<table>
<thead>
<tr>
<th>Principal stress</th>
<th>Orientation</th>
<th>Trend (°)</th>
<th>Plunge (°)</th>
<th>Units</th>
<th>MB test</th>
<th>Martin &amp; Lanyon, 2003</th>
<th>Corkum &amp; Martin, 2007</th>
</tr>
</thead>
<tbody>
<tr>
<td>Major, $\sigma_1$</td>
<td>Subvertical</td>
<td>210</td>
<td>70</td>
<td>MPa</td>
<td>7</td>
<td>6.5</td>
<td>6.5</td>
</tr>
<tr>
<td>Intermediate, $\sigma_2$</td>
<td>NW-SE</td>
<td>320</td>
<td>10</td>
<td>MPa</td>
<td>4</td>
<td>4</td>
<td>4</td>
</tr>
<tr>
<td>Minor, $\sigma_3$</td>
<td>NE-SW</td>
<td>50</td>
<td>15</td>
<td>-</td>
<td>3</td>
<td>0.6</td>
<td>2.2</td>
</tr>
</tbody>
</table>

The maximum principal stress, $\sigma_1$, which corresponds approximately with the overburden weight is subvertical and oblique to both the bedding and tectonic fracture planes (Figure 2-12). The greatest uncertainty is in the estimate of $\sigma_3$; the low range value of 0.6 MPa, derived from analysis of undercoring in 2003, was below the best estimate of pore pressure at the laboratory. In 2007 Corkum & Martin provided a revised estimate of $\sigma_3$, with a range of 2-3 MPa.

2.4 KEY EXPERIMENTS IN MONT TERRI

All experiments at Mont Terri can be assigned to the following three categories: (1) process and mechanism understanding in undisturbed argillaceous formations, (2) experiments related to excavation- and repository-induced perturbations and (3)
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experiments related to repository performance during the operational and post-closure phases.

Figure 2-13 Localization of some important experiments carried out up to 2012 (modified from Mont Terri Project, 2012).

Some important experiments related to excavation- and repository-induced perturbations (Figure 2-13) are:
- HG-A experiment (2005-2009): estimation of the extension of the damaged zone induced by the excavation works and its evolution due to the gas injection
- FE (Full-scale Emplacement Demonstration): 1:1 emplacement experiment for investigating the near-tunnel environment
- HE-E (Heater Experiment): Behaviour of the engineered barriers under the influence of heat
- MB (Mine-by Test): Deformation and hydro-mechanical effects during the excavation of tunnels and galleries
- SE-H (Self-sealing in Combination with Heat): Self-sealing of fissures in the excavation damaged zone, taking into account thermal influences
- ED-B experiment (evolution of the excavation disturbed zone around a new gallery) evolution of excavation disturbed zone (EDZ)

These experiments aims at reproducing the storage conditions expected in a disposal facility. High-level radioactive waste has to be isolated from the biosphere for extremely long periods in deep geological formation. Especially in the near field of the waste container, the host rock is affected by the excavation and the heat production of the waste. Thermal, hydraulic, mechanical and chemical (THMC) processes have to be expected. These processes could change the properties of the host rock significantly. Furthermore, the complex interaction makes it nearly impossible to treat the processes
separately. In order to evaluate the impact, coupled numerical modelling of the processes can be a great support for safety assessment. The related physical problems in the near- as well as in the far-field of the repository are characterised by the appearance of manifold interacting processes namely mechanical, hydraulic, thermal, chemical and biological ones which have a varying impact on the problem (Ziefle, 2008).

In coupled hydro-mechanical processes there are different interacting phenomena which have to be taken in account according to the objective of the simulation (Figure 2-14). The porosity will be changed depending on the volumetric strain caused by mechanical deformation and swelling or shrinking of the material. The volumetric strain caused by deformation is assumed to affect the pore space, not the size of particles. Consequently, an expansion will lead to an increase of pore space.

![Figure 2-14 Hydraulic and mechanical processes in clay materials (Ziefle, 2008).](image)

Stability issues associated with underground excavations in brittle rock masses can be grouped into three classes:

1. Structurally controlled gravity-driven processes leading to wedge-type failures

2. Stress-induced failure causing plastic yielding in weak rocks and extensile brittle fracturing in hard rocks. This process often leads to the formation of borehole breakouts in small diameter openings


Diederichs sets out a framework for stress-controlled brittle failure of tunnels identifying five behaviours: no damage, distributed damage, shear failure, spalling and unraveling as show in Figure 2-15. Compressive failures is associated with either: (i) spalling and unraveling at low confinement where microcracks propagate creating microfracturing, or (ii) macroscopic shear failure at higher confining stress, facilitate by microcrack interaction.

**Error! Reference source not found.** relates the brittle failure mode of Diederichs (2207) in Figure 2-15 to the definitions of the EDZ and HDZ. Note that the EdZ may be defined in terms other than stress change (e.g. temperature, chemistry or pore pressure change).
Figure 2-15 Schematic of Failure Envelope for brittle failure. Four zones of distinct rock mass failure mechanisms are shown: no damage, shear failure, spalling and unravelling (M.S., 2007)

The stress redistribution created by the excavated tunnel is the key cause of the EDZ, giving rise to tension, compression, and shear or deviatoric stresses in different parts of the rock around the opening. The responses of the different rock types to these stresses depend on their properties and structure. For indurated clays, owing to their bedding structure, the responses are different in the side walls compared to the ceiling and floor. Above the ceiling and below the floor there is a predominant opening of bedding planes, while on the two walls, layers of vertical fractures are formed. The EDZ is generally about one drift radius thick, with permeability increased by as much as six orders of magnitude.

Four distinct modes of damage have been observed during the construction of the Mont Terri facility:

1. Extensile brittle fracturing as multiple discrete fractures.
2. Stress-induced borehole breakouts when the tangential stress on the boundary of an opening exceeds the yield strength of the rock.
3. Slip along bedding planes if shear stress is exceeded, usually in the roof.
4. Swelling/softening when the moisture content of the Opalinus Clay is allowed to increase above its in-situ water content.

Time dependent evolution of the EDZ after excavation can be due to:
- creep, both in compression and shear
- relaxation of failed material
- pore pressure equilibration in low permeability rock
- development of partially saturated zones and cyclic saturation/desaturation and consequent loss of strength
- long-term changes due to the operation of the repository, including thermal stresses; chemical interaction with the Engineered Barrier System (EBS); and opening/closure of other excavations causing changes in stress and pore pressure.

Of particular interest in low permeability sedimentary rocks are the effects of pore pressure equilibration and the development of a partially saturated zone. In low permeability rocks strain changes couple strongly to pore pressure due to the low hydraulic diffusivity resulting in effectively “undrained” compression of the rock mass. The deformation due to excavation therefore results in significant pore pressure changes over several excavation radii. Pore pressure equilibration will occur over characteristic times $d^2/\eta$, where $d$ is the excavation diameter (m) and $\eta$ is the hydraulic diffusivity ($m^2/s$). Martin and Lanyon (2003a) suggest that the hydraulic diffusivity of the Opalinus Clay at Mont Terri is between $10^{-8}$ and $10^{-7} m^2/s$ resulting in characteristic times for pressure equilibration of ~10 years.

2.4.1 HE-D experiment

The HE-D in situ experiment is a heater experiment performed by ANDRA from 2003 to 2007 to investigate the host rock behaviour in case of directly being heated without any buffer material between heater and host rock (Wileveau, 2005). A general layout of the experiment is shown in Figure 2-16.A. The borehole is parallel to the strike of bedding, i.e. it has a trend of 240°. Here the bedding planes have an average dip of 41° to southeast, since it is located closer to the anticline.

Two heaters were emplaced in a 30 cm diameter horizontal borehole (D0), drilled from the main laboratory tunnel. Prior to the heater borehole drilling, the following instrumentation was installed: (i) 6 temperature sensors along boreholes D1 and D2; (ii) temperatures and pore pressures were measured at the same point in borehole D3 and in boreholes D7 to D17; (iii) a sliding micrometre tubing was installed in boreholes D4 and D5 to measure incremental deformations at 1m intervals (Gens, 2007). Two heating stages (650 W and 1950 W) were applied.

The main objectives of the test were:

- To determine in-situ the thermal conductivity of Opalinus Clay (OPA).
- To observe the generation and evolution of excess pore water pressure associated with the thermal load (analyse the impact of heat on the hydro-mechanical behaviour of OPA).
- To observe the rock mass deformation related to heating.

The instrumentation installation was carried out before having the heater borehole drilled, in order to record the hydro-mechanical effects. Pore water pressure response to the drilling of this borehole was observed. These measurements were used in this work to investigate the pore water pressure response to a small circular opening.
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2.4.2 Mine-by experiments

Two mine-by experiments were carried out in the Mont Terri URL: ED-B tunnel in 1997-1998 and MB niche in 2008.

2.4.2.1 ED-B TUNNEL

In 1997 an instrumented mine-by test tunnel known as the ED-B (Excavation Disturbed Zone-B) tunnel was excavated. It consisted of a 35 m long, 3.6 m diameter circular opening, excavated full face using a road header from the OP niche in the Reconnaissance Gallery (Figure 2-17.A).

The rockmass was instrumented prior to tunnel driving: (i) three piezometers located in boreholes B1-B3; (ii) one multipoint extensometer in borehole B5; (iii) three sliding micrometers/horizontal inclinometers in boreholes B6-B8; (iv) near-field deformations were measured using three sets of five-point convergence arrays in the ED-B tunnel 1-2 m behind the excavation face (Figure 2-17.B).

The main objectives of the test were:

- To observe the hydro-mechanical response of the OPA to unloading and the formation of an excavation damaged zone (EDZ).

Figure 2-16 A) Localization of the HE-D experiment. B) Experimental layout of the HE-D experiment (Wileveau, 2005).

Figure 2-17 A) Situation of the ED-B tunnel. B) Plan view of instrumentation for ED-B mine-by test (Corkum, 2007).
• To compare the short-term deformations and pore pressure response of the ED-B mine-by test with numerical simulations, in order to calibrate a stress-dependent modulus (SDM) model

Pore water pressure and deformations response to the excavation of the ED-B tunnel were observed. These measurements were used in this work to investigate the pore water pressure response to a circular opening.

2.4.2.2 MB Test

The MB mine-by test performed in 2008 involved tunnelling through a pre-instrumented region at full tunnel scale. Niche MB was excavated from Gallery 08 by successive blastings until reaching a distance of 26.6 m. The niche has an average diameter of 4.5 m (Figure 2-18.A). The orientation of the niche follows the bedding strike, which amounts 60°. Here the bedding planes have an average dip of 33° to southeast (Krug, 2010).

The sensors around the niche (Figure 2-18.B-C) were installed to measure mainly the deformation and pore pressure evolution in the near field: (i) two multi-packer systems with 5 piezometers each; (ii) two inclinometer chains with 10 segments for the measurement of angular strains; (iii) two magnetic extensometers with 20 points for the observation of axial deformations around the niche; (iv) one reverse head 10-point extensometer along the axis of Niche 2, which is shortened during the excavation process.

The main objectives of the test were:
- To evaluate the excavation-generated pressure, stress, displacement, and rock damage response in the OPA
- To understand the hydro-mechanical (HM) coupled behaviour of OPA around the excavated niche
- To observe the hydro-mechanical response of the OPA to unloading and the formation of an excavation damaged zone (EDZ).

The pore water pressure and deformation measurements were used in this work to investigate the pore water pressure response to a circular opening.
3. THEORETICAL FRAMEWORK

The aim of this section is to describe a theoretical formulation that allows describing the coupled hydro-mechanical (HM) processes that take place during the excavation of an opening in a saturated porous medium using a multi-phase, multi-species approach. Before a numerical simulation can be applied, a physical model conception is needed. The presence of a freely moving fluid in a porous rock modifies its mechanical response. Two mechanisms play a key role in this interaction between the interstitial fluid and the porous rock: (i) an increase of pore pressure induces a dilation of the rock, and (ii) compression of the rock causes a rise of pore pressure in undrained conditions. A meaningful modelling of the excavation induced processes can only be done by coupling different models. The following physical phenomena have been considered:

Water flow: i) Darcy flow, ii) vapour diffusion and iii) phase changes.
Mechanical behaviour: i) Elastic deformation ii) Visco-plasticity

Local equilibrium is assumed throughout. To construct a well-posed mathematical system for the description of the stress, pore pressure, flux and deformation in the porous medium, equations based on mass and momentum conservation principles need to be introduced. Together with the constitutive laws and equilibrium restrictions, these equations constitute the governing equations.

3.1 BALANCE EQUATIONS

Balance or conservation principles express fundamental physical observations concerning the interaction of a continuum medium and the environment. They reflect the balance of the most important physical measures for the body of interest. Within the classical theory of mechanics, this implies the balance of mass, the balance of linear and angular momentum, the balance of energy and the balance of entropy.

A general balance equation contains three terms: the change with respect to time of a property of the porous material \( \psi \), the divergence of the flux of this property and the rate of production/decay of the property, given by:

\[
\frac{\partial}{\partial t} \left( \theta_{\psi} \rho \right) + \nabla \left( j_{\psi} \right) - f^w = 0
\]

where \( \theta_{\psi} \) is the mass content of \( \psi \) per unit volume, \( j_{\psi} \) is the total mass flux of \( \psi \) and \( f^w \) is the rate of production/decay of \( \psi \) per unit volume. Using a compositional approach, the volumetric mass of a species in a phase, \( \omega_{\psi} \), is the product of the mass fraction of that species, \( \theta_{\psi} \), and the bulk density of the phase, \( \rho \).

The total mass flux of a species in a phase, \( j_{\psi} \), can be decomposed into an advective flux caused by fluid motion and nonadvective flux, and advective flux caused by solid motion, in order to simplify the equations.

3.1.1 Mass balance of water

This balance equation introduces the state variable \( P_l \), liquid pressure, which is linked with the Darcy flux \( q_l \). As water is present in liquid and gas phases, the total mass balance of water is expressed as:

\[
\frac{\partial}{\partial t} \left( \theta_{w} S_l \phi + \theta_{g} S_g \phi \right) + \nabla \left( j_{w} + j_{g} \right) = f^w
\]
where $\theta_l^w$ and $\theta_g^w$ are the mass content of water per unit volume of liquid and gas, respectively. $S_l$ and $S_g$ is the degree of saturation of liquid and gaseous phases, respectively and $\phi$ is the porosity. $j_l^w$ and $j_g^w$ are the total water mass fluxes in the liquid and gas phases with respect to a fixed reference system and $f^w$ is an external supply of water per unit volume of medium.

The definition of material derivative:

$$\frac{D}{Dt}(\psi) = \frac{\partial}{\partial t} + \nabla (\psi)$$

leads to this expression of the water mass balance:

$$\phi \frac{D}{Dt}(\theta_l^w S_l + \theta_g^w S_g) + (\theta_l^w S_l + \theta_g^w S_g) \frac{D}{Dt}(1 - \phi) \nabla \cdot \frac{\partial u}{\partial t} + \nabla \cdot (j_l^w + j_g^w) = f^w$$

### 3.1.2 Mass balance of solid

The mass balance of solid in the medium is written in terms of porosity:

$$\frac{\partial}{\partial t} (\theta_s (1 - \phi)) + \nabla \cdot j_s = 0$$

where $\theta_s$ refers to the grain content per unit volume of soil and $j_s$ is the mass flux of solid.

### 3.1.3 Balance of momentum

If inertial terms are neglected, the balance of momentum for the porous medium reduces to the equilibrium equation for macroscopic total stresses:

$$\nabla \cdot \sigma + b = 0$$

where $\sigma$ is the stress tensor and $b$ is the vector of body forces. The compatibility equation provides the link between the basic variables, displacements, $u$, and strains, $\varepsilon$:

$$\varepsilon = \frac{1}{2} (\nabla u + \nabla u^T)$$

Combining the above equations with Equation(3.4), the mass balance of water can be rewritten as:

$$\phi \beta_w \frac{D}{Dt} P_w + (1 - \phi) \beta_s \frac{D}{Dt} P_s + \nabla \cdot \left( \frac{\partial u}{\partial t} \right) + \frac{\nabla \cdot (\rho_w \cdot q_l)}{\rho_w} = 0$$

This equation may be used to explain the pore water pressure response to several solicitations (excavation, drainage and ventilation).

### 3.2 Equilibrium Restrictions

Thanks to the compositional approach adopted, phase changes do not appear explicitly in the formulation. The assumption of local equilibrium implies that the species...
concentration in the various phases can be considered as dependant variables. Equilibrium restrictions are given for the concentration of water vapour in gas and of dissolved air in water.

3.2.1 Psychrometric law

The psychrometric law gives the variation of partial vapour pressure in the gas phase as a function of the curvature of the surface of the liquid phase and the temperature. The mass of water vapour per unit volume of gas is:

$$\theta^w = \left(\theta^w\right)_0 \cdot \exp \left( -\frac{(P_g - P_i) M_w}{RT \rho_l} \right)$$

(3.8)

where \(\left(\theta^w\right)_0\) is the vapour density in the gaseous phase in contact with a planar surface \((P_g - P_i = 0)\), \(M_w\) is the molecular mass of water \((0.018 \text{ kg/mol})\), \(R\) is the gas constant \((18.314 \text{ J/mol·K})\) and \(T\) is the temperature. The vapour concentration is in equilibrium with the liquid phase.

Vapour partial pressure is computed by means of the ideal gas law. This law provides the relationship between water vapour density \(\rho^w_g\) and partial water vapour pressure \(P^w_g\):

$$\rho^w_g = \frac{M_w P^w_g}{RT}$$

(3.9)

where \(M_w\) is the molecular mass of water \((0.018 \text{ kg/mol})\), \(R\) is the universal gas constant \((8.314 \text{ J/mol·K})\) and \(T\) is the temperature.

3.2.2 Henry’s law

The solubility of air in water is controlled by Henry’s law:

$$\omega^a_l = \frac{P_a M_a}{H M_w}$$

(3.11)

where \(\omega^a_l\) is the mass fraction of air in liquid, \(P_a\) is the partial pressure of air, \(M_a\) is the molecular mass of air \((0.02895 \text{ kg/mol})\) and \(H = 10 \text{ GPa}\) is Henry’s constant.

Mass fraction of water vapour is related to the relative humidity by the expression:

$$H_r = \frac{\theta^w}{\left(\theta^w\right)_0} 100 = \frac{p_v}{p_{v0}} 100$$

(3.10)

where \(p_v\) is the vapour pressure and \(p_{v0}\) is the vapour pressure for the saturated state.

3.3 Constitutive equations

3.3.1 Hydraulic constitutive models and phase properties
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In order to find the unknowns from the governing equations, the dependent variables have to be related to the unknowns by the following additional equations.

Generalised Darcy’s law is used to relate advective fluxes of fluid to phase pressure gradient:

$$\mathbf{q}_a = -K(\nabla P_a - \rho_a g) \quad (3.11)$$

where $K = k_{\alpha} / \mu_\alpha$ is the permeability tensor, $k_{\alpha}$ is the value of relative permeability that controls the variation of permeability in the unsaturated regime and $\mu_\alpha$ denotes the dynamic viscosity. In these expressions, $\alpha$ may stand for either $l$ or $g$ depending on whether liquid or gas flow is considered.

The intrinsic permeability $k$ depends on the pore structure of the porous medium by the Kozeny’s law:

$$k = k_0 \frac{\phi_0^3}{(1-\phi)^2} \left(1-\frac{\phi}{\phi_0}\right)^2 \quad (3.12)$$

where $\phi_0$ is the reference porosity and $k_0$ is the intrinsic permeability for matrix $\phi_0$.

When solid skeleton deforms, the volume of pores and its distribution can change. Intrinsic permeability is therefore a function of volumetric deformation, which is computed on the basis of the displacements.

The relative permeabilities of the liquid and gaseous phases are dependent on the degree of saturation according to:

$$k_{rl} = \sqrt{S_e} \left(1 - \left(1 - S_e^{1/\lambda}\right)^2\right)^2, \text{ where } S_e = \frac{S_l - S_{rl}}{S_{lu} - S_{rl}} \quad (3.15)$$

The volumetric fraction of pore volume occupied by the gaseous phase is related to the liquid phase by the following expression:

$$k_{rg} = 1 - k_{rl} \quad (3.13)$$

It is also necessary to define the retention curve of materials relating degree of saturation with suction ($s = P_l - P_t$) by the Van Genuchten model:

$$S_e = \frac{S_l - S_{rl}}{S_{lu} - S_{rl}} = \left(1 + \left(\frac{P_l - P_t}{P}ight) \frac{1}{\lambda_{0}}ight)^{-\lambda_{0}}, \text{ where } P = P_0 \frac{\sigma}{\sigma_0} \quad (3.14)$$

It is an empirical relationship that expresses the equilibrium between suction and capillary forces, i.e. surface tension, in the meniscus or pore space in soils. $S_e$ is the effective saturation, which varies between 0 and 1, $S_{rl}$ and $S_{lu}$ are the lower and upper limits respectively of the degree of saturation.

$P_0$ is a temperature dependent pressure related to the air entry pressure, $\sigma_0$ is the surface tension at temperature in which $P_0$ was measured and $\lambda$ is the shape function for the retention curve.

The liquid viscosity is related to the temperature by:

$$\mu_l = (\mu_l)_0 \cdot \exp\left(\gamma_T / T\right) \quad (3.15)$$
where $\gamma_T$ is a parameter which expresses the dependency of liquid viscosity with temperature and $(\mu_L)_0$ is the standard viscosity at 20ºC.

Water density in liquid phase $\rho_L^w$ is related with liquid pressure $P_L$ and temperature $T$ by the following relation:

$$\rho_L^w = (\rho_L^w)_0 \cdot \exp\left(\beta_w \left( P_L - (P_L)_0 \right) + \alpha_w T + \gamma \omega^h \right) \quad (3.16)$$

where $\rho_L^w_0$ is the reference density of water at a reference liquid pressure $P_{L,0}$, $\alpha_w$ and $\beta_w$ are the volumetric thermal expansion coefficient and compressibility of water, respectively, and $\gamma$ is the solute variation.

Fick’s law generally relates the nonadvective flux of species to the gradient of mass fraction through a tensor $D_{\alpha i}$ that takes into account molecular diffusion and hydromechanical dispersion:

$$i_{\alpha i} = -D_{\alpha i} \cdot \nabla \omega_{\alpha i} \quad (3.17)$$

The diffusion/dispersion tensor depends on porosity, degree of saturation and phase density:

$$D_{\alpha i} = \phi \rho_{\alpha i} S_{\alpha} \tau D_{m i} \quad (3.18)$$

where $\phi$ is porosity, $\rho_{\alpha i}$ is density, $S_{\alpha}$ is degree of saturation, $\tau$ is the tortuosity, $\omega$ is the mass fraction and $D_{m i}$ is the diffusion coefficient of species $i$ in phase $\alpha$.

### 3.3.2 Mechanical constitutive laws

In general, a complete theoretical solution must satisfy equilibrium, compatibility, the material constitutive behaviour and boundary conditions. The constitutive behaviour is a description of the stress-strain behaviour of the soil. It usually takes the form of a relationship between stresses and strains and therefore provides a link between equilibrium and compatibility:

$$d\sigma = [D]d\varepsilon \quad (3.19)$$

The non-linear response to stress changes of Opalinus Clay indicate that some form of visco-elastic or visco-plastic model should be adopted. In addition, the axial and radial strain response seem to be anisotropic. In this work the Opalinus Clay behaviour is modelled by a visco-elasto-plastic frictional model with a Van Eekelen criterion, which consists of a smoothed Mohr-Coulomb plasticity surface. The basic features of the model are:

1. An elastic domain described by the bulk modulus $K$ and shear modulus $G$
2. A Mohr-Coulomb yield function indicates the beginning of plastic deformation.
3. Rate dependency is introduced as a visco-plastic mechanism.
4. A softening law to describe strength degradation.

**Linear elasticity** is a simple approach for modelling a linear reversible stress-strain behaviour of a material. **Linear isotropic elasticity** is very useful as a first approach, but is not able to predict failure and the two modes of deformation are decoupled, i.e. changes in mean stress do not cause shear strains and changes in the deviatoric stress do not cause volume change.
Since an isotropic material is one that has point symmetry, only two independent elastic constants \((E, v)\) are necessary to represent the behaviour and the constitutive matrix becomes symmetrical:

*Linear anisotropic elasticity* applies to many soils because of the way in which they were originally deposited. They exhibit an axis of symmetry in the depositional direction, i.e. only five independent parameters are required \((E_v, E_H, v_H, v_v, G_H)\).

**Figure 3-1** Axes of symmetry in transversal isotropic soils.

**Elasto-plasticity** provides a good framework in which to formulate constitutive models that can realistically simulate real soil behaviour. Unlike elastic solids, in which the state of strain depends only on the final state of stress, the deformation that occurs in a plastic solid is determined by the complete history of the loading.

Elasto-plastic models assume that the principal directions of accumulated stress and incremental plastic strain coincide. They require (i) a *yield function*, which separates purely elastic from elasto-plastic behaviour; (ii) a *plastic potential* or flow rule which prescribes the direction of plastic straining and (iii) a set of *hardening/softening rules* which describe how the state parameters like strength vary with plastic strain. If the yield and plastic potential surfaces coincide, the model is said to be associated or to satisfy the normality condition.

The *yield function* (equivalent to the yield stress in a uniaxial situation) is defined in a multi-axial situation as a scalar function of the stress state \(\sigma\) and the state parameters \(h\) related to hardening/softening:

\[
F(\sigma, h) = 0
\]  

(3.20)

The *flow rule* specifies the direction of plastic straining at every stress state by:

\[
\Delta \varepsilon^p = \lambda \frac{\partial P}{\partial \sigma} 
\]  

(3.21)

where \(\Delta \varepsilon^p\) represents the six components of incremental plastic strain, \(P(\sigma, h) = 0\) is the plastic potential function and \(\lambda\) is a scalar multiplier. The outward vector normal to the plastic potential surface at the current stress state has components which provide the relative magnitudes of the plastic strain increment components, controlled by the scalar parameter \(\lambda\), which is dependent on the hardening/softening rule.

If the plastic potential function is assumed to be the same as the yield function, i.e. \(P(\sigma, h) = F(\sigma, h)\), the flow rule is said to be associated and the incremental plastic strain vector is then normal to the yield surface. Flow rules are of great importance in constitutive modelling because they govern dilatancy effects which in turn have a significant influence on volume changes and on strength.

The *hardening/softening rules* prescribe how the state parameters \(h\) vary with plastic straining, i.e. how the position and/or size of the yield surface changes with the stress...
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A strain hardening/softening rule relates the changes in size of the yield surface to the components of the accumulated plastic strain \( h = h(e^p) \).

The consistency condition, which implies that when the stress state is on the yield surface it must satisfy the yield function \( F(\sigma, h) = 0 \), enables to obtain the elasto-plastic constitutive matrix:

\[
\{\Delta \sigma\} = [D^{pp}] \{\Delta e\}, \quad \text{where } [D^{pp}] = \left[D^e\right] - \frac{[D^e] \left( \frac{\partial F^T}{\partial \sigma} \right)^T [D^e]}{H + H_{\epsilon_{cr}}}
\]

(3.22)

where \( H = -\frac{1}{\lambda} \left( \frac{\partial F}{\partial h} \right)^T \Delta h \) is defined as the plastic modulus, which form depends on the type of plasticity. In perfect plasticity the state parameters \( h = h(e^p, \ldots) \) are constant and consequently \( H = 0 \). In strain hardening or softening plasticity the state parameters \( h \) are related to the accumulated plastic strains \( e^p \), that is:

\[
H = -\frac{\partial F}{\partial h} \frac{\partial P}{\partial e^p} \frac{\partial e^p}{\partial \sigma}
\]

(3.23)

The elasto-plastic constitutive matrix provides a relationship between incremental stresses and incremental strains in the form of:

\[
\{d \sigma\} = [D^{pp}] \{d e\}
\]

(3.24)

The incremental total strains can be split into elastic and plastic components:

\[
\Delta e = \Delta e^e + \Delta e^p = [D^{pp}]^{-1} \Delta \sigma + \lambda \frac{\partial P}{\partial \sigma}
\]

(3.25)

To implement an elasto-plastic model first the elasto-plastic matrix \([D^{pp}]\) has to be calculated, and then the state parameters \( h = h(e^p, \ldots) \) have to updated for the new yield surface.

Visco-plasticity allows a rate dependent stress-strain relation. Argillaceous rocks can undergo substantial volumetric creep when mean effective stress is raised. Creep is time-dependent deformation of a material when subjected to sustained loading. In porous materials it is possible to distinguish two main types: (a) creep in shear governed by the deviatoric component of the stress tensor and (b) volumetric creep governed by the spherical component of the effective stress tensor. Consolidation of clay-rich materials is a time dependent change in volume and is considered to have a hydrodynamic component (primary consolidation) and a creep component (secondary consolidation). Since creep strains are largely non-recoverable on unloading, they are best described as viscoplastic strains.

In this work, a visco-plastic framework is used. This implies that the consistency condition is not fulfilled at all times, i.e. stress states can be outside the yield function. Inside the yield function \( (F < 0) \) elastic deformations occur, while outside the yield function \( (F > 0) \) deformations are viscoplastic. On the yield function \( (F = 0) \) deformations are plastic, deformation rate tends to be zero. Equipotential deformation
surfaces are comprised between a plastic loading surface \( F = 0 \) and a loading surface related to a strain rate infinitely fast \( (F = \infty) \). The total strain rate \( \dot{\varepsilon} \) is divided into an instantaneous reversible (elastic) part \( \dot{\varepsilon}^e \) and an irreversible \( \dot{\varepsilon}^{vp} \) part:

\[
\dot{\varepsilon} = \dot{\varepsilon}^e + \dot{\varepsilon}^{vp}
\]  

(3.29)

A viscoplastic flow rule is defined:

\[
\dot{\varepsilon}^{vp} = \eta \left\langle \Phi(F) \right\rangle \frac{\partial G}{\partial \sigma}
\]  

(3.30)

where \( \eta \) is the viscosity, \( G \) is the viscoplastic potential, \( \Phi \) is the flow rule, and \( F \) corresponds to the static yield function. The brackets defined by Macauly, \( \left\langle \right\rangle \), control the yield function in the following way:

\[
\begin{align*}
\Phi(F) = 0 & \quad \text{if } \Phi(F) < 0 \\
\Phi(F) = \Phi(F) & \quad \text{if } \Phi(F) > 0
\end{align*}
\]  

(3.31)

Function \( F \) depends on the second invariant of stress tensor \( q \), by assuming a von Mises loading surface. The viscoplastic strain rate of Perzyna is:

\[
\dot{\varepsilon}^{vp} = \int_0^t \left( \frac{2}{3} \dot{\varepsilon}^{vp} \cdot \dot{\varepsilon}^{vp} \right)^{1/2} d\tau
\]  

(3.32)

The assumption of isotropic hardening is invoked. The constitutive law can be defined as being associated, in which case the viscoplastic potential, \( G \), corresponds to the second invariant of the stress tensor \( q \) :

\[
G(\sigma) = q = \sqrt{\frac{3}{2} s_y s_y} = \sqrt{3 J_2}
\]  

(3.33)

This assumption implies that viscoplastic strains are developed without any volume changes. This way, the viscoplastic flow rule can be expressed as:

\[
\dot{\varepsilon}^{vp} = \eta \left\langle \Phi \left( \frac{q}{\kappa} - 1 \right) \right\rangle \frac{\partial G}{\partial \sigma}
\]  

(3.34)

In the model for argillaceous model adopted the plastic multiplier \( \lambda^p \) is expressed as a function of the distance between the current stress point and the inviscid plastic locus:

\[
d\lambda^p = \frac{dt}{\eta} \left\langle F^p \right\rangle
\]  

(3.35)

where \( dt \) is the time increment, \( \eta \) is the clay matrix viscosity and \( \left\langle \right\rangle \) are the Macauley brackets. Inviscid plastic locus takes the form:

\[
\vec{F}^p = F^p - \eta \frac{dt}{d\lambda^p} \leq 0
\]  

(3.36)

The plastic multiplier is not a material constant but a scalar \( \lambda^p = \lambda^p \)

3.3.3 Elasto-plastic Mohr-Coulomb model
The Mohr-Coulomb failure criterion is a straight line tangent to the failure circles from several tests performed with different initial effective stresses, and can be expressed as:

$$\tau_f = c' + \sigma_{nf}' \tan \phi'$$  \hspace{1cm} (3.26)

where $\tau_f$ and $\sigma_{nf}'$ are the shear and normal effective stresses on the failure plane, and the cohesion, $c'$, and angle of shearing resistance, $\sigma_{nf}'$, are material parameters. Using the Mohr’s circle of stress (Figure 3-2) and noting that $\sigma'_i = \sigma'_v$ and $\sigma'_j = \sigma'_h$, Equation (3.26) can be rewritten as:

$$\sigma'_i - \sigma'_j = 2c' \cos \phi' + \left( \sigma'_i - \sigma'_j \right) \sin \phi'$$  \hspace{1cm} (3.27)

This Mohr-Coulomb failure criterion is adopted as the yield function:

$$F(\sigma, h) = \sigma'_i - \sigma'_j - 2c' \cos \phi' + \left( \sigma'_i - \sigma'_j \right) \sin \phi'$$  \hspace{1cm} (3.28)

It is more convenient to rewrite this equation in terms of stress invariants $p'$, $J$ and $\theta$. Therefore, the yield function can be expressed as:

$$F(\sigma, h) = J - (p' + a) g(\theta) = 0, \text{ where } g(\theta) = \frac{\sin \phi'}{\cos \theta + \sin \theta \sin \phi'}$$  \hspace{1cm} (3.29)

Unlike the Tresca or von Mises failure criteria, the Mohr Coulomb criterion depends on the mean effective stress, a very important feature for soils.

Figure 3-2 Mohr’s circle of effective stress.

In principal stress space the yield function plots as an irregular hexagonal cone. If an associated flow rule, with $P(\sigma, h) = F(\sigma, h)$ is adopted, the plastic strain increment vector is inclined at an angle $\phi'$ to the vertical and indicates negative, i.e. tensile plastic...
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strains (Figure 3-2). This in turn results in a dilatant plastic volumetric strain, with an angle of dilation $\psi$ equal to the angle of shearing resistance, $\phi'$. This is an unrealistic approach, because the magnitude of dilation will be much larger than that observed in real soils and the soil will dilate for ever once the soil yields. In order to obtain smaller plastic volumetric strains, a non-associated flow rule, where the plastic potential function is assumed to take a similar form to that of the yield surface is adopted:

$$P(\sigma, h) = J - (p' + a_{pp}) g_{pp}(\theta) = 0,$$

where

$$g_{pp}(\theta) = \frac{\sin \psi}{\cos \theta + \frac{\sin \theta \sin \psi}{\sqrt{3}}}$$

and $a_{pp}$ is the distance of the apex of the plastic potential cone from the origin of principal stress space. Noting that the plastic potential must also pass through the current stress stated which, because the soil is plastic, must be on the yield surface, the situation shown in Figure 3-3 is obtained. While the yield surface is fixed in $p' - J - \theta$ space, the plastic potential surface moves as to pass through the current stress state. This is acceptable as only the derivatives of the plastic potential with respect to the stress component are needed to form the elasto-plastic constitutive matrix.

![Figure 3-3](image)

**Figure 3-3** Relationship between yield and plastic potential functions (modified after Potts, 1999).

By prescribing the angle of dilation, $\psi$, the predicted plastic volumetric strains can be controlled.
4. STRESS-STRAIN ANALYSIS OF AN EXCAVATION

Stress analyses provide insight into the changes in pre-existing stress equilibrium caused by an opening, it interprets the performance of an opening in terms of stress concentrations and associated deformations. Once the excavation is made, the stresses in the vicinity of the opening are redistributed, which can overstress parts of the rock mass and make it yield.

4.1 ELASTIC CONSOLIDATION AROUND A DEEP CIRCULAR TUNNEL

When an underground excavation is made in a rock mass, the stresses which previously existed in the rock are disturbed, and new stresses are induced in the rock in the immediate vicinity of the opening (Hoek, 1982). The major and minor principal stress trajectories in the material surrounding a circular hole in a uniaxially stressed elastic plate depart significantly from being vertical and horizontal in the vicinity of the opening which deflects the stress trajectories.

![Figure 4-1](image)

Figure 4-1: In situ stresses in the MB and HE-D experiments at Mont Terri (for simplicity, the σ2 stress axis is just presented horizontally and parallel to the tunnel axis, although it plunges with 7° depression towards NE; the σ1 stress axis is also presented vertically, but plunges in reality with 70° towards SW). Stress trajectories around a circular excavation (Hoek, 1982).

In the roof and floor of a circular excavation subjected to uniaxial compressive stress tensile stress develops.

The tangential stresses are highest in the tunnel walls exceeding the peak strength of the Opalinus Clay (triaxial laboratory tests suggest a Mohr-Coulomb envelope with an effective cohesion c’ of 2MPa and an effective angle of internal friction of 25°), leading to the EDZ fractures such as steeply inclined unloading joints and shear fractures on both side walls of the excavation.

In order to calculate the stresses, strains and displacements induced around excavations in elastic materials, a set of equilibrium and displacement compatibility equations have to be solved for given boundary conditions and constitutive equations for the material.

4.2 CONVENTIONAL ANALYSIS METHODS

Traditionally geotechnical design has been carried out using simplified analyses or empirical approaches. The introduction of inexpensive computer hardware and software has resulted in considerable advances in the analysis and design of geotechnical structures.
At present, there are many different methods of calculation available for analysing geotechnical structures (Potts, 1999). As part of the design process, analysis simulates real behaviour to understand problems better and to provide estimates of the movements, stresses, etc.

In general, a complete theoretical solution must satisfy equilibrium, compatibility, the material constitutive behaviour and boundary conditions. The constitutive behaviour is a description of the stress-strain behaviour of the soil. It usually takes the form of a relationship between stresses and strains and therefore provides a link between equilibrium and compatibility:

\[
\Delta \sigma = [D] \Delta \varepsilon
\]  

(3.31)

The constitutive behaviour can either be expressed in terms of total or effective stresses. If specified in terms of effective stresses, the principle of effective stress \((\sigma = \sigma' + \sigma_f)\) may be invoked to obtain total stresses required for use with the equilibrium equations:

\[
\Delta \sigma = [D'] \Delta \varepsilon ; \Delta \sigma_f = [D_f] \Delta \varepsilon \Rightarrow \Delta \sigma = \left( [D'] [D_f] \right) \Delta \varepsilon
\]  

(3.32)

where \([D_f]\) is a constitutive relationship relation the change in pore fluid pressure to the change in strain. For undrained behaviour, the change in pore fluid pressure is related to the volumetric strain (which is small) via the bulk compressibility of the pore fluid (which is large):

\[
\Delta \varepsilon_v = n \Delta \rho_f / K_f
\]  

(3.33)

Usually it is not possible to obtain closed form analytical solutions incorporating realistic constitutive models of soil behaviour which satisfy all four fundamental requirements. The analytical solutions available (e.g. Limit equilibrium, Stress fields and Limit analysis) fail to satisfy at least one of the fundamental requirements.

Simple numerical methods can provide information on local stability and on wall movements and structural forces under working load conditions. They are therefore an improvement over the simpler analytical methods. However, they do not provide information on overall stability or on movements in the adjacent soil and the effects on adjacent structures or services.

Full numerical analysis can provide information on all design requirements. In many respects they provide the ultimate method of analysis, satisfying all the fundamental requirements. However, they require large amounts of computing resources and a deep understanding of the processes involved.

**Analytical solutions**

**Closed form solutions:** an exact theoretical solution can be obtained for a particular geotechnical structure, establishing a realistic constitutive model for material behaviour, identifying the boundary conditions and combining these with the equations of equilibrium and compatibility. However, complete analytical solutions to realistic geotechnical problems are not usually possible, being soil a highly complex multi-phase material which behaves nonlinearly when loaded.
Solutions are usually obtained either assuming an isotropic linear elastic behaviour of the soil, or considering the problem one dimensional if there are sufficient geometric symmetries.

**Simple methods:** the constraints on satisfying the basic solution requirements are relaxed, but mathematics is still used to obtain an approximate analytical solution.

- *Limit equilibrium:* an arbitrary failure surface is assumed and equilibrium conditions are considered for the failing soil mass, assuming that the failure criterion holds everywhere along the failure surface.

- *Stress field solution:* the soil is assumed to be at the point of failure everywhere and a solution is obtained by combining the failure criterion with the equilibrium equations.

- *Limit analysis:* several assumptions are made to allow a unique failure condition. The bound theorems (safe or ‘Lower bound’ theorem and unsafe or ‘Upper bound’ theorem) enable estimates of the collapse loads, which occurs at failure, to be obtained.

**Numerical analysis**

**Beam-spring approach:** this approach is used to investigate soil-structure interaction, e.g. the behaviour of loaded piles, raft foundations or tunnel linings. The soil behaviour is either approximated by a set of unconnected vertical and horizontal springs, or by a set of linear elastic interaction factors. Only a single structure can be accommodated in the analysis.

**Full numerical analysis:** all requirements of a theoretical solution are considered, but the solution is obtained in an approximate manner. This approach offers more realistic solutions. Approaches based on finite difference (FD) or finite elements (FE) methods are those most widely used. Their ability to accurately reflect field conditions essentially depend on (i) the ability of the constitutive model to represent real soil behaviour and (ii) correctness of the boundary conditions imposed.

The effect of time on the development of pore water pressures can be simulated by including coupled consolidation.

Simple methods may be appropriate when they have been calibrated against field observation. However, for complex field situations, these simple methods are less reliable.

**Table 4-1** Basic solution requirements satisfied by the various methods of analysis (Potts, 1999).
### Deep excavation in an anisotropic argillaceous rock

Stress-strain analysis of an excavation

<table>
<thead>
<tr>
<th>analysis</th>
<th>Upper</th>
<th>X</th>
<th>with associated flow rule</th>
<th>X</th>
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</thead>
<tbody>
<tr>
<td>Beam-spring approaches</td>
<td>X</td>
<td>X</td>
<td>Springs or elastic interaction factors</td>
<td>X</td>
</tr>
<tr>
<td>Full numerical analysis</td>
<td>X</td>
<td>X</td>
<td>Any</td>
<td>X</td>
</tr>
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</table>

#### 4.2.1 Tunnel excavation analysis methods

The ‘gap’ method (Rowe et al., 1983)
A predefined void is introduced into the finite element mesh which represents the total ground loss expected. The void locates the soil boundary prior to excavation, by resting the invert of the tunnel on the underlying soil and prescribing the gap parameter at the crown (vertical distance between the crown and the initial position).

The ‘convergence-confinement’ method (Panet and Guenot, 1982)
The proportion of unloading before lining construction is prescribed, so volume loss is a predicted value. An internal force vector \( (1-\lambda)F_i \) is applied at the nodes on the tunnel boundary \( (F_i) \) being equivalent to initial soil stresses \( (\sigma_i) \). \( \lambda \) is initially equal to 0 and is then progressively increased to 1 to model the excavation process.

The ‘progressive softening’ method (Swoboda, 1979)
Method developed for the modelling of NATM (or sprayed concrete) tunnelling.

The ’volume loss control’ method
Method similar to the convergence-confinement method, but instead of prescribing the proportion of unloading prior to lining construction, the analyst prescribes the volume loss that will result on completion of excavation. Comparison between conventional analysis methods and FEM

The application of numerical methods to tunnelling is important because non-numerical methods are characteristically uncoupled, i.e. the loads are determined by one technique (usually an elastic solution), and movements by another (usually empirical), the two not being linked together. The finite element method can:

- Simulate the construction sequence
- Deal with complex ground conditions
- Model realistic soil behavior
- Handle complex hydraulic conditions
- Deal with ground treatment
- Account for adjacent services and structures
- Simulate intermediate and long term conditions
- Deal with multiple tunnels
5. ANALYSIS OF HM OBSERVATIONS AROUND CIRCULAR OPENINGS IN ARGILLACEOUS ROCKS

The numerical analysis has been carried out using the finite element code CODE_BRIGHT that allows the performance of coupled termo-hydro-mechanical analysis (Olivella, 1996). The code is able to take into account all the physical phenomena and constitutive laws discussed in the previous sections.

In order to analyse the coupled hydro-mechanical effects of an excavation various in situ tests have been performed at the Mont Terri URL. Specifically, the following experiments will be considered:

- The instrumented drilling of a 30 cm diameter borehole for a heater test in the HE-D experiment (see Section 2.4.1)
- A mine-by test 4.5 m diameter tunnel performed in the MB niche (see Section 2.4.2.2)

Interpretation of the patterns of behaviour exhibited by the test observations will be aided by the results of a finite element simulation of the experiment. A specific feature of the mine-by test experiments is related to the step-by-step advance of the excavation front, which causes a progressive release of the initial radial in situ stresses and pore pressure. Moreover, the Opalinus Clay shows a strong transverse isotropy in the direction of the bedding planes. Therefore, a full three-dimensional analysis would be the most convenient solution, since it can easily incorporate both the effect of the rock mass anisotropy, and the effect of the advancing excavation front. However, it implies large computer resources in order to catch the sharp gradients caused by excavation as well as the long term evolution. Instead, a bidimensional model has been implemented in two steps: first, the calibration of the hydro-mechanical parameters has been done on the heater HE-D experiment, since there are sensors located outside and inside the EDZ. In order to study the influence of anisotropy on the hydraulic-mechanically coupled behaviour, a sensitivity analysis is carried out.

Then, in a second step, these parameters have been used to simulate the excavation response in the MB mine-by test. Several years of data on excavation disturbed zone (EDZ) behaviour are available for modelling and interpretation in the MB experiment.

5.1 HE-D

5.1.1 Discretization

The aim of this analysis is the interpretation of the pore pressure change induced by the drillage of the heater borehole and to obtain the field parameters by back-calculation to analyse the pore pressure change and deformation in the MB test. The drilling of a heater borehole in the context of the HE-D experiment has been numerically simulated by use of a 2D plane strain model. The domain modelled and the finite element mesh used are depicted in Error! Reference source not found.. The distance to the right boundary is 8 m, which corresponds to the distance from the borehole axis to the niche MI (for a general outlay of the experiment see Figure 2-16). The top, left and bottom boundaries were set at a distance of 30 m from the edge of the tunnel: 10 times the excavation diameter (0.30 m). A sensitivity analysis has been carried out to determine
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the input parameters for the simulation due to the great variability of parameters (see Section 2.3.).

The reference initial and boundary conditions are described in Table 5-1 and Table 5-2 respectively. A constant stress boundary and null pore pressure is assumed on the surface that corresponds to the niche MI. A condition of constrained lateral displacements is adopted for the opposite lateral boundary, as well as on the bottom boundary, both considered distant enough to not affect the processes simulated on the borehole. Therefore only a refined mesh is considered around the borehole (Error! Reference source not found.). The initial liquid pressure has been set to the field-measured value of 0.9 MPa, lower than the typical 2.2 MPa value of the massif because of the influence of the excavation of the MI niche. Values of 7 MPa, 5 MPa and 3 MPa as major, intermediate and minor initial principal stresses have been assigned to the domain. To simulate the excavation, the pore pressures on the borehole boundary are set to atmospheric. Before the circular hole in the center was excavated, the system was in equilibrium.

![Geometry of the HE-D experiment finite element model](image)

**Figure 5-1** Geometry of the HE-D experiment finite element model (10787 nodes; 10726 elements) and situation of the boundary conditions described in Table 5-2.

<table>
<thead>
<tr>
<th>Initial condition</th>
<th>Values applied at the beginning of the simulation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Porosity</td>
<td>$\phi = 0.15$</td>
</tr>
<tr>
<td>Stress</td>
<td>$\sigma_3 = 4\text{MPa}$ $\sigma_2 = 7\text{MPa}$</td>
</tr>
<tr>
<td>Liquid pressure</td>
<td>$p_c = 0.9\text{MPa}$</td>
</tr>
</tbody>
</table>

**Table 5-1** Initial conditions applied at the beginning of the simulation.

<table>
<thead>
<tr>
<th>Boundary condition</th>
<th>Table 5-2 Boundary conditions applied at surfaces indicated in Figure 5-1.</th>
</tr>
</thead>
</table>
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<table>
<thead>
<tr>
<th>Boundary condition</th>
<th>Boundary conditions applied at each surface</th>
</tr>
</thead>
<tbody>
<tr>
<td>BC1</td>
<td>$\sigma_z = 7\text{MPa} \quad p_w = 0.9\text{MPa}$</td>
</tr>
<tr>
<td>BC2</td>
<td>$\sigma_z = 4\text{MPa} \quad p_w = 0.9\text{MPa}$ Normal displacements and liquid flux prescribed</td>
</tr>
<tr>
<td>BC3</td>
<td>Normal displacements and liquid flux prescribed</td>
</tr>
<tr>
<td>BC4</td>
<td>$\sigma_z = 4\text{MPa} \quad p_w = 0.9\text{MPa}$</td>
</tr>
<tr>
<td>BC5</td>
<td>$\sigma_z = 4\text{MPa} \quad p_w = 0\text{MPa}$ due to presence of Niche MI</td>
</tr>
<tr>
<td>BC6</td>
<td>$\sigma_z = 4\text{MPa} \quad p_w = 0.9\text{MPa}$</td>
</tr>
<tr>
<td>BC7</td>
<td>$p_w = 0\text{MPa}$ to simulate the borehole excavation</td>
</tr>
</tbody>
</table>

5.1.2 Material parameters

A significant number of the parameters required by the formulation described are physical constants, the values of which have been indicated in Chapter 3. However, there are material specific parameters that are discussed in this section. The specific parameters used in the basic analysis are given in Table 5-3. In the analyses discussed in later sections some of the parameters may be varied to check on the sensitivity of results.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Symbol</th>
<th>Orientation to bedding</th>
<th>Units</th>
<th>Value</th>
</tr>
</thead>
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<td>Porosity</td>
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<td>%</td>
</tr>
<tr>
<td>Deformation parameters</td>
<td>Isotropy</td>
<td>Young's modulus</td>
<td>$E$</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Poisson's ratio</td>
<td>$\nu$</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Anisotropy</td>
<td>Young's modulus</td>
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<td>-</td>
<td>MPa</td>
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<tr>
<td></td>
<td>Friction</td>
<td>$\phi'$</td>
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<tr>
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<td>Intrinsic permeability</td>
<td>$k$</td>
<td>-</td>
<td>m2</td>
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</table>

Table 5-3 Reference parameters adopted for the numerical simulations. Comparative tables between different reported values on Section 2.3.

5.1.3 Description of the pore pressure measurements

Prior to the drilling of the heating borehole D0, instrumentation boreholes were installed. In October 2003 the HE-D niche was excavated, in order to install the drilling machine. In November 2003-December 2003 instrumentation boreholes were drilled from the HE-D and MI niche, and in February 2004 pore-water-pressure measurements started. The heating borehole was drilled between 2nd and 8th March 2004, followed by the installation of the heater a few days later. Only the pore-pressures measured until the beginning of the heating phase on 6th April 2004 will be considered in this work.
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Figure 5-2 Pore pressure evolution of all sensors from February 2004 to 6th April 2004. The borehole was drilled from 2nd of March to 8th of March and the heating test started on 6th April.

Figure 5-3 Measured pore pressure evolution of five sensors close to the heater borehole D0 (radial distance to D0 and axial distance from the HE-D niche indicated). The advance of the excavation front is indicated.

Pore pressures were measured in boreholes D0, drilled parallel to the heater borehole D0, and in a series of small-diameter boreholes, D7 to D17, drilled from the MI niche (Figure 5-4). Only the measurements of sensors D03, D08, D14, D15 and D16 show a clear response to the passage of the borehole (Figure 5-3) and will therefore be considered. Among them, the sensors located in boreholes D03 and D14 are the most sensitive ones. The comparison between simulated data and measurements will be
performed only with measurements of these two sensors. In Figure 5-4 a schematic outlay of sensors located in boreholes D03, D08 and D14 to D16, as well as the grid used for the simulation depicting sensors location is shown. The chronology of the drilling of the heater borehole D0 and the timing when the excavation reached the considered boreholes is indicated in Figure 5-5. As the instrumentation boreholes are located at different distances from HE-D niche (Figure 5-4a), the pore pressure response is delayed in time. Therefore the measurements have been normalized to begin at t=0 since the excavation has reached them (Figure 5-6).

Figure 5-4 (a) Plant, (b) longitudinal and (c) section views of the HE-D heater depicting sensor location. The bedding trace is indicated (modified from Garitte, 2010). (d) Section view with the grid used for the simulation.

These pore pressure measurements (Figure 5-6) will be compared with pore pressure simulations carried out. Boreholes D03 and D08 are located at approximately 8 m from HE-D niche, and boreholes D14, D15 and D16 are located at approximately 12 m. Their response is completely different, although the most different one is the pore pressure evolution of borehole D03, since it is drilled parallel to the heating borehole, and the other sensors are drilled normal to it from the MI niche. Pore pressure increase with drilling time along the parallel D03 borehole and in the normal sensor in D08. In sensors D14 to D16 there is a sharp drop of pore pressure when the drilling comes close to the measurement points, followed by a slow recovery until reaching steady values. These steady values are lower than the initial 0.9 MPa pore pressure due to rock massif drainage.
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Figure 5-5 Drilling advance of borehole D0. The time when the drilling reaches the different sensors is indicated.

Figure 5-6 Evolution of pore water pressure increments from the moment at which the heater borehole is drilled (March 2nd 2004).

Comparative simulations with consideration of anisotropy of the initial stress field and of clay stiffness have been done. The influence of anisotropy of stress state and stiffness has been investigated by means of a sensitivity analysis. It can be generally stated that
the higher the refinement of a mechanical model, the more considerable the accompanying efforts will be for both the determination of the design parameters and for carrying out the computation itself.

In Table 5-4 the values adopted in selected simulations of the sensitivity analysis are shown. The values adopted are in the range of the Opalinus Clay properties (see Section Error! Reference source not found. Properties and key parameters of Opalinus Clay).

<table>
<thead>
<tr>
<th>Initial conditions</th>
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<th>Cohesion</th>
<th>Viscosity</th>
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<td>( \sigma_3 ) [MPa]</td>
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<td>5</td>
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</table>

Table 5-4: Computations performed in the HE-D simulation and values of parameters used for each simulation. Simulations are named according to the following notation: \( E \) or \( P \) for a elastic or plastic model; \( Si \) or \( Sa \) for a stress isotropic or anisotropic model and \( Ei \) or \( Ea \) for a stiffness isotropic or anisotropic model.

First, an isotropic elastic model (E_Si_Ei simulation) is used as a first approach to the problem. Then an anisotropic stress field (E_Sa_Ei) with isotropic material stiffness and an anisotropic mechanical constitutive law (E_Si_Ea) with isotropic stress state is simulated. Finally, different simulations are carried out considering anisotropy of stress...
and stiffness (E$_{Sa}$-E$_{A}$ simulations) to analyse the sensitive of the described parameters. Because plastic effects might have a significant influence on the mechanical as well as on the hydraulic problem, the same approach is followed considering a plastic model. The elastoplastic model for argillaceous rocks described in Section 3.3.2 is adopted.

5.1.4 Elastic model

The excavation causes a sharp drop of pore pressure close to the tunnel (Figure 5-7). The drainage of the rock massif causes a progressive diminution of pore pressure. The boundary limits of the simulation depicted in Figure 5-1 have been selected in order to avoid an influence in the pore pressure dissipation close to the tunnel.

Figure 5-7 Liquid pressure evolution for simulation E$_{Si}$E$_{i}$ along x-axis. (a) Temporal evolution (b) Spatial evolution. The comparison of this simulation with two selected sensors, D03 and D14 ( ), shows that the short-term response is not well captured with this simple model.
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5.1.5 Plastic model

The use of a plastic deformation mechanism using a Mohr-Coulomb yield criterion with an associated plastic potential will cause expansive volumetric deformations in the plastic zone. This will be associated with decreasing pore water pressure in the nearly undrained regime considered, given the low permeability of Opalinus Clay and the rapid excavation rate (Figure 5-9).

Before analysing more complex models, a detailed analysis of the results obtained with a plastic model is realised. The relationship between temporal and spatial pore pressure evolution is shown in Figure 5-11 and Figure 5-10. Time and location of the different plots have been selected in order to show the most representative sections.
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Figure 5-10 Detailed temporal evolution of pore pressure evolution at selected distances from excavation contour along x-axis. The complete evolution is shown in Figure 5-9a.

Figure 5-11 Detailed spatial evolution of pore pressure evolution at selected distances from excavation contour along x-axis. The complete evolution is shown in Figure 5-9b. As suction only occurs for distances smaller than $x/D=2$ and immediately after excavation, this sharp pore pressure decrease is not detected by the sensors (Figure 5-12).
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Figure 5-12 (a) Liquid pressure evolution for simulation $P_{Si_Ei_1}$ at sensors D03 and D14. (b) Computed contour immediately after excavation ($t=1E-3$ days).

To provide a better understanding of the simulation results at the sensors location, Figure 5-13 depicts the location of five sensors and the pore pressure evolution after the excavation of the borehole. It is clearly shown that the response in the immediate vicinity of the tunnel will never be measured, because the sensors are located at different distances from the tunnel, ranging from $r/D = 2$ m (sensor D14) to $r/D = 4.2$ m (sensor D16).

Figure 5-13 Pore pressure evolution considering a plastic model ($P_{Si_Ei_1}$) at the location of the sensors. Note the differences with Figure 5-10.

Plastic displacements are much larger than elastic displacements. In Figure 5-14 it can be observed that the maximum elastic displacements (at $t=35$ days) are equal to the instantaneous displacements in the plastic simulation ($t=1E-08$ days).
Figure 5-14 Comparison of elastic and plastic displacements at various times. Positive values indicate outward displacements.
Radial stresses gradually recover their in situ value of 5 MPa (Figure 5-15) after the release of stress caused by the excavation. Circumferential stresses are highest at the side walls of the excavation (Figure 5-16). The stress peak depicts the extent of the yielding zone; outside this zone the rock massif behaves in an elastic manner. The progressive yielding of the medium can be observed. This behaviour is in accordance the evolution of plastic multiplier (Figure 5-17). 35 days after the excavation the extent of the yielding zone is \( x/D = 1 \).

Figure 5-15 Radial stresses along x-axis (P_Si_Ei_1 simulation).
The stress path that the rock mass surrounding an underground excavation undergoes can be used to assess the potential for damage around the excavation. The stress path for selected locations is shown in Figure 5-18. As seen before, the highest deviatoric stress is caused by the excavation in the elements close to it. The adoption of a viscoplastic model allows the stress state to overpass the Mohr-Coulomb yield criterion. The drop of pore pressure causes a rise in the mean effective stresses close to the tunnel (Figure 5-18). First the contour of the excavation reaches the yield criterion; as this zone yields, the load is transmitted to the next hoop.
**Figure 5-18** Stress paths at selected locations along x-axis (P_Si_Ei_1 simulation).
5.1.6 Sensitivity analysis

A sensitivity analysis has been performed to examine the effect of various parameters and, in this way, improve the understanding of the system subject to hydro-mechanical perturbations. Because many coupled HM computations had to be performed a linear mechanical constitutive model as the base case. The effect of variation of individual parameters will be demonstrated by reference to the pore pressure increment evolution in boreholes D03 and D14, the most sensitive to the excavation.

A selection of the sensitivity analyses performed will be presented, focusing on the most important or uncertain parameters: Young’s modulus $E$, cohesion $c$, viscosity $\eta$, intrinsic water permeability $k$, in situ stress field and bedding dip.

5.1.6.1 STRESS AND STIFFNESS ANISOTROPY

Stress anisotropy triggers a pore pressure increase in the minor stress direction and a pore pressure decrease in the major stress direction (Figure 5-19). The larger vertical stress causes a larger displacement at the crown and therefore pore pressure drops in undrained conditions. At the side walls the larger circumferential stress tries to reduce pore space, but as no volumetric deformation is possible in such a rapid excavation performed in a low permeability claystone, pore pressure increase. This effect is magnified in a plastic simulation (Figure 5-20), as pore pressure dissipate more slowly because of yielding. This simulation captures the measurements response in sensor D14 qualitatively.

---

Figure 5-19 Pore pressure evolution considering stress anisotropy in an elastic model (E_Sa_Ei_1).

Figure 5-20 Pore pressure evolution considering stress anisotropy in a plastic model (P_Sa_Ei_1).
A similar pore pressure response is observed when stiffness anisotropy is considered (Figure 5-21). It has been previously stated (Section 2.2.3) that Opalinus Clay is a distinctively bedded material, whose mechanical behaviour can best be described in a transverse isotropic model. In the bedding direction a larger stiffness is expected. Displacements will be larger in the direction normal to bedding, and therefore lower pore pressures will be expected considering undrained conditions. A sensitivity analysis for Young modulus values, as well as bedding dip, will be analysed later.

**Figure 5-21** Pore pressure evolution considering stiffness anisotropy in elastic model (E_Si_Ea_1). A sharper pore pressure response is observed in the location of sensor D03 in the plastic model (Figure 5-22). The magnitude of the pore pressure change is smaller compared with the effect of stress anisotropy.

**Figure 5-22** Pore pressure evolution considering stiffness anisotropy in a plastic model (P_Si_Ea_1). The combined effect of stress and stiffness anisotropy matches better the measurements in elasticity (Figure 5-23). An elasto-plastic model has been proven to simulate better the pore pressure evolution than a simple linear elastic model (Figure 5-24). The magnitude of pore pressure increase is well simulated in sensor D14 with this simple approach.
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Figure 5-23 Pore pressure evolution considering stress and stiffness anisotropy in an elastic model (E_Sa_Ea_1).

Figure 5-24 Pore pressure evolution considering stress and stiffness anisotropy in a plastic model (P_Sa_Ea_1).

5.1.6.2 LONGITUDINAL STRESS

Given the great variability of the minor principal stress obtained in tests (Section 2.3.1) a sensitivity analysis has been carried out to determine the influence of longitudinal stress. Values of 2 MPa, 3 MPa and 4 MPa have been evaluated in an elastic model (E_Sa_Ea_3)
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5.1.6.3 Young modulus

The value of Young’s modulus naturally affects deformations, but also the magnitude of the pore pressure generation. Analyses have been performed using values of $E$ equal to 6 GPa and 10 GPa in the direction parallel to bedding and values equal to 3 GPa and 4 GPa in the direction perpendicular to bedding, respectively.

Elastic Plastic

Lowering the clay stiffness results in a reduced instantaneous pore pressure generation and larger displacements. The overall pore pressure response is delayed. $E_1=6\,\text{GPa}$ and $E_2=3\,\text{GPa}$ fit better to the measurements.
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5.1.6.4 Bedding Dip

Simulations performed considering a bedding dip of 45º match better the measurements.

5.1.6.5 Cohesion

A cohesion of 1 MPa has been selected to match the measurements.

5.1.6.6 Viscosity

Viscosity controls the rate at which plasticity develops. Although the extent of the plastified envelope is similar, a much greater plastified damage is developed in a viscous medium. This happens because plasticity develops much sooner in a highly viscous medium than in an inviscid medium. It can be observed that in a slightly viscous medium, $\eta = 10^8\, MPa\cdot s$, plasticity occurs 3 days after the excavation, while in a highly viscous medium, $\eta = 10^9\, MPa\cdot s$, the tunnel contour has already plastified after five minutes. Moreover, 35 days after excavation, the plastified magnitude in the first medium accounts for 0.0014. In contrast, in the second medium the plastified value after 35 days reaches a value of 0.4 (Figure 5-33).
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Figure 5-33 The plastified multiplier has the same value for (a) $\eta = 10^6 \text{MPa.s}$ (P_Sa_Ea_15) 5 minutes after the excavation and for $\eta = 10^6 \text{MPa.s}$ (P_Sa_Ea_19) at $t=35$ days. (b) The plastic damage is much greater for $\eta = 10^6 \text{MPa.s}$ (P_Sa_Ea_15) at $t=35$ days.

The higher the viscosity rate, the more similar to the elastic response is the pore pressure evolution. As seen before, in an elastic medium the drainage of the rock massif is instantaneous.

Figure 5-34 PsaEa5

Figure 5-35 PsaEa7

Figure 5-36 PsaEa11 $45^\circ$ E1=6

Figure 5-37 PsaEa13 $45^\circ$ E1=10

5.1.6.7 PERMEABILITY

The rate of pore pressure dissipation is controlled mainly by the value of water permeability. Therefore variations of intrinsic water permeability $k$ are bound to have a profound effect on the pore pressure response.
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Figure 5-38 Effect of intrinsic permeability on pore pressure evolution. Permeability parameters in the range of reference values.
5.2 **MINE-BY TEST**

An instrumented mine-by test was performed in Niche MB. The excavation of the instrumented tunnel took place in October/November 2008. In Figure 5-39 the location of the sensors is depicted.

**Figure 5-39** A) Plant and B) section views of the MB niche depicting sensor location (modified from Garitte, 2010).

Despite the presence of microfaults the deformation monitoring around the advancing tunnel face showed a remarkably homogenous response to the excavation. Pore pressure peaks (Figure 5-40) systematically increased with decreasing passing distance of the tunnel face.

**Figure 5-40** Pore pressure evolution of two piezometers parallel to the MB niche (radial distance to center and axial distance from Gallery 08 indicated). The advance of the excavation front is indicated. The pore pressure evolution has been normalised in order to analyse the response of the sensor (Figure 5-41).
In Figures 5-41 and 5-42, the pore pressure evolution since excavation at each sensor is shown.

**Figure 5-41** Evolution of pore pressure since excavation at each sensor.

**Figure 5-42** Piezometer 16 pore pressure evolution.
Pore pressures increase with excavation time and the drop essentially to zero when the tunnel comes close to the measurement points. This pore pressure changes are caused by stress redistribution in the rock from the unloading during excavation, leading to de-stressing of the rock and the pore space.

In Figure 5-44 the location of the sensors in grid considered for this simulation is shown.
5.2.1 Predicted and measured MB excavation response

The excavation of the MB test was monitored by an array of instruments providing information about the deformations and pore pressures around the tunnel. To aid in the interpretation of the measured response, the finite element program Code Bright was used to predict the elastic rock mass response. The goal of this approach is to assess when the rock mass response deviates from the linear elastic behaviour. This approach is equivalent to a total stress or undrained approach, i.e., the effect of pore pressure is ignored. Given the low permeability of the Opalinus Clay and the rapid rate excavation, this modelling approach is appropriate for defining the approximate boundary between the elastic and non-elastic response. Quantifying the extent of the non-elastic zone will help establish the limits for the Excavation Disturbed Zone.

Following the approach described in the previous section, a sensitivity analysis was developed to determine the material properties. A simulation was performed with the values obtained with the sensitivity analysis carried out for the HE-D test (Figure 5-45).

![Figure 5-45Comparison of results obtained with a plastic full anisotropic simulation and the pore pressure measurements in borehole 15.](image-url)
6. CONCLUSIONS

In this thesis, a first acquaintance was made with numerical computations on the one hand and processes analysis in geomechanics on the other hand. A finite element code (Code_Bright) allowing for the resolution of heat, water and gas flow in deformable (un)saturated porous media was used and the theoretical formulation was studied in details. The measured response of a low permeability argillaceous rock to excavation and drilling was analysed using that numerical tool. The possibility to test out different scenarios allowed improving the understanding of the mechanisms leading to the observed rock behaviour and to the pore water pressure in particular.

In general, excavation induces a multi processes coupled response in the rock around an excavation. Rock movements resulting from excavation induce a volumetric deformation of the pore space which gives rise to strong pore-water pressure variations enhanced by the low rock permeability. In the long term, excess pore water pressure will dissipate towards the steady-state profile determined by the hydraulic boundary condition at the excavation wall. These pore-pressure changes have a significant influence on the effective stress state in the rock and may contribute to failure processes.

The influence of different parameters such as the strength, rigidity, rigidity anisotropy, time dependent deformations, initial stress state and permeability was investigated. In the drilling experiment, the observed increase of excess pore water pressure in the bedding plane and the associated decrease in the perpendicular direction can be explained qualitatively by both, stiffness anisotropy and stress anisotropy. Quantitative reproduction of the pore water pressure however, could only be explained by the combined effect of the stiffness anisotropy and the development of the EDZ. The rate of development of the EDZ and the permeability value used were found important to reproduce correctly the evolution of pore water pressure. The pore water pressure sensors installed at one to three diameters from the borehole wall were found to be outside the EDZ, but their behaviour is strongly influenced by the EDZ development.

In the mine-by test, in which the influence of a 4.5m diameter gallery is studied, pore water pressure sensors at about one diameter from the gallery wall were found to be inside the EDZ. During the passage of the gallery front, a rapid pore water pressure decrease reaching suction quite rapidly was observed in these sensors. The simulation of the gallery excavation using the same parameters as for the borehole drilling allowed for explaining this behaviour.
7. BIBLIOGRAPHY

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