

MECHANICAL AND HYDROMECHANICAL BEHAVIOR OF HOST
SEDIMENTARY ROCKS FOR DEEP GEOLOGICAL REPOSITORY FOR
NUCLEAR WASTES

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Summary

Sedimentary rocks are characterized with very low permeability (in the order of 10^{-22} m^2), low diffusivity, a possible self-healing of fractures, and a good capacity to retard radionuclide transport. In recent years, sedimentary rocks are investigated by many research groups for their suitability for the disposal of radioactive waste. Development of deep geologic repositories (DGRs) for the storage of radioactive waste within these formations causes progressive modification to the state of stress, to the groundwater regime, and to the chemistry of the rock mass. Thermal effects due to the ongoing nuclear activity can cause additional disturbances to the system. All these changes in the system are coupled and time-dependent processes. These coupled processes can result in the development of an excavation damaged zone (EDZ) around excavations. More permeable than the undisturbed rock, the EDZ is likely to be a preferential pathway for water and gas flow. Consequently, the EDZ could be a potential exit pathway for the radioactive waste to biosphere. An investigation of the Hydraulic-Mechanical (HM) and Thermal-Hydraulic-Mechanical-Chemical (THMC) behaviour of sedimentary rock formations is essential for the development of DGRs within such formations.

This research work consists of (1) an experimental investigation of the mechanical behaviour of the anisotropic Tournemire argillite, (2) modeling of the mechanical behaviour of the Tournemire argillite, and (3) numerical simulations of the mechanical and hydromechanical behavior of two host sedimentary rocks, the Tournemire argillite and Cobourg limestone, for deep geological repository for nuclear wastes.

The experimental program includes the measurements of the physical properties of the Tournemire argillite and its mechanical response to loading during uniaxial compression tests, triaxial compression tests with different confining pressures, unconfined and confined cyclic compression tests, Brazilian tests, and creep tests. Also, acoustic emission events are recorded to detect the initiation and propagation of microcracks within the rock during the uniaxial testing.

The approach for modeling the mechanical behaviour of the Tournemire argillite consists of four components: elastic properties of the argillite, a damage model, the proposed concept of mobilized strength parameters, and the classical theory of elastoplasticity. The combination of

the four components results in an elastoplastic-damage model for describing the mechanical behaviour of the Tournemire argillite. The capabilities of the model are evaluated by simulating laboratory experiments.

Numerical simulations consist of: (1) a numerical simulation of a mine-by-test experiment at the Tournemire site (France), and (2) numerical simulations of the mechanical and hydromechanical behaviour of the Cobourg limestone within the EDZ (Canada). The parameters influencing the initiation and evolution of EDZ over time in sedimentary rocks are discussed.

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List of main symbols

<u>Symbol</u>	<u>Definition</u>
σ_{ij}	total stress tensor
σ'_{ij}	effective stress tensor
σ_H	maximum in-situ horizontal stress
σ_h	minimum in-situ horizontal stress
σ_v	in-situ vertical stress
σ_{cs}	uniaxial compressive strength of the rock
σ_t	uniaxial tensile strength of the rock
σ_{peak}	deviatoric stress at the peak
$\sigma_1 - \sigma_3$	deviatoric stress
σ_{ci}	crack initiation stress level
σ_{cd}	crack damage level
ϵ_{ij}	strain tensor
ϵ'_{ij}	effective strain tensor
ϵ_a	axial strain
$\epsilon_{kk}, \epsilon_v$	volumetric strain
ϵ_r	radial strain
ϵ_{eps}	effective plastic strain
I_1	first invariant of the stress tensor σ_{ij}
J_2	second invariant of the stress deviator s_{ij}
F	force, e.g., external load or body force
f	yield function
H	hardening parameter
p	fluid pressure
g	gravitational acceleration
D_e	elastic compliance matrix
D_{ep}	elastoplastic compliance matrix
δ_{ij}	Kronecker's delta
u	displacement

E	elastic modulus
G	shear modulus
ν	Poisson ratio
ρ	density of the solid
ρ_f	density of the fluid
k	permeability of the medium
K_s	bulk modulus of solid grains
K_f	bulk modulus of the fluid
θ	loading orientation angle
β	Lode angle
v	average flow velocity
α_B	Biot coefficient
μ	fluid viscosity
ω	porosity
V	volume of porous medium
d	damage variable (mechanical)
h	damage variable (hydraulic)
t	time
C	cohesion
φ	internal friction angle
C_{pp}	cohesion in post-peak region
φ_{pp}	internal friction angle in post-peak region
C_{peak}	cohesion at the peak strength
φ_{peak}	internal friction angle at the peak strength

List of publications resulting from the work performed for this thesis

1. Abdi, H., Evgin, E., Fall, M., Nguyen, T.S., and Su, G. 2010. *Two and three dimensional numerical analyses of excavation damaged zones around deep geological repositories in sedimentary rocks*. Canadian Geotechnical Conference, Calgary, Canada, pp. 759-767.
2. Abdi, H., Nguyen, T.S., Evgin, E., Fall, M., and Su, G. 2010. *Coupled Hydromechanical Analysis of Excavation Damage Zones around an Underground Opening in Sedimentary Rock*. COMSOL Conference, Boston, pp. 1-7.
3. Abdi, H., Labrie, D., Nguyen, T.S., Barnichon, J.D., Su, G., Evgin, E., Simon, R., and Fall, M. 2012. *Laboratory investigation and preliminary modelling of the mechanical behaviour of a transversely isotropic shale*. Canadian Geotechnical Conference, Winnipeg, Canada, pp. 1-8.
4. Abdi, H., Labrie, D., Nguyen, T.S., Barnichon, J.D., Su, G., Evgin, E., Simon, R., and Fall, M. 2012. *Laboratory investigation of the mechanical behaviour of transversely isotropic Tournemire argillite*. 5th International meeting "Clay in natural and engineered barriers for radioactive waste confinement", Montpellier, 220-26 October, 2012.
5. Abdi, H. and Evgin, E. 2013. Laboratory characterization, modeling, and numerical simulation of an excavation damaged zone around deep geologic repositories in sedimentary rocks. Report prepared for Canadian Nuclear Safety Commission, 128 pages. Contract No. and Title: 87055-08-1075: R413.2.
6. Abdi, H., Labrie, D., Nguyen, T.S., Barnichon, J.D., Su, G., Evgin, E., Simon, R., and Fall, M. 2013. *A laboratory investigation on the mechanical behaviour of Tournemire argillite*. Revised and submitted for publication in the Canadian Geotechnical Journal (Manuscript – 40 pages).
7. Abdi, H., Simon, R., and Fall, M. 2013. *Modeling of the mechanical behaviour of the anisotropic Tournemire argillite*. To be submitted for publication (Journal paper).

8. Abdi, H., Fall, M., and Simon, R. 2013. *Characterization of excavation damaged zones around deep geologic repositories in sedimentary rocks*. To be submitted for publication (Journal paper).

CHAPTER 1

INTRODUCTION

1.1. Background and problem statement

Disposal of radioactive waste in deep geological formations is considered internationally as a possible solution method for the long-term management of radioactive waste. Today, many research programs/cooperative projects, e.g. DECOVALEX project, are underway in many countries for characterizing deep geologic repository (DGR) sites. Potential types of rock mass to host radioactive waste are the hard rock, salt rock, and sedimentary rock (e.g., argillaceous rocks). Nowadays, much of the research is directed towards investigating the Hydraulic-Mechanical (HM) and Thermal-Hydraulic-Mechanical-Chemical (THMC) behaviour of sedimentary rock formations (e.g., ANDRA, 2012; Lam et al., 2007; Read and Birch, 2009).

Sedimentary rock formations are typically formed by deposition and progressive consolidation of marine sediments. Consequently, they are characterized by the presence of closely spaced bedding planes, resulting in anisotropy of their behaviour (mechanical, hydraulic, thermal). A couple of underground research laboratories (URLs) have been developed to investigate the HM and THMC behaviour of sedimentary rock formations, and to obtain data that can be used to predict the deformation and failure of such formations. Examples of such laboratories are the Mont Terri test facility in Switzerland, and the Meuse/Haute-Marne and Tournemire facilities in France (Fracture Systems Ltd., 2011).

Rock mass at depth are subjected to stresses resulting from the overburden pressure and from locked-in stresses of tectonic origin. Development of a DGR within a rock mass results in stress redistribution, perturbation to the groundwater regime, and perturbation to the chemistry of the rock mass. Thermal effects due to the ongoing nuclear activity can cause more disturbances to the system. All these processes are coupled and time-dependent. They can result in the

development of an excavation damaged zone (EDZ) around the excavations. More permeable than the undisturbed rock, the EDZ is likely to be a preferential pathway for water and gas flow and it could be a potential exit pathway for the radioactive waste to biosphere.

The size of the EDZ depends mainly on the state of stress, mineralogical composition of the rock, excavation method, geometry and orientation of the tunnel, hydraulic and mechanical properties of the rock, and geological structure of the rock mass. The initiation and evolution of the EDZ over time are essential considerations in the evaluation of the long-term performance of a DGR. Many attributes could provide confidence in the efficiency of a DGR such as (1) the homogeneity and sufficient lateral and vertical extent of the rock mass, (2) transport of solutes dominated by diffusion processes, and (3) continued stability of the geosphere for the past millions of years. One of the important design requirements for a DGR is to be constructed within a relatively unfractured and stable geological media with low permeability. Large scale hydrological mapping could be an important factor in locating a DGR site.

In most rocks, two kinds of inelastic deformation might be observed during excavation: the plastic deformation inside the rock matrix and material damage induced by the initiation and propagation of microcracks. Microcracking process affects both mechanical and hydraulic properties of the rock, induces softening, and in some cases can be the main cause of instability problems (Martin, 1993; Rutqvist et al., 2009). The scale and the intensity of the microcracks depend mainly on the type of the rock and the excavation method (Emsley et al., 1997; Sugihara, 2009). In general, the crack intensity decreases with the increase in the distance from the underground opening.

HM and THMC processes have been the subject of many research activities for some decades; see for example Autio et al. (2009), Avis et al. (2009), Cho et al. (2008), Hudson et al. (2009), Rutqvist and Stephansson (2003), Rutqvist et al. (2009), Sato et al. (2000), and Tang et al. (2002). This topic requires further work especially in the context of the disposal of nuclear wastes within sedimentary rock formations. Therefore, the main objective of this research program is to investigate the HM behaviour of sedimentary rocks within the EDZ. It consists of a laboratory testing program, modeling, and numerical simulations: 1) the laboratory testing part consists of investigating the mechanical behaviour of a sedimentary rock, namely the anisotropic Tournemire argillite, 2) the modeling part consists of the development of an elastoplastic-

damage model for describing the mechanical behaviour of the Tournemire argillite, and 3) the numerical simulation part consist of: i) numerical simulation of HM behaviour of the Tournemire argillite within the EDZ, and ii) the results obtained in steps (1), (2), and (3.i) helps in understanding the HM behaviour of sedimentary rocks, consequently, the M and HM behaviour of another sedimentary rock within the EDZ, namely the Cobourg limestone, is also investigated.

1.2. Objectives and scope of this research

The main objectives of this research program are:

1. To establish the state-of-the-art knowledge on the development of DGRs around the world and in Canada. This includes: potential types of rock mass to host radioactive waste; EDZ initiation and evolution; sealing and self-sealing of the EDZ; strength, deformation, and failure of a rock mass; mathematical formulations of poroelasticity, elastoplasticity, and continuum damage mechanics; and finite element codes.
2. To develop and carry out a laboratory testing program for investigating the mechanical behaviour of the anisotropic Tournemire argillite.
3. To record acoustic emission events in order to: (1) understand the processes involved in the initiation and propagation of microcracks within the argillite, and (2) estimate the amount of damage that can be caused to the mechanical properties of the argillite.
4. To formulate and validate an elastoplastic-damage model for describing the mechanical behaviour of the anisotropic Tournemire argillite. This includes the determination of the elastic parameters of the argillite, development of general equations for estimating mobilized strength parameters at any stress level and for any loading orientation angle, development of a damage model, and integration of the developed equations and damage model with the classical theory of elastoplasticity.

5. To use the experimental and modeling results obtained in step (2) to step (4), and data and equations from open literature to:
 - i. Numerically simulate a mine-by-test experiment performed in 2003 at the Tournemire site, France.
 - ii. Numerically simulate the mechanical and hydromechanical behaviour of the Cobourg limestone within the EDZ, Canada.
6. To estimate the sizes of the EDZ around both DGRs and discuss the main parameters influencing the initiation and evolution of the EDZ over time in sedimentary rocks.

1.3. Research plan of this thesis

The research plan of this thesis is as follows (Fig. 1.1):

1. Conduct a literature review to establish state-of-the art knowledge on the development of DGRs around the world and in Canada. This is the focus of Chapter 2.
2. Conduct a literature review of the concepts of poroelasticity, damage mechanics, and elastoplasticity, and evaluate the capabilities and limitations of different available Finite Element codes for simulating HM processes in geological media. This is the focus of Chapter 3.
3. Test a large number of the Tournemire argillite specimens and describe the experimental results. This is the focus of Chapter 4.

4. Interpret the experimental results and discuss the key aspects of the observed mechanical behaviour of the argillite. This is the focus of Chapter 5.
5. Carry out a significant amount of theoretical work and develop an elastoplastic-damage model for the mechanical behaviour of the anisotropic Tournemire argillite. This is the focus of Chapter 6.
6. Numerically simulate the mechanical and hydromechanical behaviour of two sedimentary rocks within the EDZ, namely the Tournemire argillite and Cobourg limestone, and interpret the numerical results. This is the focus of Chapter 7.
7. Provide conclusions and recommendations for future work. This is the focus of Chapter 8.

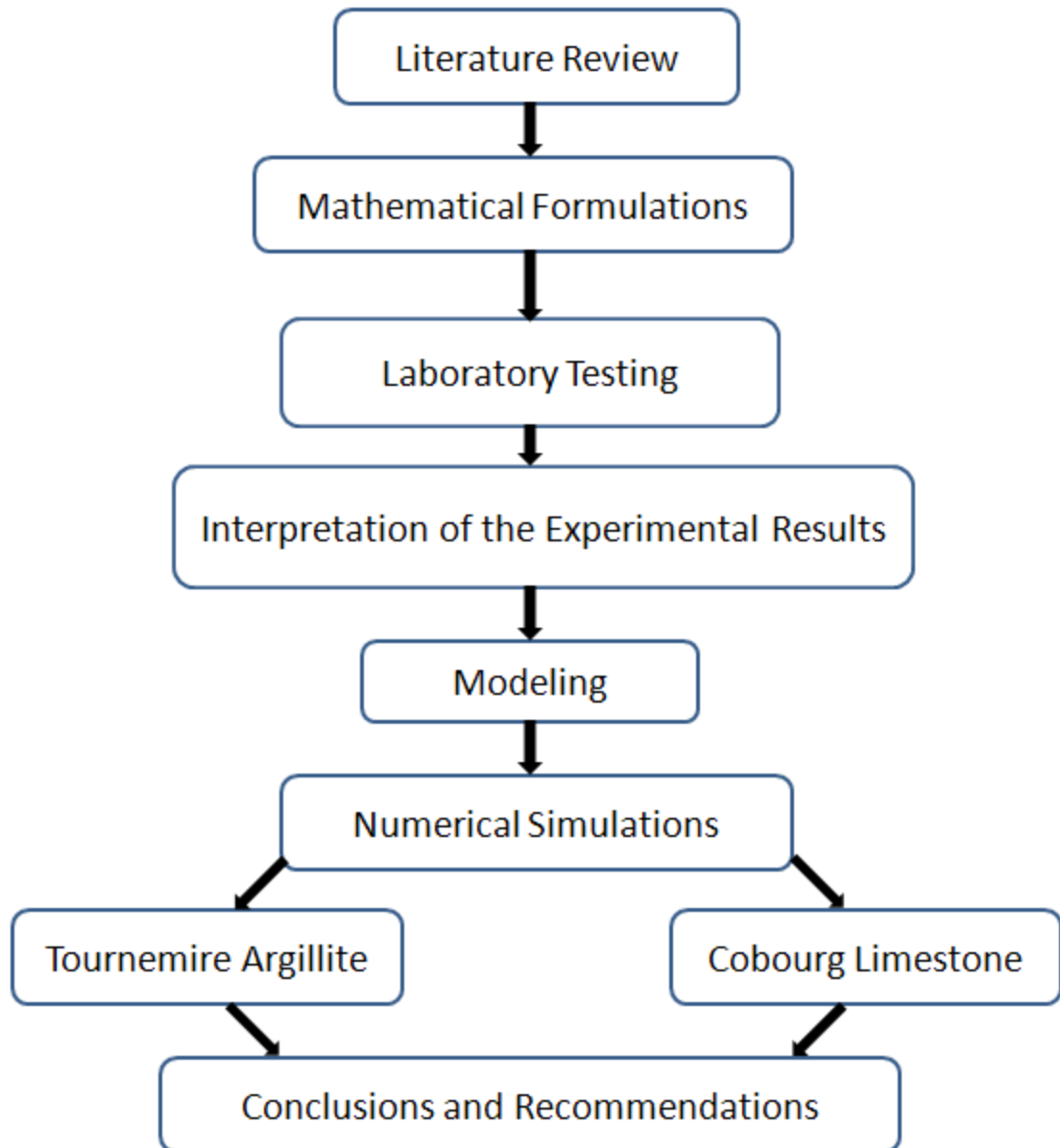


Figure 1.1. Research plan for the thesis.

1.4. Statement of the originality

My contribution to understanding of rock behaviour and EDZ in a DGR can be summarized as follows:

1. An experimental database on the mechanical properties of the anisotropic Tournemire argillite (France) has been generated. The argillite specimens were extracted at a depth 250 m below the ground level and from boreholes drilled at different inclinations with respect to the orientation of bedding planes. Around fifty uniaxial tests, triaxial tests with different confining pressures, unconfined and confined cyclic tests, and creep tests were carried out using a triaxial machine. Cylindrical argillite specimens of 61.3 mm in diameter and 133 mm in height were used. The specimens were loaded in five different directions to bedding planes; i.e., $\theta=0^\circ$, 30° , 45° , 60° , and 90° . Around twenty three (23) Brazilian tests were also carried out using specimens with the same diameter and a nominal length of 40 mm. The specimens were also loaded in five different directions to bedding planes. Acoustic emission events were recorded during uniaxial testing. Basic tests were also carried out to determine the physical properties of the argillite.
2. Key aspects of the observed mechanical behaviour of the argillite are described: elastic behaviour, plastic behaviour, cyclic behaviour, creep behaviour, transition from contraction to dilation of volumetric deformation, damage due to inelastic deformations, microcracking of the argillite, and failure behaviour.
3. Based on the acoustic emission recorded data, a concept describing the mobilization of strength parameters during loading the Tournemire argillite has been proposed and validated (concept of mobilized strength parameters).
4. An elastoplastic-damage model for describing the mechanical behaviour of the Tournemire argillite has been developed and validated. The model consists of four components: the elastic properties of the argillite, the developed damage model, the

proposed concept of mobilized strength parameters, and the classical theory of elastoplasticity.

5. The experimental and modeling results obtained in step (1) to step (4), and data and equations from open literature have been used to:
 - i) Simulate a mine-by-test experiment performed in 2003 at the Tournemire site, France.
 - ii) Simulate the mechanical and hydromechanical behaviour of the Cobourg limestone within the EDZ.
6. The numerical results obtained in step (5) have been interpreted. The sizes of the developed EDZ around both DGRs have been estimated. The main parameters influencing the initiation and evolution of the EDZ over time in sedimentary rocks have been discussed.

CHAPTER 2

LITERATURE REVIEW ON THE STATE-OF-THE-ART KNOWLEDGE ON THE DEVELOPMENT OF DGRs

2.1. Background

Many international cooperative projects have been implemented in support of the performance assessment for underground disposal of radioactive waste. The general goals of these projects were to conduct cooperative research on modeling coupled THMC processes (Fig. 2.1) in geologic formations. As an example, the DECOVALEX project was designed to model the evolution of the EDZ and investigate the effects of coupled THMC processes around a DGR in hard rock. A summary of the results from the DECOVALEX-THMC project can be found in, e.g., Hudson et al. (2008).

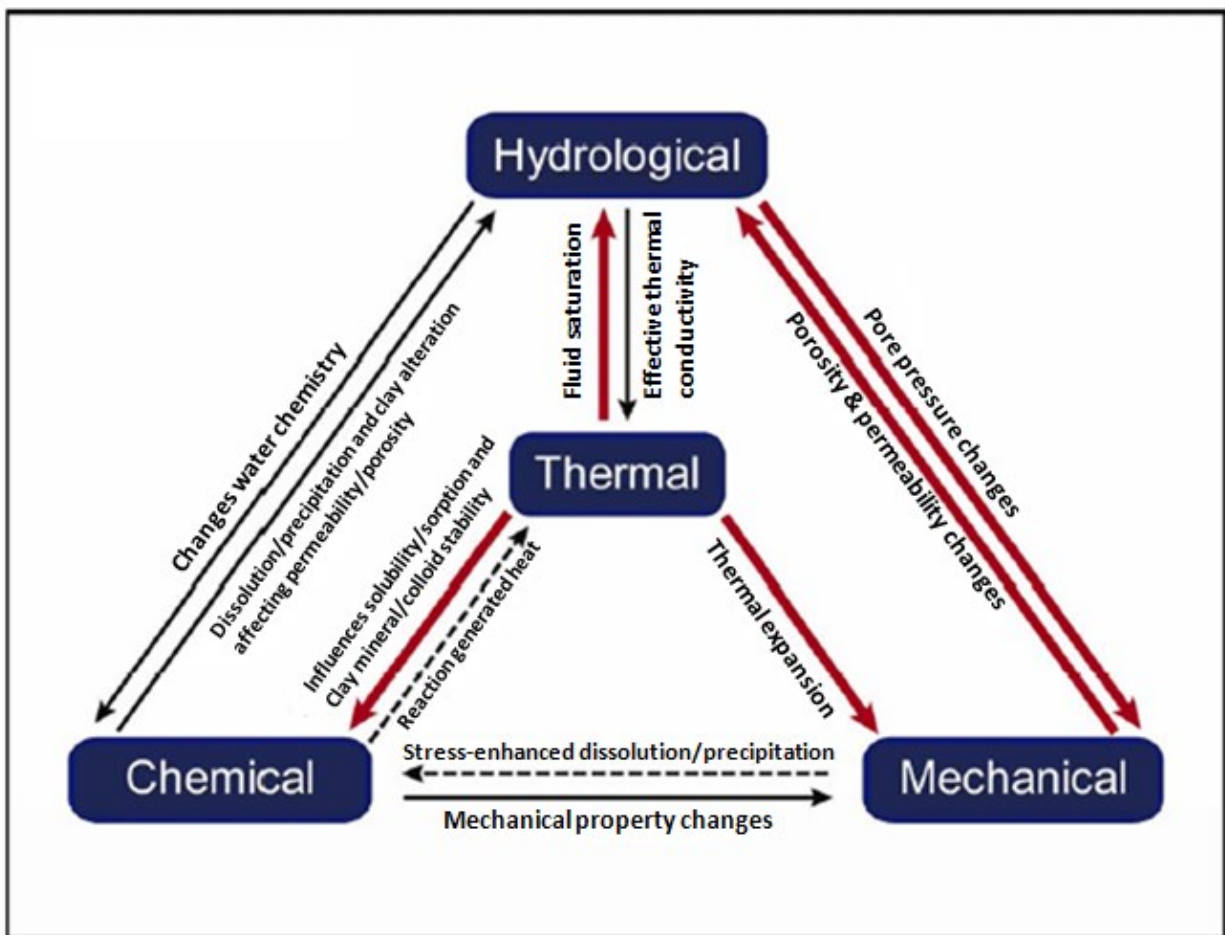


Figure 2.1. Coupled Thermal-Hydrological-Mechanical-Chemical (THMC) processes (Manepally et al., 2012).

Tunnelling affects the in situ stress field. Consequently; tensile, compressive, and shear stresses can develop in different parts of the rock around the tunnel. For unsupported tunnels, the stress normal to the walls is vanished while tangential stresses may increase significantly and can lead to local failures under tension and/or shear. Such processes can be accelerated due to the hydraulic loads. A part of the near-field rock could become compressed or expanded and porewater pressure may increase or decrease locally.

Kaiser et al. (2000) stated that two forms of instability can develop around underground openings. Structurally controlled failures are common in low confining stress environments at shallow depths, and stress-induced yielding occurs when stress magnitudes reach the rock mass strength. In weak and soft rocks the yielding may result in large convergence displacements. In hard rock the yielding may result in relatively small convergence displacements.

In recent years, a lot of efforts have been made for a potential repository in three broad rock types, namely, crystalline rock, salt, and argillaceous formations (Souley et al. (2001), Martin (1993), Langer (1999), Liang et al. (2006), Niandou et al. (1997), and Tang et al. (2002)). All these efforts have involved large-scale field tests, laboratory characterization, theoretical studies, and numerical modeling. Among the three rock types, field studies of crystalline rocks are probably the most extensive. These have included large-scale tests which have the advantage of being able to account for realistic anisotropy and heterogeneity effects that are always present in situ.

2.2. Potential types of rock mass to host radioactive waste

2.2.1. Crystalline rocks

Several countries are considering construction of repositories in crystalline rock below the groundwater table. These rocks generally have extremely low permeability, except where they are fractured. Major excavation response experiments in crystalline rocks have been finalized

including Stripa and Aspo (Sweden), URL-Whiteshell Research Area (WRA) (Canada), Grimsel (Switzerland), and KURT (South Korea).

The Stripa project was carried out in an abandoned iron mine (Carlsson 1986). The objective of this project was to test the predictive capabilities of radar and seismic characterization methods and numerical groundwater models. Most of the experiments were carried out at the 360 meter level in massive granite. The functionalities of the bentonite as a buffer material were evaluated. The orientation and the extent of fractures in the site were mapped successfully. Stress-permeability relationships were developed and major flow paths were identified.

The URL-WRA project was developed in the Canadian Shield for the purpose of carrying out large scale in situ tests in the granite of Lac Du Bonnet. The main working levels of the URL are at depths of 240 m and 420 m. The geology and fracture distributions in the vicinity of the URL site were extensively investigated (Soonawala, 1984). Souley et al. (2001) found that the in situ measurements of permeability and predictions were globally in agreement, and the depth of 50–70 cm is a good estimation of the extent of the EDZ around an elliptical cross section tunnel (3.5 m high by 4.4 m wide). Rutqvist et al. (2009) carried out numerical modeling for the excavation-induced damage, permeability changes, and fluid-pressure responses. The modeling indicated that stress-induced permeability increase above the tunnel is a result of micro and macro-fracturing under high shear stresses, whereas permeability increase alongside the tunnel is a result of opening of existing micro-fractures under the decrease in mean stress.

Also at the URL-WRA project, the tunnels excavated at the 420-meter level showed signs of instability such as spalling and notch development (Martin, 1993). It is generally assumed that cohesion and friction of the material are mobilized at the same displacements such that both components can be relied on simultaneously. However, damage testing of samples of Lac du Bonnet granite has shown that as friction is mobilized in the sample, cohesion is reduced (Martin and Chandler, 1994).

The Grimsel project consists of several 3.5 m diameter tunnels excavated in granite. The purpose of this project is to carry out tests in many branches of science including geology, geophysics, hydrogeology, rock mechanics, and nuclide transport. The data obtained in this project was used

to improve the understanding of the interaction between modelling, laboratory tests, and in situ tests. The investigation included heat transfer, rock stress measurements, EDZ, and chemical transport.

In South Korea, a small scale underground research laboratory (KURT) was developed in granite to investigate the behaviour of barriers (Cho et al., 2008). Borehole drilling, geological survey, and in situ and laboratory tests were carried out to validate the numerical modeling. It was found that the size of the EDZ varies from 1.1 m to 1.5 m around a horseshoe shape tunnel (6 m high by 6 m wide). The degradation in the elastic modulus and rock strength within the EDZ were around 50% and 15%, respectively.

International experts carried out a variety of experiments at a depth of nearly 500 meters in the Aspo underground hard rock laboratory to test the repository's barriers (Autio et al., 2003). It was found that the porosity of the damaged rock zone is higher than the porosity of undisturbed rock. For the experimental deposition tunnels of diameter 1.75 m, the average depth of the damaged zone varies from 5 mm to 20 mm from the excavation wall. The extent of the EDZ and the hydraulic conductivity of the EDZ adjacent to the floors of the tunnels are generally larger than in the roof and wall sections (Autio et al., 2005).

The excavation method is an important factor in controlling the excavation damage zone. Using a tunnel boring machine (TBM) method, less than 5 mm of damage were recorded at Aspo and at Grimsel projects. Excavation using drilling and blasting resulted in more damage. In the ZEDEX project, Emsley et al (1997) found that the extent of the damaged zone is significantly greater in the drift excavated by blasting compared to the drift excavated by TBM. Similar observations have been reported for crystalline rock in Japan (Sugihara, 2009).

In general, it is observed that only a small number of fractured zones account for the majority of the flow of fluids in crystalline rocks. Consequently, DGRs should be developed within a relatively un-fractured, low permeability, and stable geological media to isolate the radioactive waste from conductive fractured zones. Also, non-destructive techniques should be used to

characterize fracture zones, as destructive techniques such as drilling exploratory holes can become potential leakage pathways.

Some apparent differences are observed in the initiation and evolution of the EDZ at different projects: Stripa, Grimsel, Aspo, URL-WRA, and KURT. Perhaps these differences are due to the differences in the in situ stresses, tunnel shape, tunnel orientation, network of fractures in the rock mass, and excavation techniques.

2.2.2. Salt rocks

Bedded salt formations are also under consideration as radioactive waste repositories. The response of salt rock to a stress is very complex, but a substantial progress has been made to understand the behaviour of the rock. The creep phenomenon and plastic deformation are well known mechanical responses of salt formations. Due to these two characteristics, the prediction of the long-term performance of DGRs in salt rock could be challenging.

Salt formations have little flowing water that could transport the contaminants to the surface, they are relatively easy to be mined, have good heat conduction, and heal its own fractures due to its plastic behaviour (Langer, 1999). The big concern is the leakage that could occur to overlying strata if the repository is breached, and fractures may also exist in the salt inter-beds. In addition, radiolysis and corrosion of the waste package could generate a significant amount of gas, producing pressure buildups that might generate fractures and establish leakage pathways.

In 1957, the National Academy of Science, in the United States, declared that the most promising storage option of radioactive waste was in salt formations. Consequently, the Waste Isolation Pilot Project (WIPP) site in New Mexico has been studied extensively to assess its suitability as a repository for radioactive waste, and has been a licensed facility since 1999. Based on the results obtained from in situ gas permeability measurements, Stormont (1997) showed that damage within a salt formation could be detected and an EDZ could be characterized. However, in 2010 the American Administration ordered the closure of the WIPP site because of strong opposition in the state of New Mexico.

In Germany, the Gorleben salt dome has been investigated for its suitability to host at depths between 840 m and 1200 m a repository for all kinds of radioactive waste (Lidskog and Andersson, 2001). A wide spectrum of geotechnical and geophysical studies have been performed to measure the deformation behaviour of underground openings in the salt formation and to develop models. Also, in Germany, large-scale field experiments were performed in the Asse mine to investigate the coupled THMC processes induced in geological salt formations. It was found that the healing process caused a reduction in permeability of the formation from 10^{-16} m^2 to 10^{-18} m^2 in 90 years; not full healing yet to intact rock value of 10^{-20} m^2 . The healing process is the result of viscoplastic deformation and re-crystallization by presence of brine.

It is expected that radioactive waste in a salt rock repository will continue to emit radiation and thermal energy for decades after placement, resulting in a significant rise of the rock temperature. Liang et al. (2006) investigated the physical and mechanical characteristics of thenardite salt rock under different temperatures ranging from 20°C to 240°C . They concluded that the behaviour of the rock at high temperatures is still advantageous to the integrity of salt rock repository, assuring the safe isolation of nuclear wastes from the biosphere.

Long-term performance of caverns in salt formations, where deformation can continue to develop over time (creep) and cause expansion of the EDZ, is more of a concern for the safe disposal of radioactive waste than in crystalline rocks.

2.2.3. Sedimentary rocks

The current research is more focused on sedimentary formations. They exhibit a very favourable environment for disposal of radioactive waste because of their low permeability and diffusivity, combined with a possible self-healing of fractures and a good capacity to retard radionuclide transport. A big concern is that due to the excavations and the associated THMC disturbances, the favourable properties of such formations could change and the host rock could lose part of its barrier function. In addition, the associated wet-dry cycles as well as heating of the repository system are of concern and need a detailed evaluation to assess their influence on the long-term performance of the system.

The French Institute for Protection and Nuclear Safety has selected the Tournemire site for its own research program on deep geological disposal (Bonin, 1998). The objective of this project is to investigate the THMC processes and transport by diffusion and advection. The EDZ around the century-old tunnel, galleries excavated in 2003, and galleries excavated in 2008 were assessed (Rejeb and Cabrera, 2006). The Tournemire experiment shows that the excavation generates hydromechanical disturbances which can extend up to 6 times the mean radius of the gallery (2.5 m).

In central Japan, it was found that the size of the EDZ in the Neogene sedimentary rock at Tono mine is a function of the excavation method. The mechanical excavation method is more effective than blasting in limiting the extent of the EDZ and change in rock mass properties (Sato et al., 2000).

Due to its low permeability and the lack of tectonically disturbed zones, the Opalinus clay at Mont Terri in Switzerland has been selected as a potential host for radioactive waste (Martin et al. 2002, Bossart et al. 2004). Several experiments have been carried out to characterize the EDZ and it was found that the properties of the EDZ are dominated by stress-induced fractures and changing moisture conditions. Research shows that EDZ consists of an inner zone and an outer zone. The inner zone is made of an interconnected network of unsaturated fractures connected to the excavation. The connectivity of the induced fractures in the outer zone to the excavation is marginal.

Furthermore, at Mont Terri the effects of the ventilation and resaturation of the EDZ during the phase of repository operation and waste emplacement were assessed through in situ experiments. Numerical models were developed to simulate the short term evolution of the EDZ. Laboratory tests indicated that the behaviour of the Opalinus clay is highly nonlinear at low confining stresses (Corkum and Martin, 2007). The EDZ was found to be largely related to the existing stress field and rock anisotropy due to the presence of bedding planes (Popp et al., 2008).

Tang et al. (2002) investigated fluid flow, stress, and damage in sandstone. A good relationship was established between the loading and the flow properties along various portions of the complete stress-strain curve as shown in Fig. 2.2.

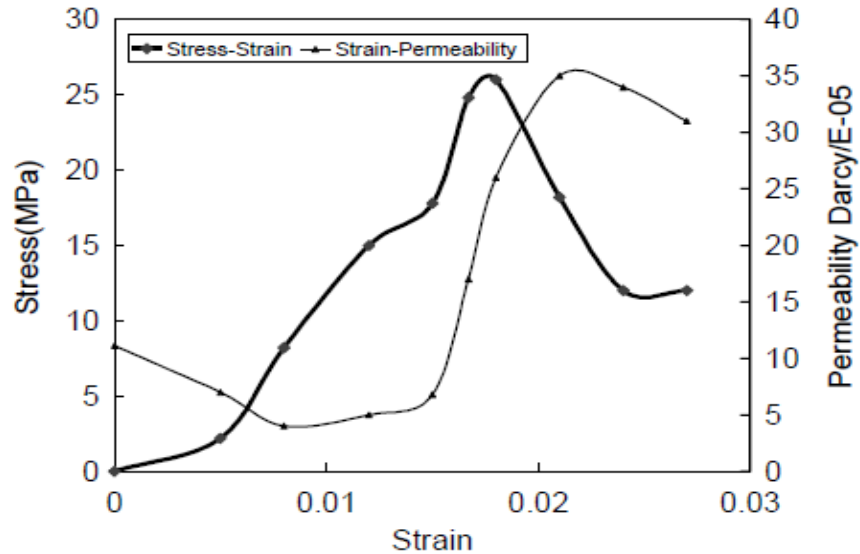


Figure 2.2. Results of triaxial tests on sandstone: relationships among stress, strain, and permeability (Tang et al., 2002).

A series of conferences, on ‘Clays in natural and engineered barriers for radioactive waste confinement’ were organized by ANDRA. They showed that the extent and shape of the EDZ in argillaceous rocks could be quite different from that of hard rock. The mechanical behaviour of argillaceous rocks exhibits time dependent effects. However, it seems that constitutive models applicable to hard rock could be adapted to argillaceous rocks, if one takes into account the additional time effects due to viscosity.

2.3. EDZ initiation and evolution

The EDZ is the perturbed rock zone around an underground opening following excavation. It may create a preferred pathway for radionuclide migration, particularly in low permeability rock mass. The behaviour of EDZ is mainly controlled by the initial stress field, THMC properties of the rock, mineralogical composition of the rock, existence of natural fracture zones, geometry and orientation of the opening, and the excavation method. Microcracks and fractures, and in

general a rearrangement of rock structures, can occur in this zone, resulting in an increase in permeability of the rock mainly through the excavation-induced fractures and microcracks.

Developing an understanding of the initiation and evolution of the EDZ is an important aspect of a DGR development. The strategy for assessing the role of the EDZ in the development of DGR is threefold (Perras et al., 2010); i) to understand possible role in creating discrete pathways along the excavated and backfilled openings for mass transport in rock, ii) to minimize damage extent through excavation methods and the geometry of the openings, and iii) to evaluate the performance of sealing methods.

It is important to note that the EDZ behaviour is a dynamic phenomenon, dependent on changing conditions such as moisture, temperature, and strength degradation (Millard et al., 2009). Self-healing of microcracks and fractures is itself a slow process, with characteristic times possibly of the order of 100 years or more (Bock et al., 2009). On top of these factors are the even longer term issues of chemical reactions and biological activities. All these issues make the problem a very challenging and rich field for scientific study.

Large-scale field tests, laboratory tests, and numerical analyses have been conducted to study the initiation and evolution of the EDZ. As an example, Rutqvist et al. (2009) applied different numerical approaches to model the evolution of the EDZ around a heated nuclear waste emplacement drift in a fractured rock. These approaches include boundary element, finite element, and finite difference methods. The numerical results indicate that thermally induced differential stresses near the top of the emplacement drift may cause progressive failure and permeability changes.

There are no clear rules for defining the boundary between the damaged, disturbed, and undisturbed rock mass properties. It is recognized that the EDZ is time-dependent process, and stress redistribution may lead to localized compression and swelling. For example, in the region of tangential stress concentration the maximum stress may exceed the previous overconsolidation stress and the material may behave more like normally consolidated sediment rather than an overconsolidated one (Blumling et al., 2007). In the unloaded areas, suction may

attract moisture and these materials may locally swell. Hence the EDZ may experience several complex processes and the material properties may significantly alter with time.

In open literature, it is suggested that the excavation damaged zone can be subdivided into three smaller zones. The type of rock involved plays a big role in the size of each zone (McEwen 2003). As an example, a schematic view of EDZ is presented in Fig. 2.3 (Rutqvist and Stephansson, 2003).

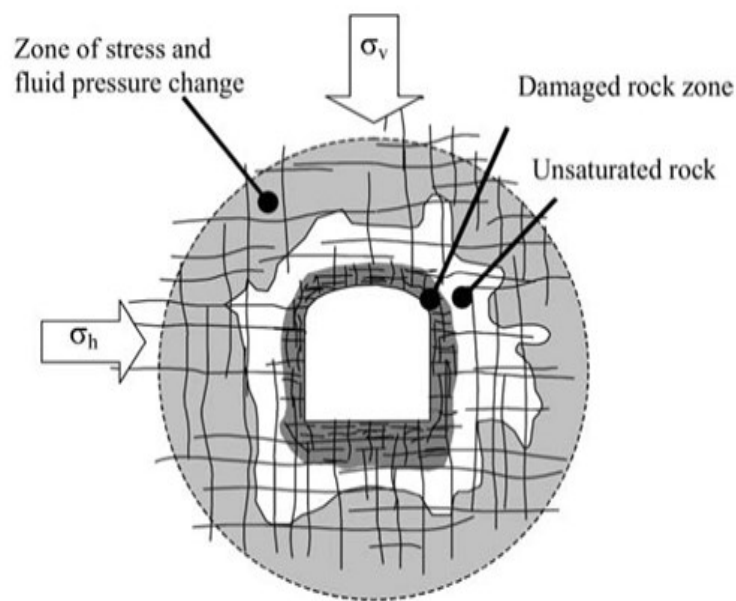


Figure 2.3. Schematic view of the EDZ around a tunnel (Rutqvist and Stephansson, 2003).

Nuclear Waste Management Organization (NWMO) of Canada subdivided disturbed rock into three different zones for a shaft, as shown in Fig. 2.4 (Highly damaged zone (HDZ), Excavation damaged zone (EDZ), and Excavation disturbed zone (EdZ)).

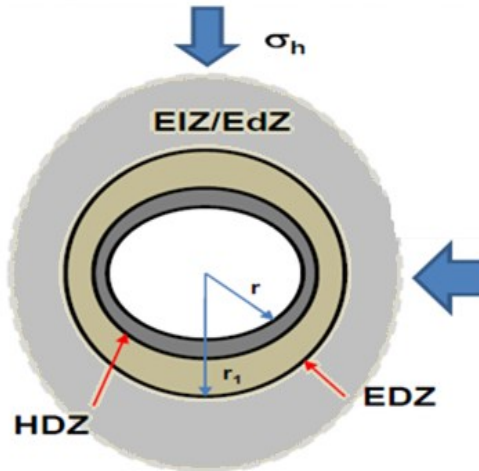


Figure 2.4. Schematic view of the EDZ for a shaft (NWMO of Canada, 2010).

There is no international consensus to relate the definition of the EDZ with time dependent processes, which are important for the long-term safety of a repository for geological disposal of radioactive waste (Tsang and Bernier, 2004).

Here, it is proposed that the initiation and evolution of the EDZ in sedimentary rocks can be structured into three phases: the initial phase (development of the DGR), the transient phase (operational phase of the DGR), and the long-term phase (closure of the DGR).

The *initial phase* corresponds to the initiation of the EDZ which alters the mechanical, hydraulic, thermal, and chemical properties of the rock (0~5 years). Predominant processes during this phase are the hydro-mechanical processes. A sketch of the initial phase is shown in Fig. 2.5. In this phase, the EDZ can be divided into three different subzones:

- *Highly damaged subzone (HDZ)*: Deformations, microcracks, fractures, spalling, or yielding of the material can occur in this subzone. Its permeability may increase significantly.
- *Excavation damaged subzone (edz)*: Hydromechanical and geochemical perturbations can develop in this zone. These perturbations may induce change in flow and transport properties.

- *Excavation disturbed subzone (EdZ)*: Possible hydromechanical and geochemical modifications can develop in this zone without changes in flow and transport properties.

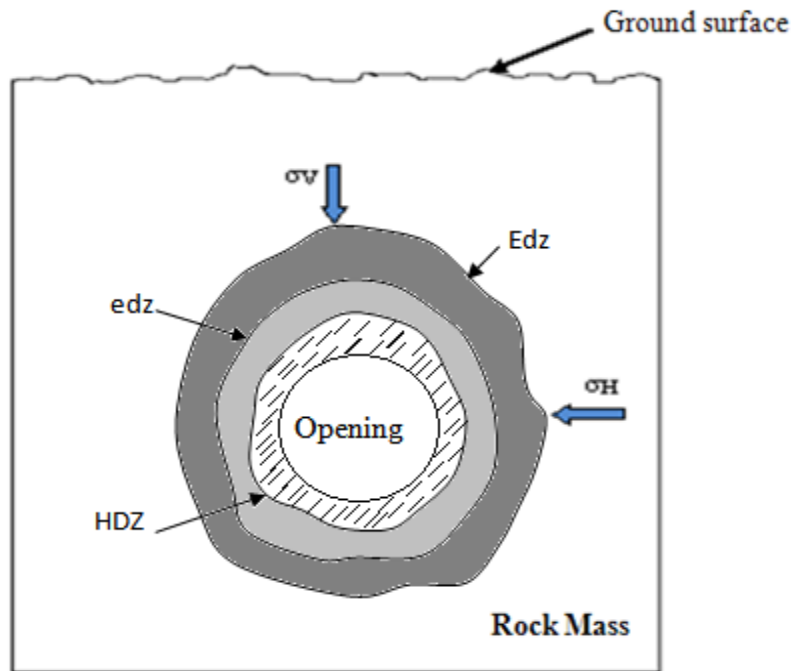


Figure 2.5. EDZ- Model geometry (initial phase – not to scale).

The *transient phase* corresponds to the operational phase of the DGR (5~30 years). During this phase, the access tunnels and emplacement rooms are ventilated and exposed to the atmosphere. Dry-wet cycles and chemical reactions can develop due to seasonal effects and ventilation. This can result in possible formation of new microscale and macroscale fractures, softening, and an increase in the size of the EDZ. Predominant processes during the transient phase include hydromechanical phenomena coupled with geochemical processes (principally due to oxidation) and strength degradation.

The *long-term phase* corresponds to the post-closure phase and gradual resaturation of the DGR. Resaturation can lead to a partial or complete closure of microcracks which were formed during the initial and transient phases. EDZ evolution during this phase will be mainly determined by the thermal impact on the hydraulic and mechanical properties of the rock, geochemical changes, gas-pressure build-up, strength degradation, and rock mass relaxation. The damage within the EDZ can be further accelerated by external loads such as glacial loading-unloading and seismic activities.

Figure 2.6 gives a summary of the factors relating to EDZ.

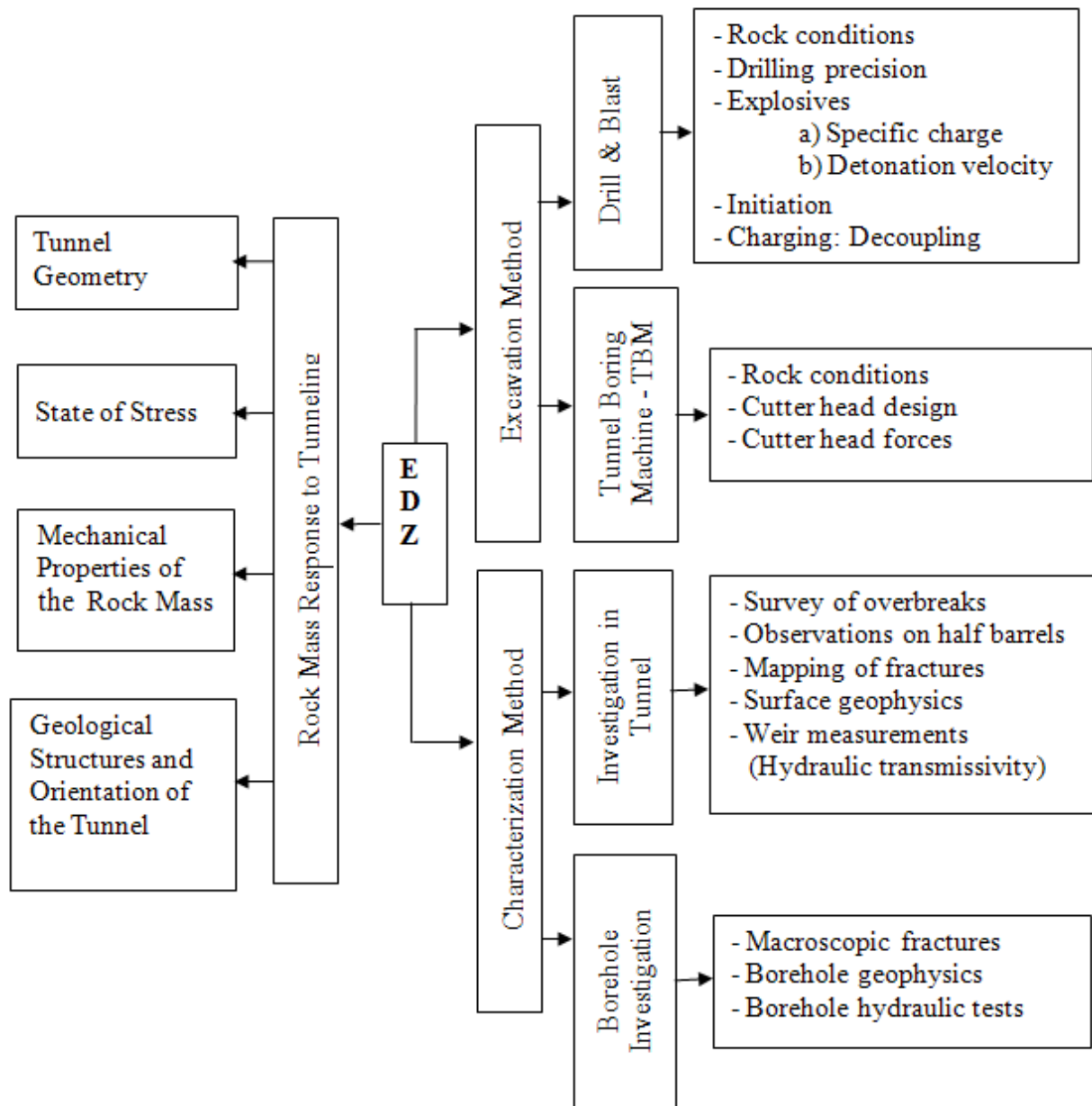


Figure 2.6. Summary of the factors relating the rock mass response to tunnelling, excavation method, and characterization methods (Hudson et al., 2009).

2.4. Self-sealing and sealing of natural and induced fractures in argillaceous formations

Depending on the mineralogical content, argillaceous rock may have some self-sealing capacity (Bock et al., 2009). After backfilling and closure of DGR, the resaturation process start and it may take years for the resaturation process to complete. Swelling of the argillaceous rock or backfill material and deposition of minerals can lead to partial or complete closure of the developed microcracks and fractures in the rock. This process of self-sealing may reduce the flow through the EDZ, and it can be an important aspect in characterizing potential host rock mass. The time required for self-sealing to occur in argillaceous formations depends mainly on its mineralogical composition and geologic history. In plastic clays, self-sealing can occur within a few months, whereas, in more indurated argillaceous formations self-sealing process may take years to occur.

At Mont Terri URL, a long-term plate loading experiment was carried out to simulate self-sealing effects of the EDZ (Buehler et al., 2001). The hydraulic conductivity of the fracture network was reduced significantly as a result of the mechanical loading of a tunnel wall. The RESEAL project was designed to demonstrate sealing techniques of the EDZ (Volckaert et al., 2000). It was observed during testing that the self-sealing of microcracks is very slow process and progressively taking place from within the host-rock towards the interface host rock–shaft lining. The RESEAL project also includes a borehole sealing experiment to test the bentonite–host rock interaction. The permeability of the seals was about a factor 10 lower than the permeability of the host rock formation and the behaviour of the seals under gas injection condition confirmed the feasibility of the sealing of a borehole.

In the context of evaluating the influence of EDZ on the geomechanical performance of compressed air energy storage (CAES) in lined rock caverns, Kim et al. (2013) showed the importance of minimizing the EDZ and implementing an efficient sealing technique. International research seems to indicate that the technology exists to seal the EDZ and the implications of the EDZ on the long-term safety of DGRs would be minimal (Nguyen, 2007).

2.5. Strength, deformation, and failure of a rock mass

2.5.1. In situ stresses, rock mass strength, and strength degradation

The design of underground tunnels requires the estimation of in situ stresses and rock mass strength. The in situ stress state varies considerably throughout a rock mass (Martin and Chandler, 1993). The measurement of in situ stresses is difficult, but it is essential to establish the magnitude and direction of the stresses with some reasonable degree of confidence. Although over-coring and hydraulic methods are used for estimating in situ stresses, the convergence method proved to be more reliable for estimating in situ stresses at the tunnel scale (Martin et al., 1997).

The strength of a rock mass is difficult to assess. Generally, it is estimated from laboratory tests, but the samples used in the tests are of relatively small size and do not represent the rock heterogeneity. In addition, the effect of potential discrete features or discontinuities is not considered. Therefore, translating laboratory strength to field strength can be problematic. The difficulty lies in extrapolating the laboratory strength, obtained from small samples of intact rock, to account for the presence of joints, fractures, and discontinuities. On the other hand, applying in situ testing approaches for determining the strength of a rock mass is not economically feasible. Back analysis of observed failures can provide good values for large scale rock mass strength, but this works in cases in which rock mass failure has occurred.

It is well known, deep underground tunnelling causes redistribution in stress field and, consequently, high stresses areas and damage may develop around tunnels. The degradation in the strength properties of the material may create instability problems, and the long-term monitoring of the variation in the strength properties is important to ensure the stability of underground structures.

2.5.2. Failure of a rock mass

Most rocks are classified as brittle materials. The elastic deformation of rocks is caused primarily by changes in intermolecular distances under applied loads. The inelastic deformation is caused by plastic deformations and the initiation and propagation of microcracks as this process can lead to the development of macroscopic fractures. The accumulation of microcracks throughout the rock mass is responsible for the nonlinear character of the mechanical response and ultimately for the failure of the rock.

The deformation and strength of a rock measured in a laboratory depend on several factors. Among them are loading/strain rate, confining pressure, temperature, and moisture content. The presence or absence of water affects the behaviour of the rock as water appears to weaken the chemical bonds and forms films around mineral grains along which slippage can take place. Dry rocks tend to behave in brittle manner, whereas wet rocks tend to behave in ductile manner.

Martin et al. (1999) indicated that at low in situ stress magnitudes, the failure process due to excavation in rock mass is controlled by the continuity and distribution of the natural fractures. However as in situ stress magnitudes increase, the failure process is dominated by new stress-induced fractures growing parallel to the excavation boundary.

Micro-structural changes within materials, undergoing deformation through compression, cause a reduction in various material mechanical properties (Fang and Harrison, 2002). Fig. 2.7 shows a typical stress–strain curve established by unloading and reloading a rock specimen during a triaxial compression test (Martin and Chandler, 1994). This figure clearly demonstrates that both strength and elastic modulus of the failing rock reduce gradually as fractures develop.

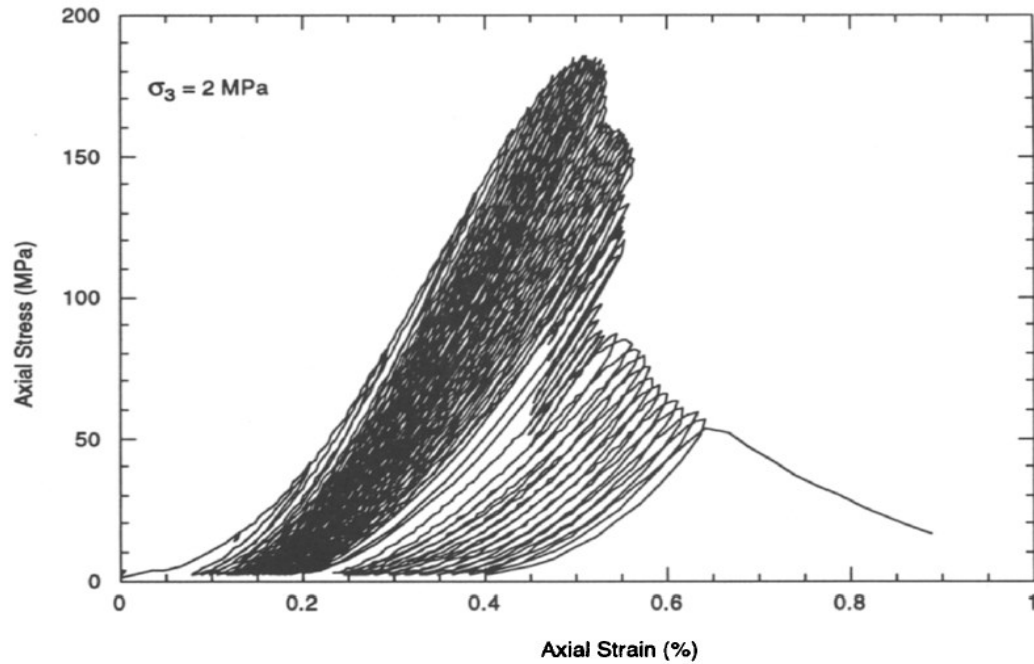


Figure 2.7. Example of the repeated loading and unloading (Martin and Chandler, 1994).

Martin and Chandler (1994) showed that the stress-strain relationship for a brittle material can be divided into five regions. Eberhardt et al. (1999) re-examined this idea as shown in Fig. 2.8. The region I represents the closure of existing microcracks. In region II, the rock is presumed to be a linear, homogeneous, elastic material. Region III marks the beginning of dilation (stable crack growth). Region IV represents the onset of unstable crack growth. The peak strength of the material marks the beginning of the region V. For Lac du Bonnet granite, Martin and Chandler (1994) showed that the initiation and propagation of new cracks start to occur at around 40% of the peak strength, and the unstable cracking starts to occur at around 80% of the peak strength.

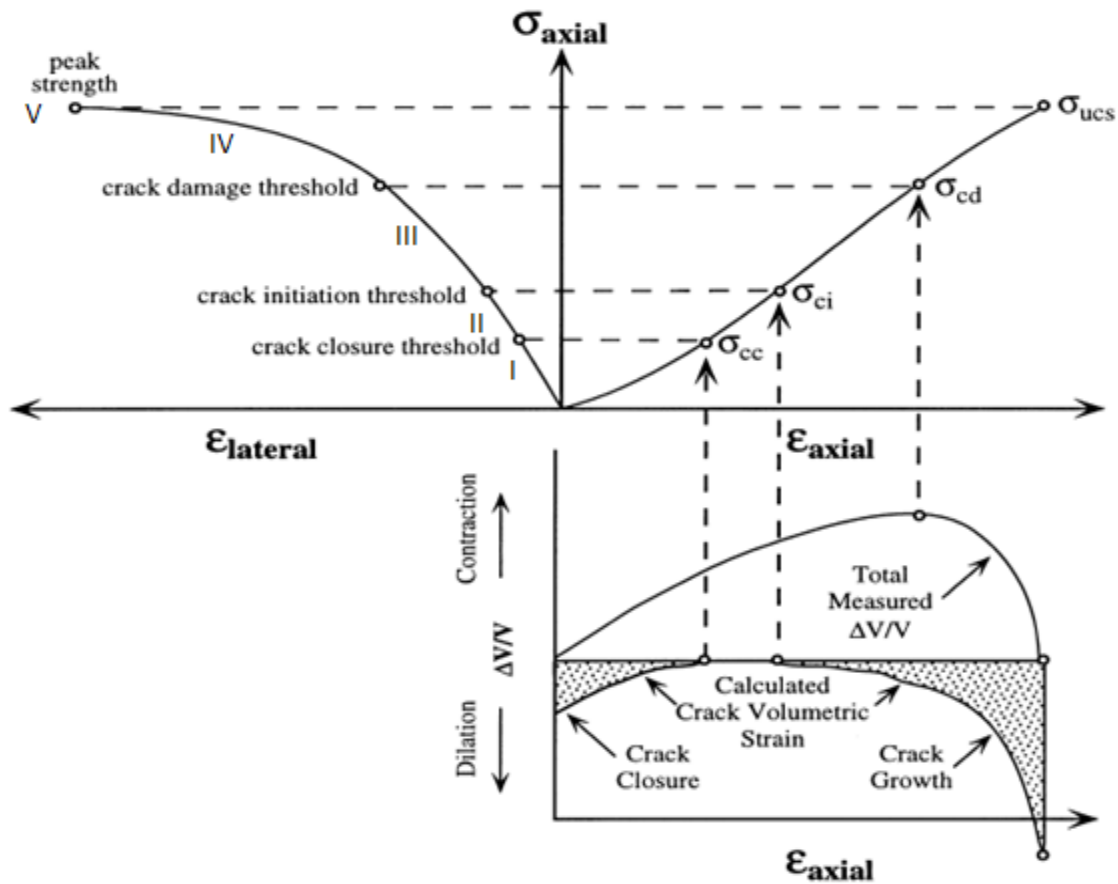


Figure 2.8. Stress-strain diagram showing the stages of crack development (Eberhardt et al., 1999).

In a mine-by-test experiment that was designed to examine the progressive nature of brittle failure process around an underground opening in granite, Martin et al. (1997) showed that the failure process, the formation of the v-shaped notch, is linked to the tunnel face advance. The notch formation is dominated by a slabbing process that occurs at scales ranging from the grain-size to several centimetres in thickness.

During brittle failure process, Martin et al. (1999) indicated that the cohesion and friction are not mobilized simultaneously. Around underground openings the brittle failure process is dominated by a loss of the intrinsic cohesion of the rock mass such that the frictional strength component can be ignored for estimating the depth of brittle failure.

2.5.3. Mechanisms that lead to failure around excavations

In brittle rocks, the failure of the rock within the EDZ can be classified into three classes; see for example: Martin et al. (2001) and Diederichs (2007).

Structurally controlled failure: In low-stress environment, loss of confinement and the existence of suitably oriented discontinuities can result in structurally controlled wedge-type failures and in general affect the tunnel roof.

Stress-controlled failure: The stress-controlled failure is associated with spalling and unravelling at low confinement stress or macroscopic shear failure at higher confining stress.

Stress and structural failure: This kind of failure can be caused by a slip on bedding planes.

2.5.4. Thermo-mechanical behaviour of a rock mass

The heat transfer within the rock mass can cause differential thermal expansion of constituent mineral grains, dehydration, and dissociation of certain minerals such as dolomite and calcite. One of the objectives of the European TIMODAZ project is to investigate the effect of thermal changes on the EDZ around nuclear waste disposal facilities (Charlier et al., 2008).

Wai and Lo (1982) investigated the strength of limestone at a temperature of about 84°C. The results indicate that the temperature had only a negligible effect on the strength of the rock, and the elastic modulus and Poisson's ratio were insensitive to temperature change. On the other hand, Masri (2010) investigated the effect of the temperature on the mechanical behaviour of the Tournemire argillite. The results indicate that increasing the temperature from 20°C to 250°C causes an increase in the Poisson's ratio of the rock and a decrease in the strength of the rock, elastic modulus, and the degree of anisotropy. The effect of temperature is also investigated by Takemura et al. (2006) by submerging sandstone in hot water for hundreds of days. They found

that this process causes some weathering of the rock, and also the strength and elastic properties of the rock are sensitive to temperature changes.

Here, only the low and intermediate level radioactive waste disposal is considered. These types of waste do not generate a lot of heat. But, it is important to consider the effect of the generated heat on the Hydraulic-Mechanical-Chemical (HMC) properties of the rock.

2.5.5. Inherent anisotropy in geomaterials

Many geomaterials display inherent anisotropy which is strongly linked with the microstructural arrangement. Such anisotropy may occur in granular media, but it is most typically associated with sedimentary rocks which are characterized by the existence of bedding planes. Some rocks of southern Ontario are anisotropic in both deformation and strength characteristics (Lo and Hori, 1979). Some geological formations behave more anisotropically than others due to well defined bedding planes (Chappell, 1990).

Angabini (2003) investigated the deformation behaviour of an anisotropic limestone. The rock samples showed some degree of anisotropy in their elastic properties, ultrasonic velocities, grain orientation, and gas diffusion. The anisotropy was caused by tectonic movements. Ajalloeian and Lashkaripour (2000) found that mudrocks exhibit some degree of anisotropy in compressive strength as a result of a partial alignment of plate-like clay minerals with small thickness. The mechanical behaviour of Opalinus clay is anisotropic (Salager et al., 2012).

Martin (1990) conducted an extensive program to characterize the far-field in situ stress states in granite. One of his findings is that major geological features, such as thrust faults, can act as in situ stress domain boundaries, and both the stress magnitudes and orientations can change when these boundaries are crossed. Also, stress relief in the form of microcracking can occur and can result in anisotropy and nonlinear rock response.

The presence of bedding planes within a rock formation can dominate the failure process depending on their spacing and their shear properties. The anisotropic strength can result in a less symmetrical EDZ.

2.5.6. Creep behaviour of a rock mass

Creep can damage rock fabric and may even lead to failure, and it is usually reported in terms of strain rate. Hard rock's do not display significant creep under loads present in conventional engineering problems. For these rocks, the dominant mechanism is microcrack generation and propagation along grain boundaries. A cumulative damaging process either stabilizes or accelerates depending on loading conditions. Salt rocks creep when subjected to any appreciable shear stress. They exhibit instantaneous strain on stress change followed by creep. Argillaceous formations may exhibit some creep especially along discontinuities and weakness planes which may be filled with different type of minerals.

2.5.7. Failure criteria

With a better understanding of the rock mass strength, it is possible to reduce instability problems that may occur due to underground excavations. The existing rock mass failure criteria are stress dependent and often include several parameters that describe the rock mass properties. These parameters are often based on laboratory tests and classification systems.

One of most frequently used failure criterion in rock mechanics is the Hoek-Brown failure criterion (Hoek et al., 2002). The Mohr-Coulomb failure criterion is also used in many rock mechanics projects. The $MSDP_u$ failure criterion is also applied in a limited number of rock mechanics projects. The formulations of the Hoek-Brown, Mohr-Coulomb, and $MSDP_u$ failure criteria are as follows.

Hoek-Brown failure criterion (HB):

The Hoek-Brown failure criterion is an empirical failure criterion, and it is used to establish the strength of a rock mass in terms of major and minor principal stresses. The increasing number of applications of the criterion to different rocks has necessitated some modifications, and new elements were introduced into the criterion. It is found that the criterion predicts strength envelopes which agree with values determined from laboratory triaxial tests of intact rock, and from observed failures in rock mass. The Generalized Hoek-Brown failure criterion (Hoek et al., 2002) for a rock mass can be expressed as follows.

$$\sigma'_1 = \sigma'_3 + \sigma_{cs} \left(m_b \frac{\sigma'_3}{\sigma_{cs}} + s \right)^a \quad [2.1]$$

where: σ'_1 is the maximum effective stress at failure; σ'_3 is the minimum effective stress at failure; m_b is a reduced value (for the rock mass) of the material constant m_i (for the intact rock); s and a are constants which depend on the characteristics of the rock mass; σ_{cs} is the uniaxial compressive strength of the intact rock samples.

$$m_b = m_i e^{\frac{GSI-100}{28-14D}} \quad [2.2]$$

$$s = e^{\frac{GSI-100}{9-3D}} \quad [2.3]$$

$$a = 0.5 + \frac{1}{6} \left(e^{\frac{-GSI}{15}} - e^{\frac{-20}{3}} \right) \quad [2.4]$$

where GSI is the geological strength index (relates the failure criterion to geological observations in the field), m_i is a material constant for the intact rock, and D is a disturbance factor which depends on the degree of disturbance to which the rock mass has been subjected by blast damage and/or stress relaxation (It varies from 0 for undisturbed in situ rock mass to 1 for very disturbed rock mass).

Estimation of the parameters for the generalized Hoek-Brow failure criterion:

- *Uniaxial compressive strength (σ_{cs}):* It is estimated from the results of uniaxial tests on intact rock specimens.
- *Geological strength index (GSI):* The GSI is related to the quality of the rock. It is estimated from a chart and depends on rock structure and block surface conditions. As an example, for the Cobourg formation, a value between 65 and 85 can be used (OPG Report-2, 2008).
- *Disturbance factor (D):* It is related to the amount of damage that can be caused by the excavation technique to the rock. As an example, for TBM the value of D is equal to 0.
- *Intact rock parameter (m_i):* The m_i depends on the type of the rock. As an example, the values of m_i are 8 and 17 for shale and sandstone, respectively.

Mohr-Coulomb failure criterion (MC):

The Mohr–Coulomb failure criterion is a set of linear equations in principal stress space describing the conditions for which a rock mass will fail, with any effect from the intermediate principal stress σ_2 being neglected. It can be expressed in a general form as follows.

$$\frac{\bar{\sigma}}{g(\beta)} - \alpha \sigma_m - \eta = 0 \quad [2.5]$$

where: $g(\beta) = \frac{3 - \sin \varphi}{2\sqrt{3} \cos \beta - 2 \sin \beta \sin \varphi}$; $\alpha = \frac{2\sqrt{3} \sin \varphi}{3 - \sin \varphi}$; $\eta = \frac{2\sqrt{3} c \cos \varphi}{3 - \sin \varphi}$; $\bar{\sigma} = \sqrt{J_2}$; $\sigma_m = -\frac{1}{3} I_1$, and $\beta = \frac{1}{3} \arcsin\left(\frac{3\sqrt{3} J_3}{2\sqrt{J_2^3}}\right)$.

β is the Lode angle, c and φ are the strength parameters, I_1 is the first invariant of the stress tensor σ_{ij} , and J_2 is the second invariant of the stress deviator s_{ij} .

MSDPu failure criterion:

The MSDPu (Mises-Schleicher and Drucker-Prager unified) failure criterion is proposed by Li et al. (2005). It can provide a practical framework to account for the inelastic response of different types of geomaterials. The yield function is expressed in terms of commonly used stress invariants.

$$F = \sqrt{J_2} - F_\pi \sqrt{\alpha^2(I_1^2 - 2a_1 I_1) + a_2^2} \quad [2.6]$$

$$\alpha = \frac{2 \sin \varphi_r}{\sqrt{3}(3 - \sin \varphi_r)} \quad [2.7]$$

$$k = \frac{\sigma_{cs} - \sigma_t}{12 \alpha} + \frac{\alpha \sigma_{cs} \sigma_t}{\sigma_{cs} - \sigma_t} \quad [2.8]$$

$$a_1 = \frac{\sigma_{cs} - \sigma_t}{2} - \frac{\sigma_{cs}^2 - (\frac{\sigma_t}{b})^2}{6 \alpha^2 (\sigma_{cs} + \sigma_t)} \quad [2.9]$$

$$a_2 = \sqrt{\left(\frac{\sigma_{cs}^2 + (\frac{\sigma_t}{b})^2}{3(\sigma_{cs} + \sigma_t)} - \alpha^2 \right) \sigma_{cs} \sigma_t} \quad [2.10]$$

$$F_\pi = \frac{(1-b^2)f_\beta + (2b-1)\sqrt{(1-b^2)f_\beta^2 + 5b^2 - 4b}}{(1-b^2)f_\beta^2 + (1-2b)^2} \quad [2.11]$$

$$f_\beta = \sqrt{3} \cos \beta - \sin \beta \quad [2.12]$$

where σ_{cs} is the uniaxial compressive strength, σ_t is the uniaxial tensile strength, φ_r is the angle of residual strength, b is a parameter accounting for the effect of the Lode angle β , I_1 is the first invariant of the stress tensor σ_{ij} , J_2 is the second invariant of the stress deviator s_{ij} , and J_3 is the third invariant of the stress deviator s_{ij} .

The mechanical properties of many rock formations are anisotropic. A couple of attempts have been made to develop an anisotropic failure criterion for rock formations; see for example, Pietruszczak (2010). However, so far there is no a widely accepted anisotropic failure criterion that can be applied for anisotropic rock formations (Gao et al., 2010). An isotropic failure criterion could be applied for sedimentary rocks and the effect of inherent anisotropy could be minimized by selecting appropriate strength parameters (directional-dependent) of the rock.

2.6. Modeling of the mechanical behaviour of sedimentary rocks

Sedimentary rocks occur in layers separated by bedding planes. Experimental evidence indicates that most sedimentary rocks display an anisotropy of strength and deformation (Nasseri et al., 2003; Colak and Unlu, 2004; Karakul et al., 2010) and the degree of anisotropy varies from one sedimentary rock to another. As an example, Sagar et al. (2011) indicates that the anisotropic mechanical characteristics of Opalinus clay may be attributed to two factors: (i) a structural anisotropy attributable to the fabric, and (ii) an inherent anisotropy attributable to the stress history.

The mechanical behaviour of sedimentary rocks is anisotropic and nonlinear. In recent decades, different approaches have been proposed to describe the mechanical behaviour of sedimentary rocks:

- The first approach is to vary certain parameters in an arbitrary manner according to the loading orientation angle (Mc Lamore and Gray, 1967; Jaeger, 1960). This approach suffers from relating different parameters to the microstructure of the rock.
- The second approach is to use tensors of different orders (Boehler, 1978; Cazacu, 1995). This approach requires a large number of experimental data in order to estimate the parameters needed for modeling.
- The third approach is to use the microstructure tensor (Kanatani, 1984; Yang et al., 2001; Pietruszczak and Mroz, 2001; Chen et al., 2010; Chen et al., 2012). This approach has limitations in terms of practical applications.

Numerical methods, damage mechanics concepts, and the proposed approaches (above) have been combined and used to model the mechanical behaviour of sedimentary rocks. In many cases, the failure mechanism of sedimentary rocks is interpreted in terms of the loading orientation angle (Yasar, 2001; Zhang et al., 2010).

A combined finite/discrete element method was used to develop a numerical model for Opalinus clay that accounts for the mechanical anisotropy (Lisjak et al., 2014). The finite element method was used to simulate the elastic part (transversely isotropic elasticity). A fracture model (finite discrete elements) was used to simulate microcracks and fractures. The modeling approach seems to be able to capture both intra-bedding plane extensional cracks and bedding plane shear failure mechanisms. However, the model has still major limitations such as dimensional limitations and time-dependent behaviour. The anisotropic behaviour of Opalinus clay was also analyzed using the theory of plastic multi-mechanisms (Sagar et al., 2012). It was found that multi-mechanism plasticity is a suitable constitutive tool for the interpretation of the mechanical anisotropy for this shale.

Coupled elastoplastic damage models for anisotropic rocks could be developed within the framework of thermodynamics (Shao et al., 2006; Zhu et al., 2008; Chen et al., 2012). In thermodynamic theory, the variation of damage is assumed to be related to the variation of elastic and plastic strains. Viscoplasticity also could be used to describe the time-dependent behaviour of sedimentary rocks (Fossum and Brannon, 2006). An elastoviscoplastic constitutive model with strain softening is developed for soft sedimentary rock using an equation for sub-loading yield surface (Zhang et al., 2005).

Jia et al. (2007) developed an elastoplastic damage model for describing the hydromechanical behaviour of saturated and unsaturated argillite. The proposed model was formulated within the framework of poroplasticity and continuum damage mechanics. The model contains twelve parameters and a large number of laboratory experiments are needed to determine the parameters. Chen et al. (2010) also developed a coupled elastoplastic damage model for the description of anisotropic sedimentary rocks. The model is based on the concept of

microstructure tensor. A large number of experiments are required to identify the parameters of the model: elastic parameters, plastic parameters, and damage parameters.

Other techniques also have been used to model the mechanical behaviour of sedimentary rocks. You et al. (2011) developed a two-phase Distinct Lattice Spring Model (DLSM) for transverse isotropic materials, in which the layer structure of the anisotropic materials is explicitly represented. Jin et al. (2003) developed a depositional model that reconstructs numerically the geometrical structure and mechanical properties of natural sedimentary rocks in two and three dimensions. The model reproduces the mechanical properties of the rock.

2.7. Geological disposal of radioactive waste in Canada

2.7.1. Background

In Canada, the long term management of the used nuclear fuel is under the authority of the Nuclear Fuel Waste Act. The Nuclear Waste Management Organization (NWMO) was established in 2002 in accordance with the Act to assume responsibility for the long-term management of Canada's used nuclear fuel. The selected approach for the long-term management of used fuel is Adaptive Phased Management. In its report to the government, NWMO is considering a deep geological repository in either granitic rocks or sedimentary rocks as potential hosts for disposal of the used nuclear fuel.

Started in 1970s, the URL project was developed by Atomic Energy of Canada Limited (AECL) in Canadian Shield in Winnipeg for qualifying the granite of the Lac Du Bonnet to dispose the used nuclear fuel. The environmental assessment was reviewed by Seaborn Panel, a Federal Environmental Assessment Review Panel later in 1980s to 1990s. In 1998, the Panel concluded that the AECL concept is acceptable technically but broad public supports have not been demonstrated. Currently, the Ontario Power Generation (OPG) is considering constructing a Deep Geologic Repository (DGR) for the long-term management of its low and intermediate level waste (LILW) in sedimentary rock formations of Southern Ontario at its Bruce nuclear power plant site. The Environment Impact Statement has been submitted to federal governments for review. The NWMO is under a site selection process to identify an informed and willing community to host facilities for the management of Canada's used nuclear fuel for the long term in either granitic rocks or sedimentary rocks.

Research in Canada on geological disposal for used nuclear fuel is in a full swing at nuclear sector to bridge the knowledge gaps by funding research and characterization programs and by initiating collaborative works with the European organizations such as ANDRA in France and NAGRA in Switzerland and others. Here, we review only the design of the DGR for LILW in Ontario and the geology of the Bruce site. Figure 2.9 describes the conceptual design of the DGR which consists of a series of emplacement rooms, access tunnels, and shafts. The emplacement

rooms will be excavated at a depth of 683 m within a limestone formation. The limestone is characterized with relatively high-strength and low permeability (order of 10^{-21} m²).

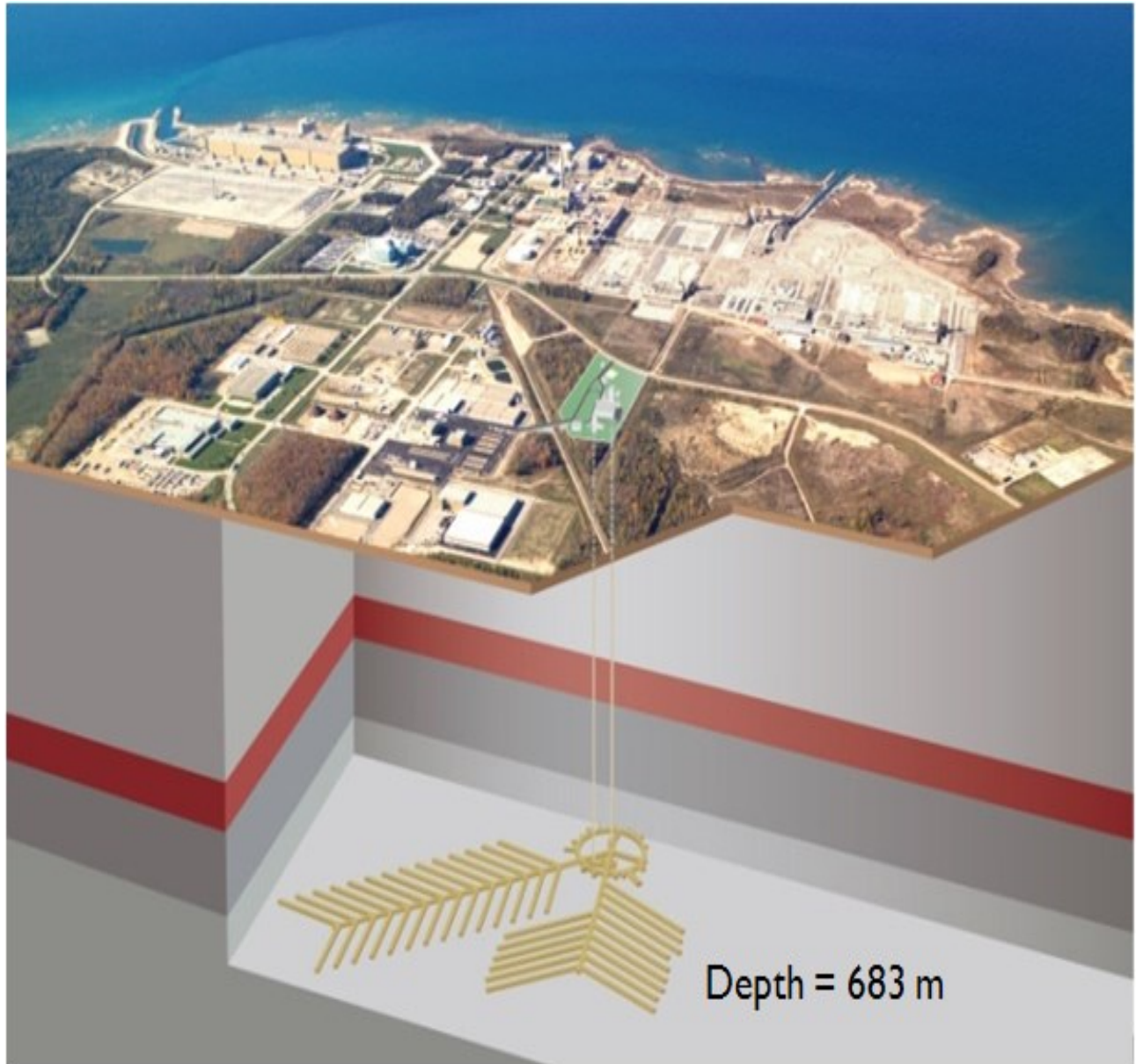


Figure 2.9. Preliminary design of the proposed DGR (OPG Report-1, 2008).

The long term performance of the geological disposal relies on robustness of multiple barriers (natural and engineered). For sedimentary rock formations of the Southern Ontario, the following attributes could provide confidence on its long term efficiency: 1) homogeneity and sufficient lateral and vertical extent of the rock mass; 2) transport of solutes is dominated by diffusion processes; and 3) stability of geosphere for the past millions of years.

2.7.2. The disposal concept for used nuclear fuel

The disposal concept combines both natural and engineered barriers, resulting in an integrated system of multiple barriers. The primary purpose of the system is isolation and containment of the radioactive nuclear wastes for hundreds of thousands of years or more. The main engineered and natural barriers in and around a DGR for the disposal of used nuclear fuel can be briefly described as follows.

- *Waste form:* The wastes will be incorporated in materials which will immobilize and resist dissolution in groundwater.
- *Containers:* The waste form will be encapsulated in containers made of a material that is resistant to degradation by groundwater.
- *Backfill material:* The containers will be surrounded by a backfill material chosen to inhibit groundwater circulation around the containers and to retard the movement of radionuclide that may become dissolved in the groundwater.
- *Natural geological formation:* The performance of the natural geological formation as a barrier depends on its permeability and groundwater regime. To minimize the groundwater flow rates, a desirable characteristic of the host rock formation would be the scarcity of joints. Another characteristic of the formation is the residual heat generated by the waste should be dissipated with minimal effect on the effectiveness of all barriers. The backfilling of tunnels and sealing of shafts should be designed to enhance the effectiveness of all barriers and retard the ingress of water.

The geological formation is the physical barrier to intrusion and radionuclide migration. Once radioactive materials are leached from the waste and enter the rock-groundwater system, retardation processes will delay the radionuclide migration, allowing longer periods for radioactive decay. The radioactive decay, adsorption, dilution and physical dispersion of the radionuclide throughout the rock mass will contribute to limiting the quantity and concentration of radionuclide to an acceptable level which may eventually reach the biosphere.

2.7.3. Geology of the Bruce site for OPG DGR for LILW

The proposed DGR for LILW will be developed within Ordovician sedimentary rock formations. These formations were formed approximately from 430 to 500 Ma ago, and perturbed by nine glacial events during the latter half of the Pleistocene. The DGR will be surrounded and overlain by multiple layers of low permeability sedimentary rocks. The layers of sedimentary rocks were formed within the depression of the Michigan basin. The groundwater at the level of the repository is saline (>100 g/L). A cross-section of the Michigan basin showing the rock formations is given in Fig. 2.10. The types of rock include carbonates, shale, evaporate and sandstone which are located above the Pre-Cambrian crystalline basement rock.

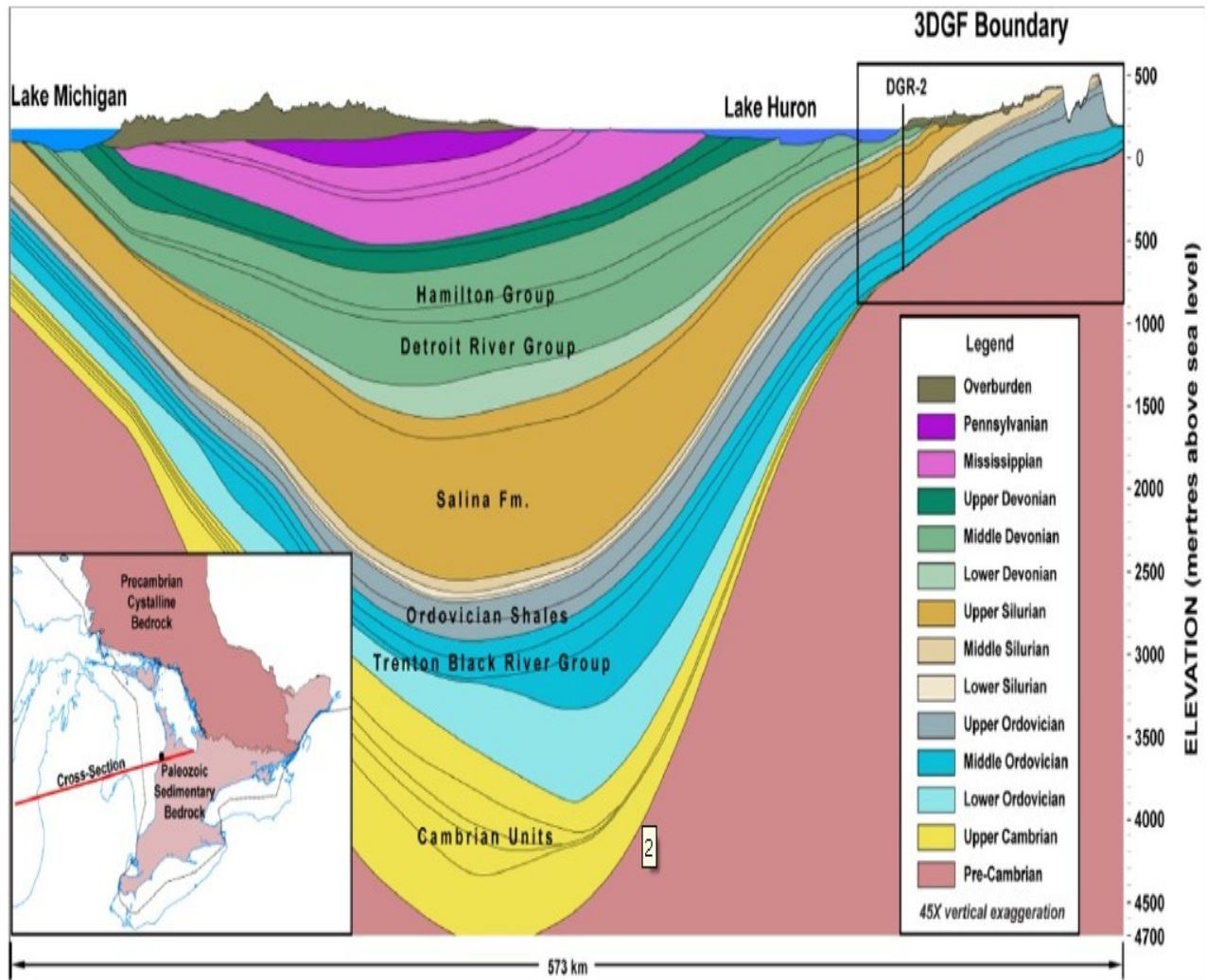


Figure 2.10. Rock formations of Michigan basin (OPG Report-1, 2008).

Field investigation has been done and several boreholes are drilled. Rock formations observed in borehole DGR2 at the Bruce site are shown in Fig. 2.11. The repository will be located in the Cobourg limestone.

Ontario is located in the Mid-Plate stress province and is characterized by high horizontal compressive stresses (Adams and Bell, 1991). This behaviour is due mainly to regional tectonic activities and past glaciations. The existence of high horizontal stresses in many sedimentary rocks of Ontario have been well documented; e.g., Lo (1978) and Lee (1981). Based on the regional data (Gartner Lee Limited, 2008), at the level of the repository the ratio $\sigma_H/\sigma_v = 1.7$ to

2.5, while the ratio $\sigma_h/\sigma_v = 1.0$ to 1.2 (Fig. 2.12). The orientation of the maximum horizontal stress in the Michigan Basin appears to be in a NE direction.

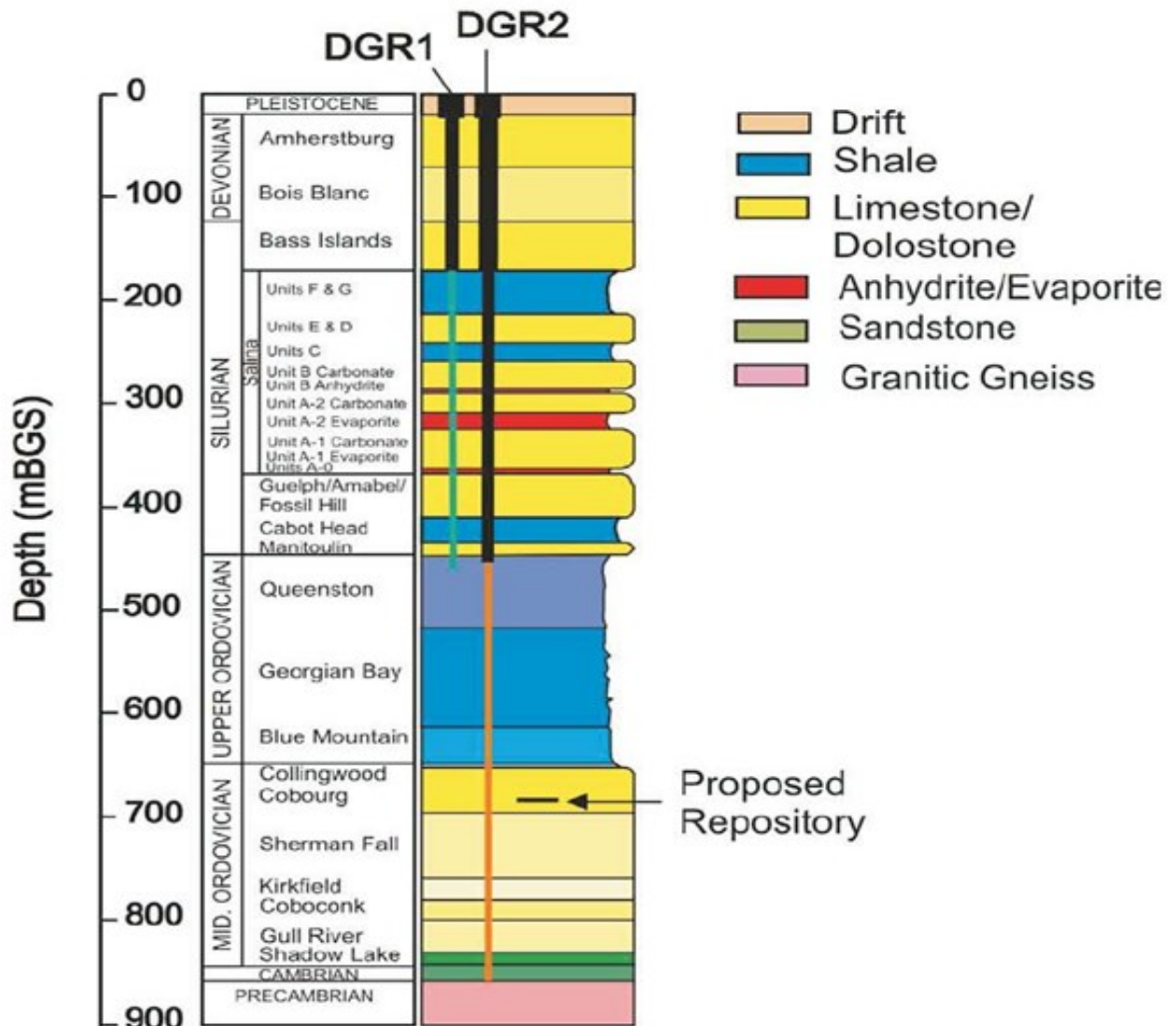


Figure 2.11. Rock formations observed in borehole DGR2 (OPG Report-1, 2008).

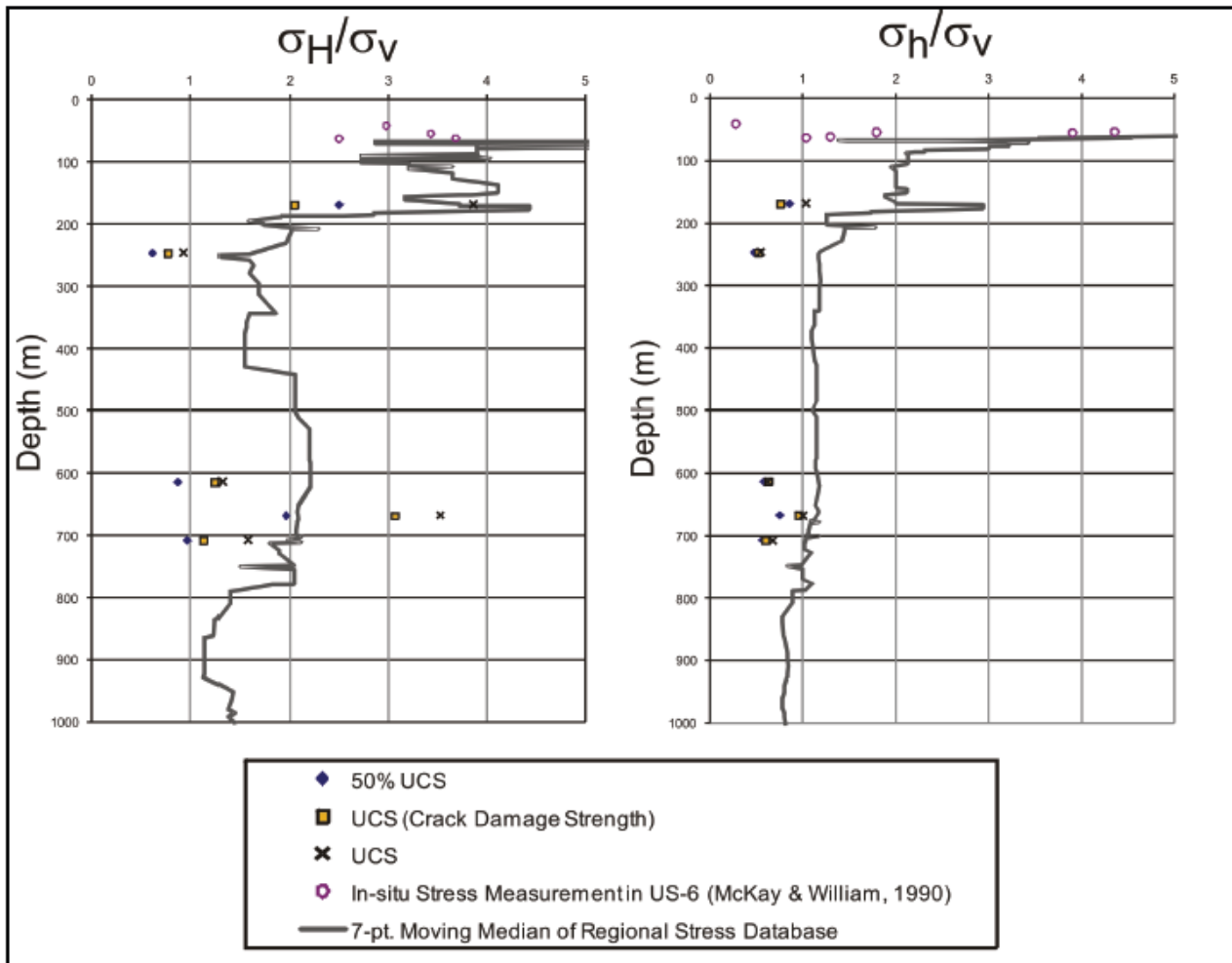


Figure 2.12. In-situ stress ratios based on far-field regional data using the moving median technique (from OPG Report 2008e).

CHAPTER 3

POROELASTICITY, ELASTOPLASTICITY, CONTINUUM DAMAGE MECHANICS, AND FINITE ELEMENT CODES

3.1. Background

Sedimentary formations can be treated as porous media. Poroelasticity formulations, elastoplasticity formulations, and continuum damage mechanics concepts can be used together to describe the interaction between fluid motion and deformations within such media. This Chapter summarizes such formulations and concepts. It also describes a couple of Finite Element codes that can be used for simulating the mechanical and the hydromechanical behaviour of sedimentary formations within the EDZ.

Numerical simulation of the mechanical behaviour of sedimentary formations can be achieved by combining continuum mechanics concepts, continuum damage mechanics concepts, and the theory of elastoplasticity. Numerical simulation of the hydromechanical behaviour of sedimentary formations can be achieved by combining poroelasticity formulations, continuum damage mechanics concepts, and the theory of elastoplasticity.

3.2. Poroelasticity formulations

This section summarizes poroelasticity formulations. More details can be found in Nguyen (1995).

3.2.1. The continuum representation of a porous medium

The laws of continuum mechanics are expressed on a large scale: macroscopic scale. The concept of Representative Elementary Volume (REV) (Bear, 1972) is adopted to represent the properties of porous medium. A REV is a finite volume in a porous medium that contains a number of pores and solid grains. It surrounds a point mathematically defined in that medium. The mean properties in the REV are assigned to the mathematical point. With the introduction of the REV, a property such as porosity can be easily defined.

The REV should be large enough to contain a sufficient number of pores and solid grains so that the mean value of a given property has a statistical significance. On the other hand, the REV should be sufficiently small so that the variation of such property from one domain to the next may be approximated by continuous functions.

3.2.2. Effective stress concept

The application of an external load to a porous medium causes the development of a total stress field σ_{ij} . This total stress can be decomposed into one component borne by the fluid and another component borne by the solid material (effective stress). This concept was proposed in 1923 by Terzaghi. In 1941, Biot extended this concept to include finite compressibility's of the fluid and the solid material. Biot's equations were developed to solve three dimensional problems.

The fluid is assumed to be incapable of resisting any shear, thus the stress component borne by the fluid is a scalar (hydrostatic fluid pressure). The total stress can be expressed in terms of the effective stress and the fluid pressure as follows.

$$\sigma_{ij} = \sigma'_{ij} + \alpha_B \delta_{ij} p \quad [3.1]$$

where σ_{ij} is the total stress tensor, σ'_{ij} is the effective stress tensor, p is the fluid pressure, δ_{ij} is the Kronecker's delta (=1 if $i=j$; =0 if $i \neq j$), and α_B is Biot coefficient (Biot, 1941).

For soils the value of α_B is equal to unity. For rocks the value of α_B is less than unity, e.g., for granite of the Canadian Shield $\alpha_B=0.6$.

The total stress can result in the deformation of the solid skeleton and the pore fluid. The resulting strain can be divided into two parts; the strain associated with the action of p and the strain associated with the action of σ'_{ij} . The porewater pressure p is hydrostatic and it can induce only a volumetric strain $\epsilon_{kk} = \frac{p}{K_s}$, where K_s represent the bulk modulus of solid grains. The action of the effective stress can result in strain component ϵ'_{ij} .

The total strain is the sum of the above two strains:

$$\varepsilon_{ij} = \varepsilon'_{ij} + \varepsilon_{kk} \quad [3.2]$$

For incompressible solids the K_s is large and the second component of the total strain can be neglected. Hence, the deformation behaviour of the solid matrix is only dependent on the effective stress.

The validity and limits of the effective stress concept in Geomechanics have been discussed by many authors; see for example, Skempton (1960), Oka (1988), and Oka (1996). But, it is shown that the effective stress concept is applicable to soft rocks (Oka, 1996).

3.2.3. Darcy's law

Darcy's law describes fluid movement through interstices in a porous medium with reasonable accuracy. The hydraulic potential field that drives the fluid through the medium can be visualized by considering the difference in both pressure and elevation potential from the start to the end points of the flow lines. In the context of the disposal of nuclear waste in geological formations, the Darcy's law can be used to describe the transient water inflow into the emplacement rooms, shafts, and access tunnels. It can also be used to predict the resaturation process which will take place after closure of the DGR.

Darcy's law is a phenomenological constitutive equation:

$$v_i = -\frac{k_{ij}}{\omega \mu} (\nabla p + \rho_f g_j) \quad [3.3]$$

where v is the Darcy velocity or specific discharge vector, k_{ij} is the permeability tensor of the medium, ω is the porosity of the medium, μ is the viscosity of the fluid, p is the fluid pressure, ρ_f is the density of the fluid, g_j is the gravitational acceleration, and ∇ is the gradient operator.

3.2.4. Equilibrium equations

The quasi-static equilibrium equation of the system can be expressed as follows.

$$\frac{\partial \sigma_{ij}}{\partial x_j} + F_i = 0 \quad [3.4]$$

where σ_{ij} is the total stress tensor and F_i is the volumetric body force vector.

By substituting Equation [3.1] into the Equation [3.4], we obtain:

$$\frac{\partial \sigma'_{ij}}{\partial x_j} + \alpha_B \frac{\partial p}{\partial x_j} + F_i = 0 \quad [3.5]$$

3.2.5. Equation of fluid mass conservation

Let's consider a small element of volume V of the porous medium. The net flux of the fluid mass through the boundary of the volume is equal to the rate of fluid mass accumulation. This will lead to the following equation:

$$-\frac{\partial}{\partial x_i} (\rho_f \omega v) = \frac{\partial}{\partial t} (\omega \rho_f) \quad [3.6]$$

where v is the velocity of the fluid.

If the density of fluid is approximately constant, the left hand side of the Equation [3.6] can be replaced by:

$$\frac{\partial}{\partial x_i} (\rho_f \omega v) \approx \rho_f \frac{\partial}{\partial x_i} (\omega v) \quad [3.7]$$

The right hand side of the Equation [3.6] can be rewritten as follows; (see Nguyen (1995) for more details):

$$\frac{\partial}{\partial t} (\omega \rho_f) = \rho_f \left[- \left(\frac{\omega}{K_f} - \frac{\omega}{K_s} + \frac{\alpha}{K_s} \right) \frac{\partial p}{\partial t} + \alpha_B \frac{\partial \varepsilon_{kk}}{\partial t} \right] \quad [3.8]$$

where K_f is the bulk modulus of the fluid, K_s is the bulk modulus of the solid, α_B is the Biot coefficient, and ε_{kk} is the volumetric strain.

Using the Equation [3.3], Equation [3.7], and Equation [3.8], the equation of fluid mass conservation can be expressed as follows:

$$\frac{\partial}{\partial x_i} \left(\frac{k_{ij}}{\mu} \left(\frac{\partial p}{\partial x_j} + \rho_f g_j \right) \right) - \left(\frac{\omega}{K_f} - \frac{\omega}{K_s} + \frac{\alpha}{K_s} \right) \frac{\partial p}{\partial t} + \alpha_B \frac{\partial}{\partial t} \left(\frac{\partial u_i}{\partial x_i} \right) = 0 \quad [3.9]$$

Equation [3.5] and Equation [3.9] form the set of governing equations that can be used to simulate hydromechanical processes associated with the development of a DGR within a sedimentary formation.

3.3. Theory of elastoplasticity

The plasticity theory includes the definition of the yield function (f), a flow rule, a hardening rule, and decomposition of the total strain into the elastic part and plastic part. The theory has been applied to different materials including soils, concrete, and rocks. Applying an associated flow rule, some brittle rocks show dilatancy that is significantly higher than the measured in the experiments (Paterson and Wong, 2005). Therefore, non-associated plasticity is recommended when modeling the behavior of friction materials (Ottosen and Ristinmaa, 2005).

The plastic strain occurs when the yield conditions are satisfied. Usually, the yield function f is expressed as a function of stress invariants, hardening parameter, and strength parameters. The behaviour of the material is elastic if $f < 0$. However, in actuality $f > 0$ is not allowed and requires calculation of the amount of plastic flow and how the hardening level evolves such that $f = 0$ is achieved. This is accomplished by solving for a consistency parameter, which allows a means for determining the level of plastic flow and hardening such that the condition $f = 0$ is satisfied. The f can be expressed in a general form as follows.

$$f = f(I_1, I_2, I_3, H, c, \varphi) \quad [3.10]$$

where H is the hardening parameter; c and ϕ are the strength parameters; I_1 , I_2 , and I_3 are the invariants of the stress tensor σ_{ij} ,

One of the concepts of elastoplasticity is that the total strain consists of two components: elastic strain and plastic strain.

$$d\varepsilon_{ij} = d\varepsilon_{ij}^e + d\varepsilon_{ij}^p \quad [3.11]$$

The elastic strain is the strain that is removed during unloading. The plastic strain is permanent strain that remains after unloading. Any time plastic strains are taking place it follows that plastic flow is taking place. Yield stress changes with the development of hardening.

The incremental form is always used to model nonlinear material behaviour. At the beginning of each increment of strain or stress, it is not known whether the new level of strain or stress causes yielding or not. For this reason it is initially assumed that the incremental strain or stress is elastic. In the elastoplasticity algorithms, the resulting new stress level is termed a trial stress. This trial stress is plugged into the yield condition f . The value of f will indicate whether the material yields or not. If the yielding occurs, the amount of plastic strain is calculated.

The incremental stress-strain relationship can be expressed as follows:

$$d\sigma_{ij} = D_{ijkl}^{ep} d\varepsilon_{kl} \quad [3.12]$$

where D_{ijkl}^{ep} is the elastoplastic constitutive matrix, $d\sigma_{ij}$ and $d\varepsilon_{ij}$ are the increments of stress and total strain, respectively. Given an increment of strain, an increment of stress is calculated.

The expression of D_{ijkl}^{ep} is (Zienkiewicz and Taylor, 2005; Brady and Brown, 2006):

$$D_{ijkl}^{ep} = D_{ijkl}^e - \frac{D_{ijpq}^e \frac{\partial f}{\partial \sigma_{pq}} (\frac{\partial f}{\partial \sigma_{pq}})^T D_{ijpq}^e}{A + (\frac{\partial f}{\partial \sigma_{pq}})^T D_{ijpq}^e \frac{\partial f}{\partial \sigma_{pq}}} \quad [3.13]$$

where: D_{ijkl}^e is the elastic constitutive matrix, and the parameter A is defined as follows (Zienkiewicz and Taylor, 2005; Brady and Brown, 2006):

$$A = - \frac{\partial f}{\partial H} (\sigma_{ij})^T \frac{\partial f}{\partial \sigma_{pq}} \quad [3.14]$$

3.4. Concepts of continuum damage mechanics

The behaviour of sedimentary rocks under mechanical loads is affected by the initiation, growth, and coalescence of microcracks leading to the formation and propagation of macrocracks and eventually to failure. Continuum damage mechanics (CDM) offers possibilities for microcrack analyses as it uses a scalar variable or tensor to describe the deterioration of the material integrity due to the initiation and propagation of microcracks. It has been introduced by Kachanov in 1958, and then a further contribution was made by Rabotnov (1971) with the development of the effective stress approach.

More rigorous developments of CDM began in the 1970s based on micromechanics and thermodynamics principles, see for example, Lemaitre and Chaboche (1978), Lemaitre and Mazars (1982), Ladeveze (1986), Lemaitre (1992), Ristinmaa and Ottosen (1998), Santaoja (2001), and Olsson and Ristinmaa (2003). Micromechanics consist of deriving the behavior of materials at the macroscale from the study of specific mechanisms at the microscale. Using thermodynamics principles, the behaviour of the material can be described based on the existence of energy potentials from which one can derive the state laws and the kinetic constitutive equations. Entropy production within the system can be used as a damage evolution metric (Basaran and Nie, 2004).

3.4.1. Measurement of damage

There is no technique for direct measurement of damage. According to Mazars and Pijaudier-Cabot (1989), the damage can be measured using two techniques: measurement of damage based

on the observations of the microstructure and measurement of damage based on the variation of material properties. The latter technique involves the analysis of the variation of the elastic modulus, variation of ultrasonic wave's propagation, variation of the micro-hardness, variation of density, acoustic emission events, and variation of the electrical resistance.

3.4.2. Mechanical representation of damage

The concept of REV can be adopted to define the material damage at a mathematical point. In CDM, the discontinuous and discrete elements of damage are not considered within the REV; rather their combined effects are lumped together through the use of a macroscopic internal variable. In this way, the formulation may be derived consistently using mechanical and thermodynamic principles. The size of the REV depends on the type of the material (Lemaitre, 1992).

3.4.3. Approaches for formulating CDM constitutive equations

In continuum damage mechanics, formulation of the constitutive equations is based on one of the following two approaches.

3.4.3.1. Strain equivalence principle

Strain equivalence principle, as illustrated in Fig. 3.1, follows the effective stress concept and helps us to avoid a micromechanical analysis for each type of defect and each type of damage mechanism. It is stated that any stress-strain constitutive equation for damaged material may be derived in the same way as for a virgin material, except that the usual stress is replaced by the effective stress (Lemaitre, 1992):

$$(\sigma_{ij})_{eff} = \frac{\sigma_{ij}}{1-d} \quad [3.15]$$

where d is the damage parameter (mechanical).

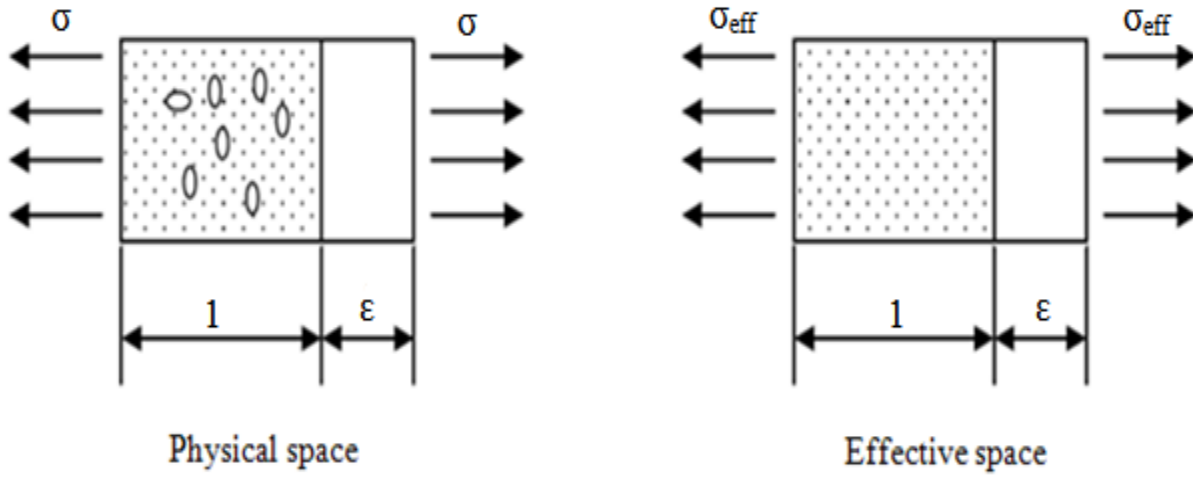


Figure 3.1. Hypothesis of strain equivalence concept (Simo and Ju, 1987).

3.4.3.2. *Stress equivalence principle*

Stress equivalent principle is illustrated in Fig. 3.2. The hypothesis of stress equivalence can be described as follows: the stress associated with a damaged state under the applied strain is equivalent to the stress associated with its undamaged state under the effective strain (Simo and Ju, 1987). The effective strain can be expressed as:

$$(\varepsilon_{ij})_{eff} = (1 - d)\varepsilon_{ij} \quad [3.16]$$

where d is the damage variable (mechanical).

Simo and Ju (1987) stated that the strain equivalence principle is associated with a strain-based formulation of the constitutive equations while the stress equivalence principle corresponds to a stress-based formulation.

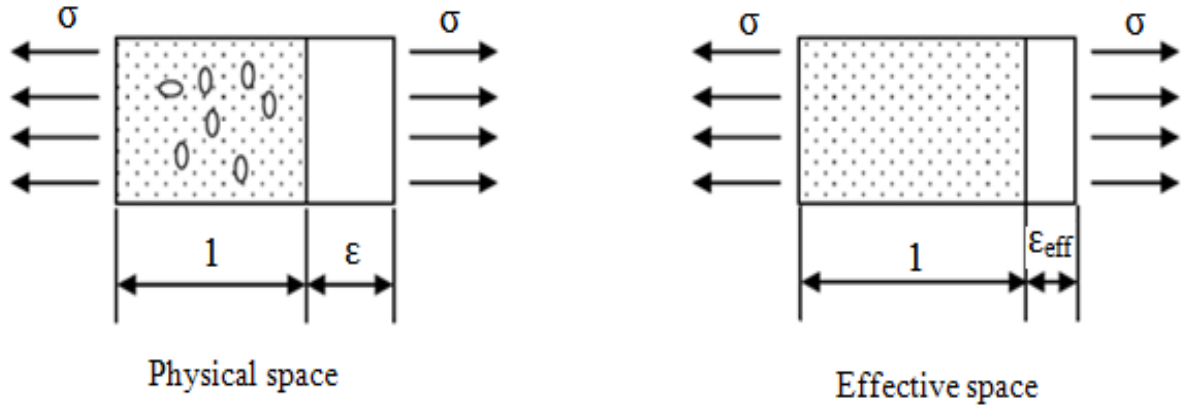


Figure 3.2. Hypothesis of stress equivalence concept (Simo and Ju, 1987).

3.4.4. Mathematical representation of the damage variable

For simplicity reasons, the initiation and propagation of microcracks throughout the material is assumed to be uniformly distributed everywhere (isotropic damage). Based on this assumption, Kachanov (1958) defined a damage parameter d (mechanical) as shown in Fig. 3.3. The damage parameter can be treated as a scalar quantity and completely characterizes the three-dimensional damage state. When the material has few defects such as very small population of cracks and pores, the value of d is close to zero. When defects become more abundant and their effects become more important, the value of d increases. The stress-strain relationship can be expressed as:

$$\varepsilon_{ij} = \frac{\sigma_{ij}}{E_{ij}(1-d)} \quad [3.17]$$

where the damage parameter d can be expressed in terms of stress or strain.

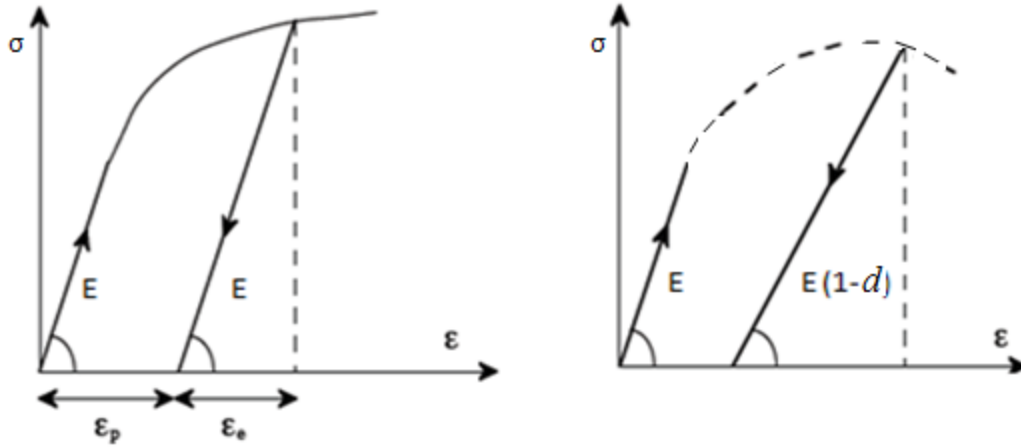


Figure 3.3. Stiffness degradation (d is the scalar damage variable).

The concept of isotropic damage is followed by many researchers. Tai and Yang (1986) followed the same concept, but they mentioned that the damage is caused by the microcrack evolution and necking. Salari et al. (2004) proposed a coupled elastoplastic damage model for geomaterials, in which damage is assumed to be isotropic and is represented by a single scalar variable that evolves under expansive volumetric strain.

However, most brittle materials develop anisotropic damage for which the damage variable can no longer be a scalar. In these situations, the damage vector or tensor can be used to describe the damage state. The concept of tensorial damage is first formulated by Vakulenko and Kachanov (1971), as they introduced a crack density tensor as a parameter that characterizes the effect of microcracking on the properties of the material. The concept of tensorial damage was further developed by Dragon and Mroz (1979).

Leckie and Onat (1981), Onat (1986), and Chaboche (1984) developed tensorial damage models for predicting anisotropic creep damage. Chow and Wang (1987) developed an anisotropic damage model for ductile behavior. Lee et al. (1985) proposed a model for damage evolution and a damage criterion for deformation instability of metal plates.

Here, the concept of isotropic damage is adopted. The initiation and propagation of microcracks throughout sedimentary rocks is assumed to be uniformly distributed everywhere and to be independent of loading orientation.

3.5. COMSOL Multiphysics

The development of a DGR within sedimentary rock formations can generate coupled THMC processes within the rock mass. A finite element code is needed to simulate such coupled processes. The capabilities and limitations of different available finite element codes such as ABAQUS, ANSYS, PLAXIS, COMSOL, and FLAC have been evaluated. Even though it suffers for some limitations such as the simulation of microcracks and fractures, the COMSOL Multiphysics was found to be the best toolset that can be used to simulate the coupled processes.

The COMSOL Multiphysics is an engineering, design, and finite element analysis software environment for modeling physics and engineering applications. It offers an extensive interface to MATLAB and its toolboxes for a large variety of programming, preprocessing and post-processing possibilities. In addition to conventional physics-based user interfaces, COMSOL Multiphysics also allows for entering coupled systems of partial differential equations (PDEs).

The COMSOL Multiphysics simulation environment facilitates all the steps in the modeling process: defining geometry and boundary conditions, specifying physics, meshing, solving, and then visualizing the results. The COMSOL Multiphysics user interface gives options for specifying equations and parameters and link them with other physics interfaces.

The COMSOL Multiphysics consists of a number of predefined Modules for applications ranging from fluid flow and heat transfer to structural mechanics and electrostatics. Material properties, source terms, and boundary conditions can all be spatially varying, time-dependent, or functions of the dependent variables. The COMSOL Multiphysics supports the mixing of

physics interfaces and coupling with any application specific Module. It produced thousands of publications in different areas of physics and engineering applications.

One of the objectives of this research project is to simulate the mechanical and hydromechanical behaviour of sedimentary rocks within the EDZ. The Structural Mechanics Module is used to simulate the mechanical behaviour of the rock. The Geomechanics Module is used to simulate the hydromechanical behaviour of the rock. More details about COMSOL and its formulations are provided in Appendix B.

CHAPTER 4

A LABORATORY INVESTIGATION ON THE MECHANICAL BEHAVIOUR OF TOURNEMIRE ARGILLITE

4.1. Background

A large laboratory testing program was carried out at CANMET Laboratories in Ottawa, Canada, to investigate the mechanical behaviour of the Tournemire argillite. The argillite is characterized by the presence of horizontal closely spaced bedding planes. Eighty rock samples of the argillite were obtained from seven boreholes drilled at different angles in the walls and floors of existing galleries at the site of the Tournemire Underground Research Laboratory (URL), France. The depth of the galleries is about 250 m. The experimental program includes the measurements of basic physical properties of the argillite and its mechanical response to loading during uniaxial compression tests, triaxial compression tests with various confining pressures, unconfined and confined cyclic compression tests, Brazilian tests, and creep tests. Acoustic emission was also recorded to detect the initiation and propagation of microcracks during the uniaxial compression testing.

At the location of sampling, the initial in situ stress field is around 4 MPa (Rejeb and Cabrera, 2006). Thus, confining pressures of 0, 4, and 10 MPa were used in triaxial and cyclic tests in the present investigation. A displacement rate of 0.03 mm/min was used to load all specimens during testing, except for the Brazilian tests that were carried out with a loading rate of 3.4 kN/min. The rock specimens were loaded following different inclinations with respect to the orientation of bedding planes (i.e. $\theta=0^\circ$, 30° , 45° , 60° , and 90°). In addition to the tests to determine the physical properties of the rock, around seventy (70) load tests have been completed at CANMET Labs. Most of the tests were performed at the natural moisture content of the rock specimens (3.86%) as delivered to the laboratory. ASTM and IRSM standards for testing have been followed.

The main objective of the testing program is to identify the mechanical properties of the Tournemire argillite and to provide experimental data that would be useful for the development of a constitutive relation. Such a model is required to assess the damage or stability of underground openings in similar types of rock. This Chapter describes the geology of the Tournemire site, the general layout of the URL, the mineralogy and hydraulic properties of the argillite, the testing program, and the experimental results.

4.2. Description of the Tournemire Site

The Tournemire site is located in a Mesozoic marine basin on the southern limit of the French Massif Central (Fig. 4.1). Sedimentary formations of this basin are characterized by three main Jurassic layers: a 250 m thick nearly horizontal layer of argillite and marls of Toarcian and Domerian age, located between two aquifer limestone and dolomite layers of Carixian and Aalenian age (Barbreau and Boisson, 1993; Boisson et al., 1997). The Tournemire massif has faults and fractures, and water flow takes place along lower and upper limestone aquifers. The Tournemire argillite is characterized by the presence of closely spaced bedding planes dipping at 4° towards the North as shown in Fig. 4.2, and contains randomly distributed fractures.

X-ray diffraction analysis of the argillite showed that the mineralogical composition consists of 55% of clay minerals, 19% quartz, 15% calcite and 11% of other minerals such as dolomite, pyrite, siderite, and feldspars (Schmitt, 1994). According to the experimental results obtained in the lab; the porosity of the argillite varies between 6% and 9%, its density is about 2550 kg/m^3 , its natural moisture content varies between 3 and 5%, and the coefficient of anisotropy is around 2.16. The permeability of the argillite is very low and estimated to be $10^{-21} \sim 10^{-22} \text{ m}^2$ (Matray et al., 2007). The porewater pressure, measured in various vertical and horizontal boreholes, varies between 0.2 and 0.6 MPa at the level of the URL (Rejeb and Cabrera, 2006; Matray et al., 2007).

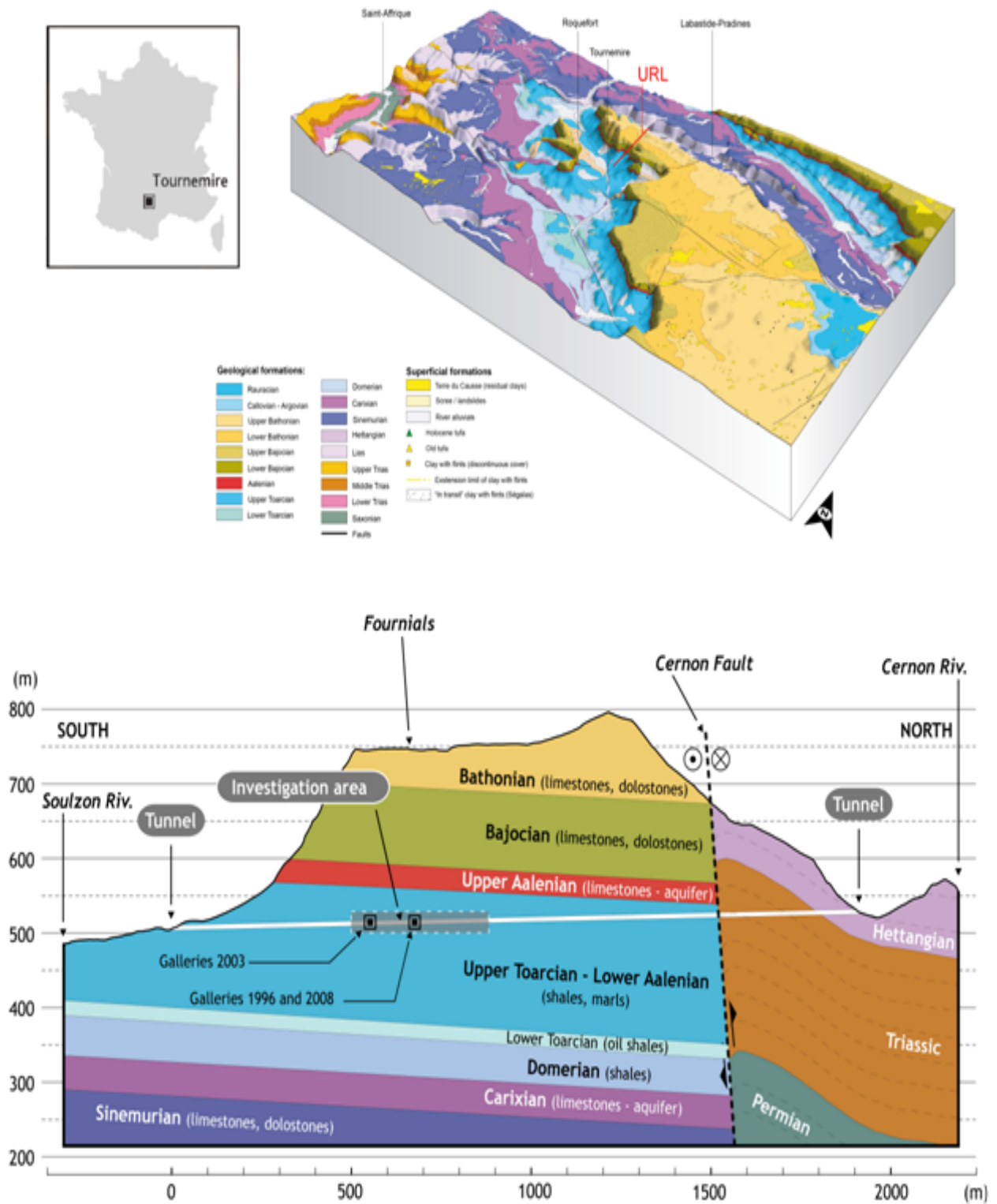


Figure 4.1: The Tournemire URL: location and geological environment (top); Geological cross section along the URL (bottom) (Rejeb et al., 2007).

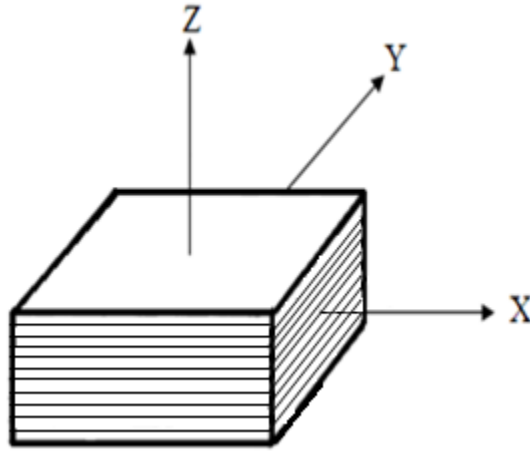


Figure 4.2. Stratified argillaceous Tournemire argillite (transverse isotropic rock).

4.3. General layout of the URL and boreholes

Figure 4.3 provides a general view of the URL in Tournemire. It consists of a century-old tunnel, the sixteen year-old east and west galleries, the nine year-old east and west galleries, the four year-old north, south, and west galleries, and the four year old niche. The century-old tunnel is about 2 km long and 250 m deep. Rock specimens were obtained from Gallery South_08 and Gallery West_08. As shown in Figures 4.3 and 4.4, the drilling (sampling process) was carried out at different angles (θ) with respect to the orientation of bedding planes: parallel (boreholes M13 and TD4M, $\theta=0^\circ$), perpendicular (boreholes M8 and M9, $\theta=90^\circ$), and three intermediate inclinations (borehole M12, $\theta=30^\circ$; borehole M11, $\theta=45^\circ$; and borehole M10, $\theta=60^\circ$).

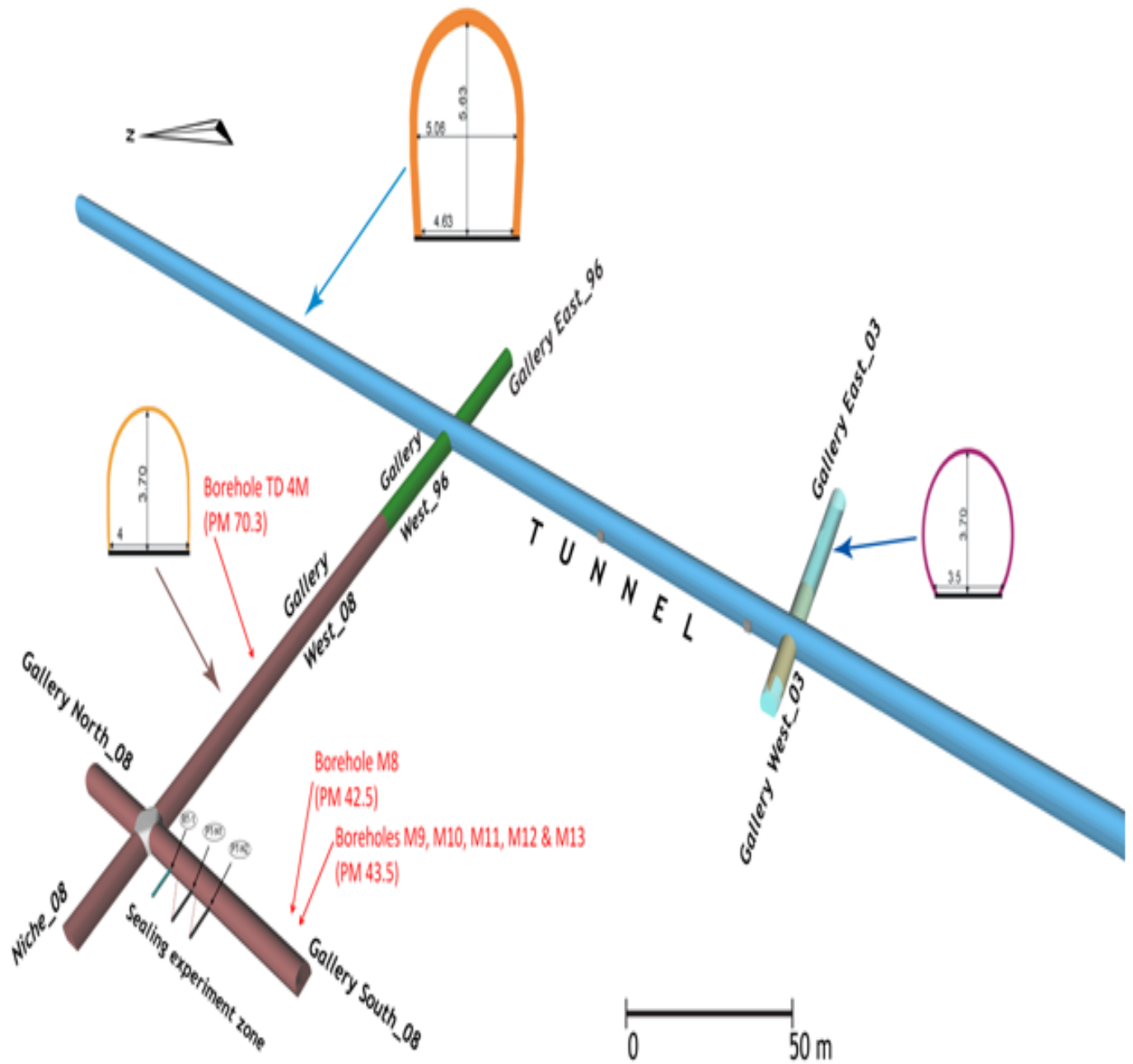


Figure 4.3: General layout of the Tournemire URL.

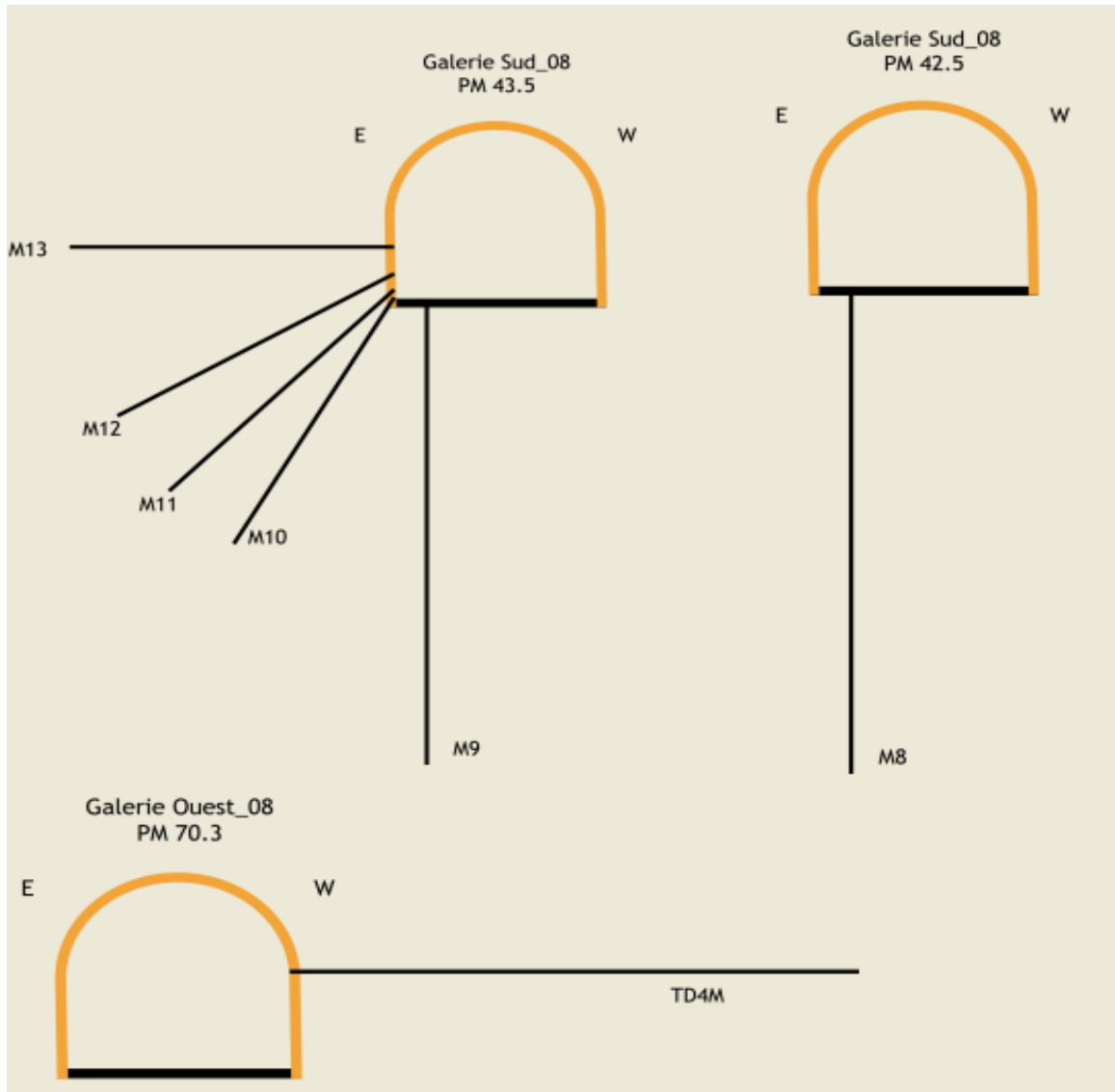


Figure 4.4: General layout of the boreholes: M8, M9, M10, M11, M12, M13, and TD4M.

4.4. Description of testing equipment and test specimens

The testing equipment consists of a MTS rock mechanics system, Model 815 (Fig. 4.5). This 4600 kN computer-controlled servo-hydraulics compression load frame is equipped with a triaxial cell capable of generating confining pressure of up to 140 MPa. The triaxial cell is equipped with three linear variable differential transducers arrayed around the specimen at 120°

intervals for the measurement of axial deformations, an extensometer with a circumferential kit attachment for the measurement of circumferential deformation, TestStar II Digital Controller, and the TestWare SX software. One of the capabilities of the machine is to quickly and accurately alter test conditions and load paths according to project requirements. This triaxial machine is used regularly by leading researchers for projects ranging from routine testing to fundamental rock mechanics research. A detailed description of the MTS system is reported in Appendix A. Acoustic emissions (AE) are recorded through a measuring system developed by Physical Acoustics Corporation (PAC) as shown in Fig. 4.5.d which is operated simultaneously but independently from the main MTS.

Specimens from drill cores are prepared by cutting them to the specified length and are thereafter grinded and measured. The end-surfaces are flattened in order to obtain an even load distribution. A membrane is mounted on the envelop surface of the specimen in order to seal the specimen from the surrounding pressure. The specimen is inserted into the pressure cell. Deformation measurement equipment and acoustic emission sensor are mounted on the specimen. The cell is closed and filled with water. A hydrostatic pressure is applied in the first step. The specimen is then further loaded.



Figure 4.5: Testing equipments: a) MTS Rock Mechanics Testing System, b) setup using a granite specimen, c) setup using Tournemire argillite specimen, and d) Acoustic emission device.

Cylindrical argillite specimens of 61.3 mm in diameter and 133 mm in height (Fig. 4.6.c) were used for uniaxial, triaxial, cyclic, and creep tests, consistent with the specifications of the ASTM Standard for the preparation of test specimens (ASTM D4543). Specimens with the same diameter and a nominal length of 40 mm (Fig. 4.6.b) were used for Brazilian tests. Usual ASTM procedures (C496 and D3967) were used in the Brazilian tests.

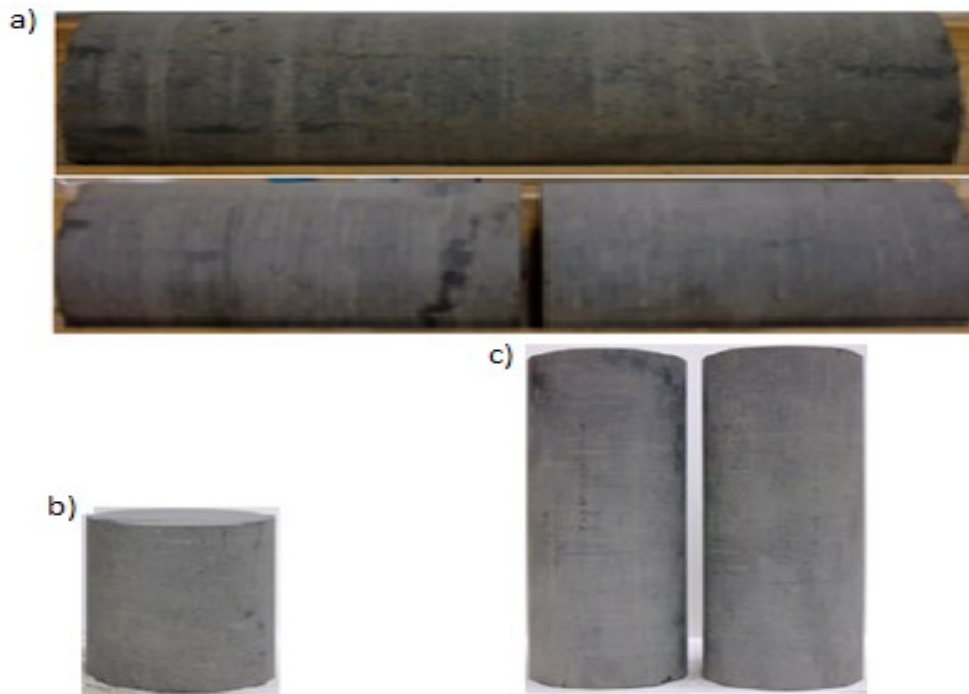


Figure 4.6. Rock specimens: a) As received, b) Brazilian specimen, and c) triaxial specimen.

4.5. Testing Program

In addition to the measurement of basic physical properties of the argillite such as the moisture content, density, porosity, and P-wave velocity, the following tests are also carried out at CANMET Laboratories (Table 4.1). Acoustic emissions were recorded to detect the initiation and propagation of microcracks during the uniaxial testing. The permeability of the argillite is very low (in the order of 10^{-22} m²) and the measurements of porewater pressure have not been considered in this testing program as it requires special equipments (not available in the laboratory).

Table 4.1: Description of the tests conducted at CANMET Laboratories.

LOADING ORIENTATION	DESCRIPTION OF THE TEST	BOREHOLE NUMBER	CONFINING PRESSURE (MPa)	ACOUSTIC EMISSION
$\theta=0^\circ$	Six uniaxial tests	M13, TD4M	0	Yes
	Two triaxial tests	M13	4 and 10	No
	One cyclic test	M13	0	Yes
	One cyclic test	M13	10	No
	Six Brazilian tests (3 wet and 3 dry)	M13, TD4M	na	na
	One creep test	M13	0	No
$\theta=30^\circ$	Three uniaxial tests	M12	0	Yes
	Two triaxial tests	M12	4 and 10	No
	One cyclic test	M12	0	Yes
	One cyclic test	M12	10	No
	Three Brazilian tests	M12	na	na
	One creep test	M12	0	No
$\theta=45^\circ$	Three uniaxial tests	M11	0	Yes
	Two triaxial tests	M11	4 and 10	No
	One cyclic test	M11	0	Yes
	One cyclic test	M11	10	No
	Three Brazilian tests	M11	na	na
	One creep test	M11	0	No
$\theta=60^\circ$	Three uniaxial tests	M10	0	Yes
	Two triaxial tests	M10	4 and 10	No
	One cyclic test	M10	0	Yes
	One cyclic test	M10	10	No
	Three Brazilian tests	M10	na	na
	One creep test	M10	0	No
$\theta=90^\circ$	Eight uniaxial tests (6 wet and 2 dry)	M8, M9	0	Yes
	Four triaxial tests	M8, M9	4 and 10	No
	One cyclic test	M9	0	Yes
	One cyclic test	M9	10	No
	Six Brazilian tests (3 wet and 3 dry)	M8, M9	na	na
	One creep test	M9	0	No

NOTE: The definition of the loading orientation angle, θ , is provided in Fig. 4.7.

4.6. Description of experimental results

Before proceeding with the presentation of results, the orientation angle (θ) should be defined. The loading orientation, θ , is the angle between the vertical axis (z), representing the direction of the major principal stress, and the x' axis which is parallel to the bedding planes as shown in Fig. 4.7. For example, $\theta=90^\circ$ means the major principal stress is perpendicular to the orientation of bedding planes and $\theta=0^\circ$ means the major principal stress is parallel to the orientation of bedding planes, as shown in Fig. 4.8. In this research work, the term 'depth' represents the distance of the sample within the borehole from the borehole collar.

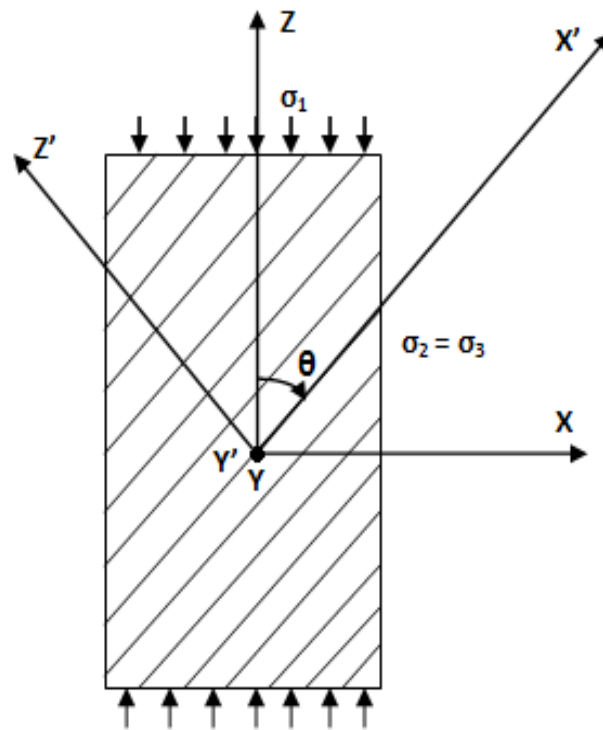


Figure 4.7. Loading orientation angle with respect to the orientation of bedding planes.

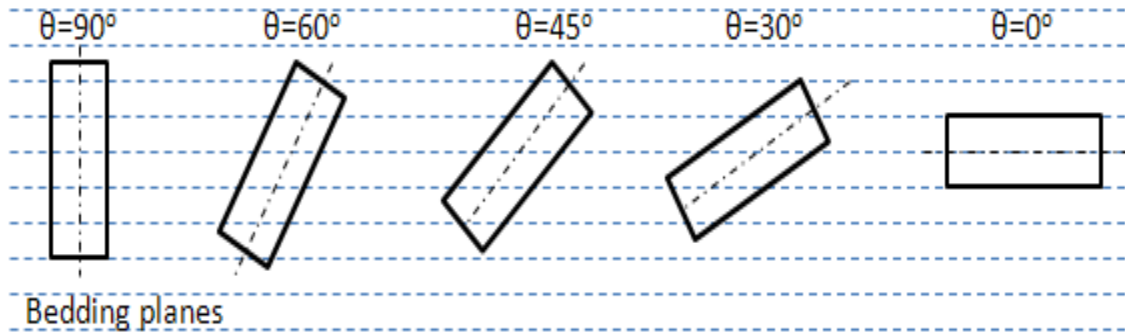


Figure 4.8. Sampling of the argillite at five different loading orientation angles.

4.6.1. Basic physical properties of the Tournemire argillite

The following properties were determined from laboratory experiments. The moisture content of the rock specimens is 3.86% with standard deviation (std) equal to 0.078, the bulk density is 2550 kg/m³ (std=0.0244), and the porosity is 9.52% (std=0.211).

4.6.2. Coefficient of anisotropy

P-waves are a type of elastic wave that can travel through a medium. The P-wave has the highest velocity and is therefore the first to be recorded. The travel time of the wave depends on the interior structure of the medium. The relationship between different physical-mechanical properties of various rock types with the P-wave velocity has been investigated; see for example, Khandelwal (2013) and Bery and Saad (2012). Okaya et al. (2004) used P-wave technique to investigate the anisotropy of the crystalline Bohemian massif, southeast Germany. Their results indicate that the anisotropy can be explained by “intrinsic” material properties associated with the well-developed foliation fabrics. Here, the P-wave travel time is used to estimate the coefficient of the anisotropy of the material. More details about the AE technique are provided in the Appendix C.

The measurements of the travel time of P-wave have been carried out in two principal directions: parallel ($\theta=0^\circ$) and perpendicular ($\theta=90^\circ$) to bedding planes. Different depths have been

considered. Table 4.2 describes the measurements of the P-wave velocity. The average value of the coefficient of anisotropy is about 2.16.

Table 4.2. Determination of the coefficient of anisotropy based on the measurements of P-wave velocity in the two principal directions.

Loading orientation	Borehole No.	Sample No.	Depth (m)	P-Wave velocity (km/sec)	Average (P-wave)	Coefficient of anisotropy
0°	TD4M	TD4M-525	5.25	3.94	3.831782	2.16
	TD4M	TD4M-540-A	5.40	3.90		
	TD4M	TD4M-540-B	5.55	3.87		
	TD4M	TD4M-610-A	6.10	3.77		
	TD4M	TD4M-610-B	6.25	3.78		
	TD4M	TD4M-690	6.90	3.73		
90°	M8	M8-524	5.24	2.03	1.773375	
	M8	M8-544	5.44	2.14		
	M8	M8-576	5.76	1.39		
	M8	M8-611	6.11	1.88		
	M8	M8-646	6.46	1.55		
	M8	M8-661	6.61	1.32		
	M8	M8-677	6.77	1.19		
	M8	M8-693	6.93	1.67		
	M8	M8-708	7.08	1.93		
	M9	M9-510	5.10	3.05		
	M9	M9-580	5.80	1.51		
	M9	M9-595	5.95	1.96		
	M9	M9-618	6.18	1.57		
	M9	M9-635	6.35	2.05		
	M9	M9-685	6.85	1.31		
	M9	M9-779	7.79	1.82		

4.6.3. Acoustic Emission data and mobilized strength parameters

Acoustic Emissions (AEs) technique is a non-destructive method and it has been used to investigate the initiation and propagation of microcracks and fractures in rocks; see for example, He et al. (2010) and Lockner (1993). Here, it is used for detecting the initiation and propagation of microcracks in rock specimens subjected to mechanical loading. The intensity of microcracking is assumed to depend on the number of recorded AE events (Hits). More details about AE technique are provided in Appendix D.

Typical “AE Signatures” for an oven dry specimen and for a wet specimen of the Tournemire argillite are shown in Figs 4.9 and 4.10, respectively. In these figures, one of the horizontal axes represents the elapsed time during testing. “Parametric 1(Volts)” axis provides a measure of the applied load and “Hits” represents the number of AE events. The loading of the specimen starts at point A and proceeds towards point B. In Fig. 4.9, all AE events between points A and B are related to microcrack closer. Once these microcracks are closed, they do not propagate for some time during the loading. For that reason, there are not many AE events recorded between the points B and C. Starting at around point C, the number of hits increase significantly until the stress level reaches to its maximum value at point D. For the purpose of developing a constitutive relation, it would be possible to follow the approximations illustrated in Figure 4.11.

1. The material resistance to applied load between points A and C (shown in Fig. 4.9) can be based mainly on cohesion. In this part, there is a small amount of frictional resistance because of the existing microcracks in the material before testing. For additional AE events to take place either new microcracks to develop or the opposing surfaces of propagating microcracks (fractures) to slide against each other causing micro-tremors (AE event).
2. Between points C and D, both the cohesive resistance and frictional resistance are active. In this part, the rate of increase in cohesive resistance is decreasing and the rate of increase of frictional resistance is increasing rapidly (see Fig. 4.11.a).
3. After point D (peak strength), it is assumed that there is no cohesion left in the sample and only the frictional resistance between the opposing surfaces of fractures supports the applied load.

The Tournemire argillite has a high clay content (55%) which makes this rock to behave less brittle than a hard rock. Considering all experimental AE data, new microcracks start to develop approximately at a stress level $\sigma_{ci} = 70\% \sim 75\%$ of the peak strength (σ_{ci} = crack initiation stress). At a stress level above σ_{ci} , the development and propagation of microcracks accelerate with the increase in stress level. In addition, according to the observations made in the present experimental data, the crack damage stress level (σ_{cd} - unstable crack growth) as defined by Martin (1993) is reached approximately at a stress level of $\sigma_{cd} = 85\% \sim 90\%$ of the peak strength.

For the Tournemire argillite the inelastic straining starts to develop at the beginning of the stress-strain curve as a result of the closure of the existing fissures and bedding planes. The experimental data suggest that the mobilization of strength parameters under increasing axial strain can be approximated as shown in Fig. 4.11.a. The values of the mobilized strength parameters are influenced by the loading orientation angle θ and the confining pressure. In summary, upon loading the specimen, the mobilized cohesion starts to increase very rapidly with the increase in axial strain. The rate of increase in the mobilized cohesion tends to slow down when the stress level increases beyond σ_{ci} (Point C). The mobilized cohesion reaches a maximum value at the peak strength. However, as the axial strain increases beyond its value corresponding to the peak strength, the cohesion reduces rapidly to a negligible value. On the other hand, the mobilized friction angle is small at very low stress levels. Initially, the rate of its increase is also slow. When the stress level increases beyond σ_{ci} , the increase in the mobilized friction angle becomes large and its value reaches a maximum value at the peak strength. In the post peak region, the frictional resistance becomes the only component of the post-peak strength.

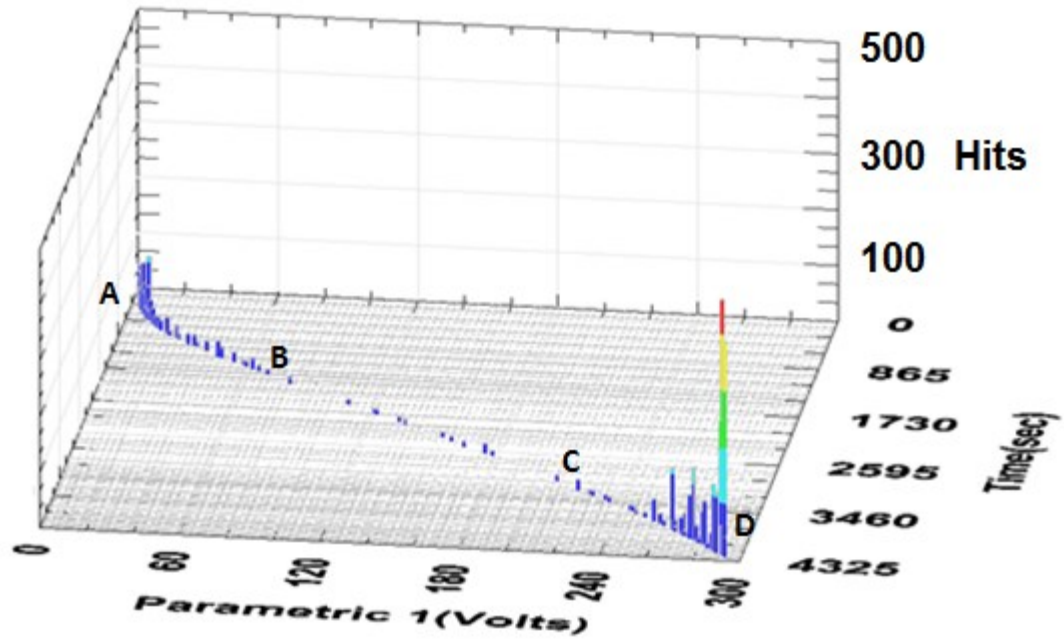


Figure 4.9: A typical AE Signature for a uniaxial test (dry sample).

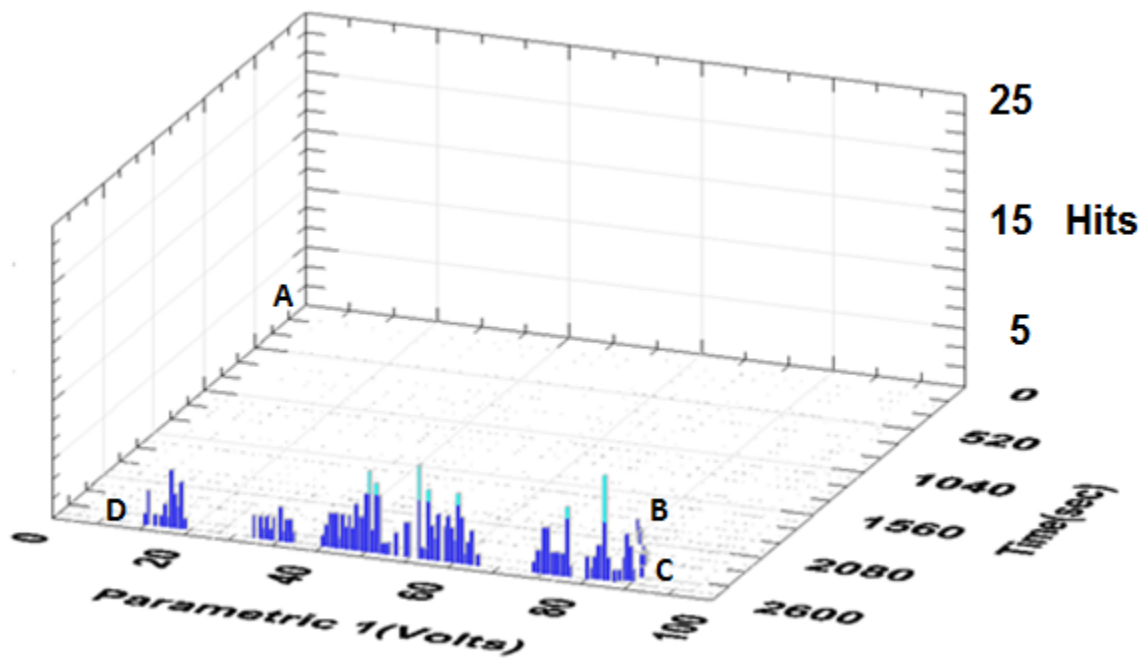


Figure 4.10: A typical AE Signature for a uniaxial test (wet sample).

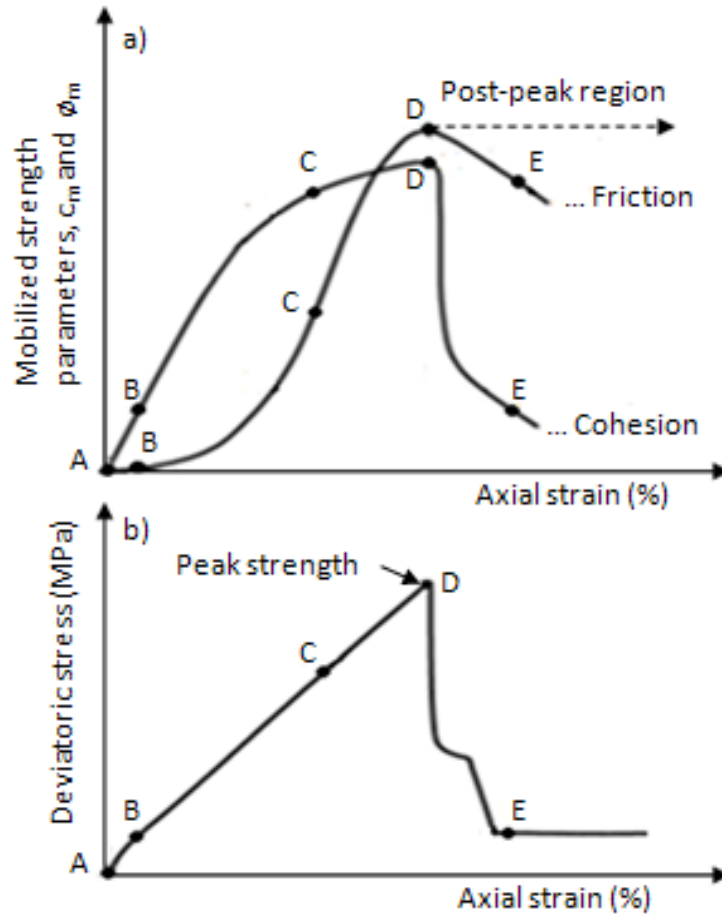


Figure 4.11. Mobilization of strength parameters: a) c_m and ϕ_m as a function of axial strain, b) typical stress-strain curve for the Tournemire argillite.

4.6.4. Mechanical properties of the argillite at two different water contents

Uniaxial tests were carried out on oven dry samples and samples at their natural water content (3.86%) obtained from vertical boreholes ($\theta=90^\circ$). Specimens were taken from a depth ranging from 5.1 m to 6.85 m. Typical results are shown in Figs 4.12 and 4.13. The results indicate that the compressive strength, elastic modulus, volumetric strain, and axial strain at the peak are higher for the oven dry sample. The volumetric strain is calculated from the experimental data as follows.

$$\varepsilon_v = \varepsilon_a + 2 \varepsilon_r \quad [4.1]$$

where ε_a is the axial strain and ε_r is the radial strain (measured at the middle of the specimen).

The Poisson's ratio is higher for the sample at its natural water content. The uniaxial compressive strength of oven dry sample is three times higher than the corresponding value obtained from a sample tested at its natural water content. In both cases, the volumetric strain change is mainly in contraction, and the transition from contraction to dilation is occurring at the peak strength.

The effect of the water on the strength of the rock depends on the mineralogical composition of the rock (Li et al., 2005). The more water sensitive constituents such as clay and silt it contains, the more deterioration effect the water has. If the rock contains certain amounts of clay or silt, not only the cohesive strength but also the basic friction angle will be altered by the degree of saturation. The Tournemire argillite has a high clay content of about 55% and its mechanical properties depend on the degree of saturation.

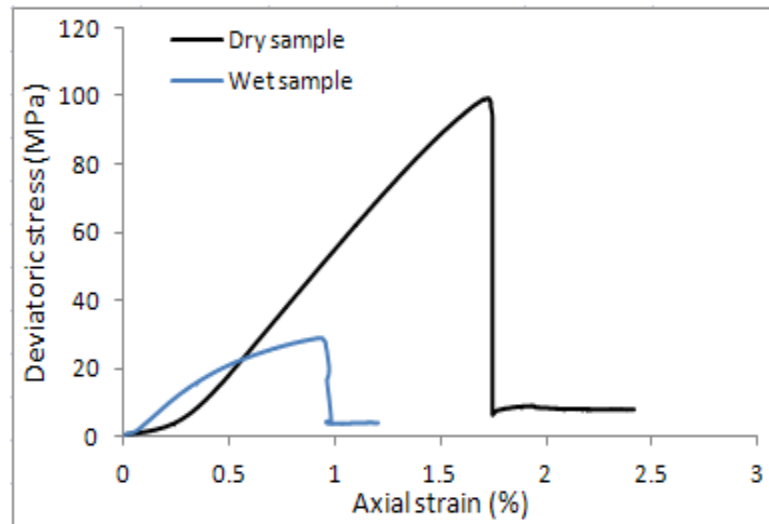


Fig. 4.12. Deviatoric stress-axial strain curves for a dry sample and a wet sample.

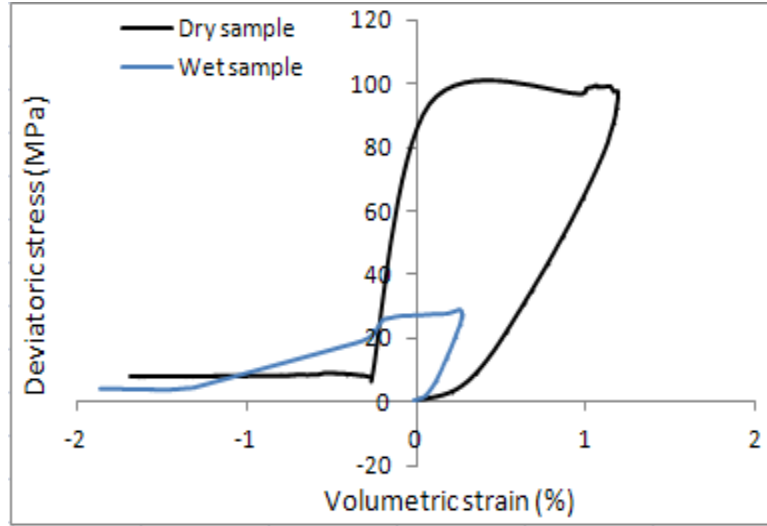


Fig. 4.13. Deviatoric stress-volumetric strain curves for a dry sample and a wet sample.

4.6.5. Brazilian indirect tensile tests

Fifteen Brazilian tests were carried out on wet specimens obtained from different boreholes (M8, M9, M10, M11, M12, M13, and TD4M) at a depth ranging from 3.55 m to 7.4 m. Also, six Brazilian tests were carried out on dry specimens obtained from borehole M8 and M9 at a depth ranging from 6 m to 8 m. Typical tensile strength-displacements curves for wet specimen and dry specimen are shown in Fig. 4.14, respectively.

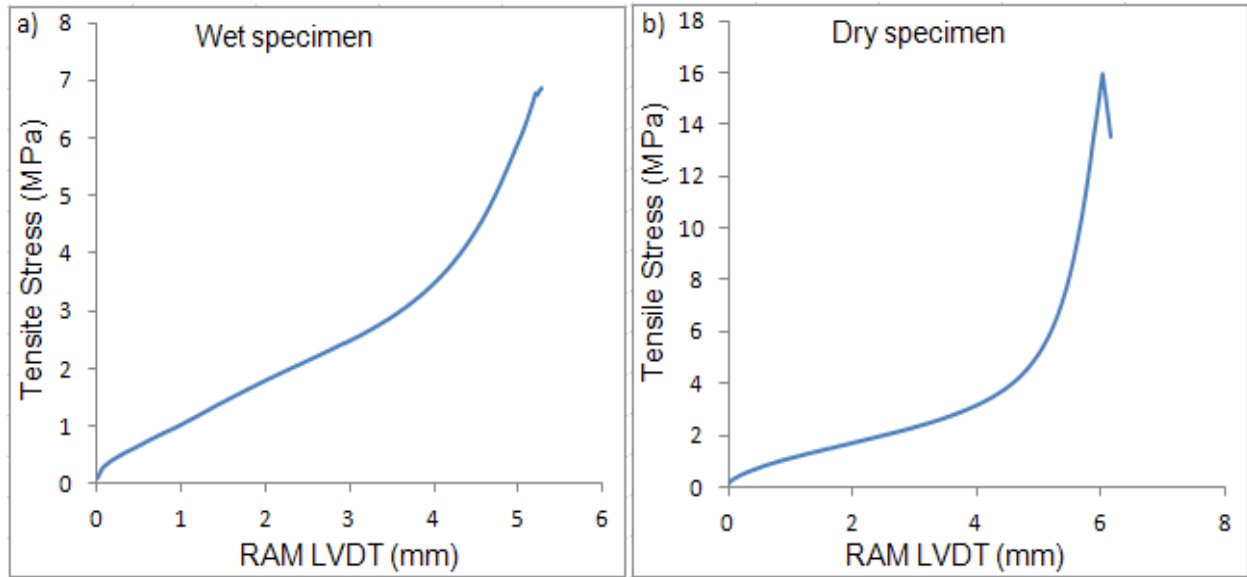


Figure 4.14. Tensile strength-displacement curves: a) wet specimen, and b) dry specimen.

The experimental results obtained from testing wet specimens at five different loading orientation angles are shown in Fig. 4.15. Each point represents an average of the results of three tests. The tensile strength is influenced by the loading orientation angle, θ .

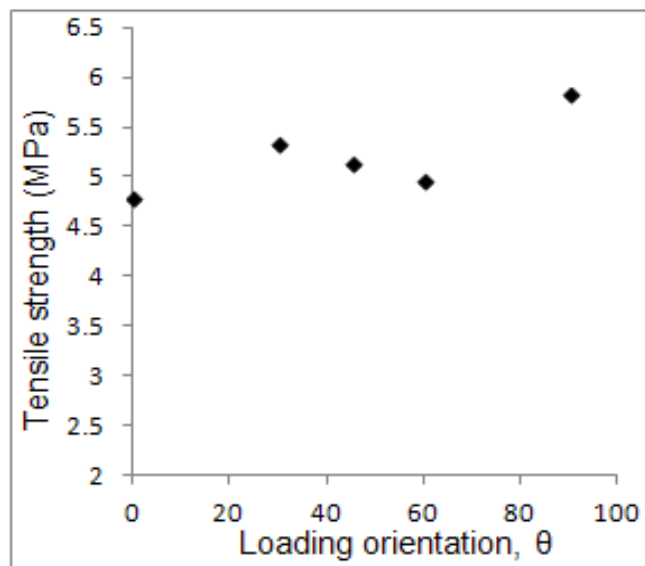


Figure 4.15: Variation of tensile strength with loading orientation (wet samples).

The tensile strength obtained from dry samples is around 15 MPa. This strength is three times higher than the corresponding value obtained from samples tested at their natural water contents.

4.6.6. Peak strength of the argillite at two different locations within the URL

Six uniaxial tests were performed to determine the peak strength of the argillite at two different locations within the URL. Three specimens were obtained from borehole TD4M and three others were obtained from borehole M13 (Fig. 4.3). Both boreholes were drilled at same elevation within the URL and at $\theta=0^\circ$. The distance between the two boreholes is about 100 m. The depth of the specimens varies between 3.55 m and 6.1 m. The specimens were tested at their natural water content.

The obtained deviatoric stress-axial strain curves and deviatoric stress-volumetric strain curves are shown in Figs 4.16 and 4.17, respectively. The specimens obtained from borehole M13 showed peak strength of 30 MPa (in average) which is higher than the average peak strength obtained from the corresponding specimens sampled from borehole TD4M, by about 30%. Similarly, the specimens obtained from borehole M13 showed slightly higher values of volumetric strains than the corresponding specimens obtained from borehole TD4M. The difference in the results could be due to the difference in the overburden pressure at the two locations of sampling as shown in Fig. 4.18.

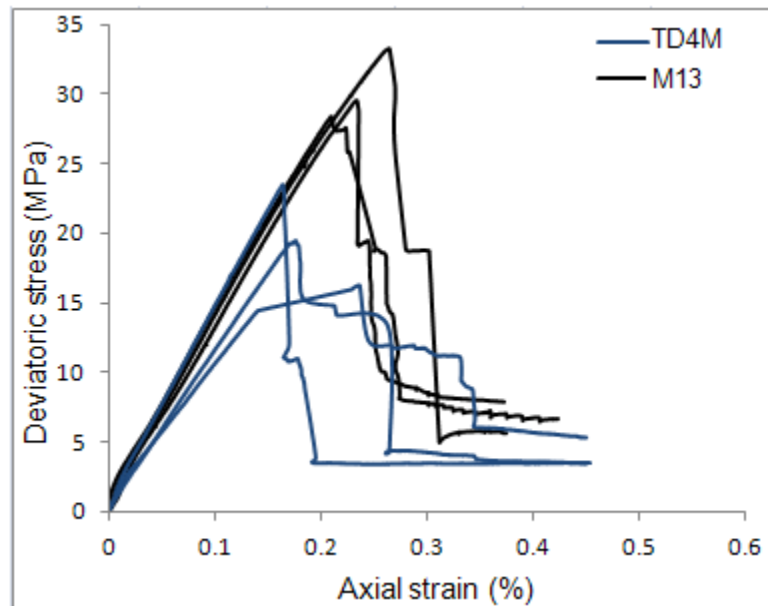


Figure 4.16. Deviatoric stress-axial strain curves obtained from specimens sampled at two different locations within the URL ($\theta=0^\circ$), and with the same elevation (wet samples).

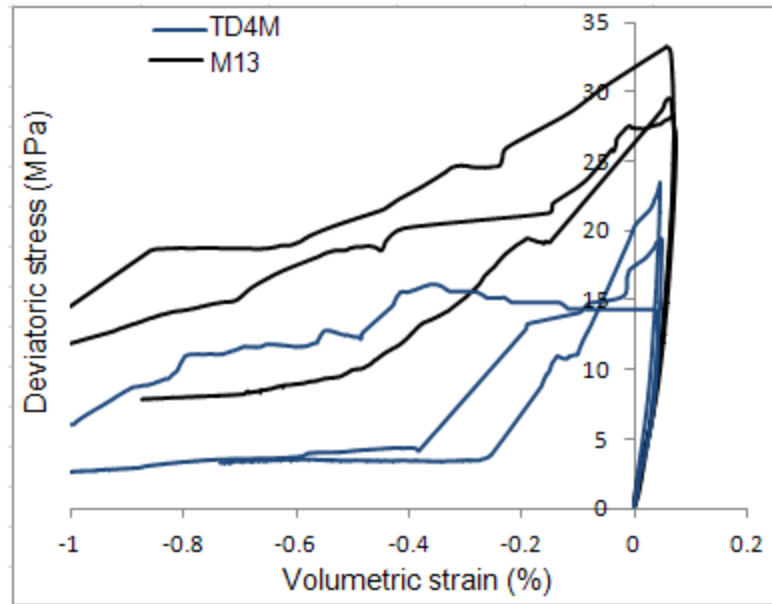


Figure 4.17. Deviatoric stress-volumetric strain curves obtained from specimens sampled at two different locations within the URL ($\theta=0^\circ$), and with the same elevation.



Figure 4.18. The Tournemire experimental site (Images of the Tournemire URL – Internet).

4.6.7. Uniaxial and triaxial tests

Uniaxial compression and triaxial compression tests were carried out on wet specimens, which were obtained from depths ranging from 3.55 m to 6.85 m. The loading orientation angles considered are $\theta=0^\circ$, 30° , 45° , 60° , and 90° . Confining pressures of 4 MPa and 10 MPa are applied in triaxial tests. The experimental results are described in Tables 4.3(a), 4.3(b), 4.3(c), 4.3(d), and 4.3(e) for $\theta=0^\circ$, 30° , 45° , 60° , and 90° , respectively. Figure 4.19 shows the deviatoric stress-axial strain curves for all loading orientations and confining pressures. Figure 4.20 shows the deviatoric stress-volumetric strain curves for all loading orientations and confining pressures.

The results indicate that the behaviour of the Tournemire argillite is highly nonlinear. The material shows hardening behaviour and then softening behaviour. The peak strength, peak axial strain, volumetric strain, and post-peak strength are influenced by the confining pressure and loading orientation angle, θ . The volumetric strain is mainly contraction up to the peak strength where a sharp transition to dilation occurs due to rapid formation and propagation of microcracks. The Tournemire argillite behaves as a brittle material. A sudden collapse of the material, e.g. moving quickly from peak strength to post-peak strength, is observed in both uniaxial and triaxial compression tests. In Fig. 4.20, for the test with $\theta=90^\circ$ and confining pressure of 10 MPa, a fracture is developed within the upper part of the specimen, above the middle section of the sample. As a result, the actual value of the volumetric strain after the peak strength is not captured by the instrumentation.

Table 4.3(a). Experimental data for $\theta=0^\circ$: Axial strain, volumetric strain, and deviatoric stress.

$\sigma_3 = 0$ MPa			$\sigma_3 = 4$ MPa			$\sigma_3 = 10$ MPa		
ϵ_a	ϵ_v	$\sigma_1 - \sigma_3$	ϵ_a	ϵ_v	$\sigma_1 - \sigma_3$	ϵ_a	ϵ_v	$\sigma_1 - \sigma_3$
(%)	(%)	(MPa)	(%)	(%)	(MPa)	(%)	(%)	(MPa)
0	0	0	0	0	0	0	0	0
0.0105	0.0106	1.73	0.015	0.017	2.71	0.0209	0.023	2.09
0.0216	0.017	3.67	0.028	0.028	5.17	0.039	0.039	4.54
0.0424	0.0302	6.13	0.042	0.038	7.74	0.0539	0.049	7.07
0.067	0.0424	9.07	0.062	0.051	10.82	0.073	0.0605	10.2
0.0937	0.0527	12.5	0.085	0.062	13.98	0.096	0.0728	13.82
0.114	0.0593	15.35	0.107	0.073	17.06	0.11	0.0782	15.96
0.1396	0.0647	18.64	0.129	0.081	20.03	0.134	0.0885	19.4
0.16	0.0692	21.37	0.152	0.088	23.0	0.163	0.0987	23.3
0.1873	0.0731	24.6	0.173	0.094	25.56	0.193	0.107	27.11
0.208	0.0732	27.03	0.197	0.101	28.54	0.219	0.114	30.15
0.229	0.0569	29.17	0.219	0.104	31.0	0.25	0.121	33.56
0.23	-0.165	19.19	0.241	0.109	33.35	0.271	0.127	35.82
0.257	-0.484	10.06	0.259	0.112	35.3	0.30	0.132	38.6
0.346	-0.794	8.03	0.284	0.116	37.7	0.328	0.137	41.19
			0.303	0.118	39.49	0.355	0.14	43.44
			0.326	0.12	41.44	0.383	0.143	45.59
			0.353	0.119	43.53	0.413	0.146	47.7
			0.391	0.0006	31.59	0.439	0.147	49.4
			0.439	-0.067	22.37	0.468	0.147	51.1
			0.564	-0.168	19.22	0.492	0.147	52.4
			0.688	-0.25	17.87	0.529	0.146	53.9
						0.55	0.0803	38.9
						0.663	0.0666	31.21
						0.83	0.0555	27.28

Table 4.3(b). Experimental data for $\theta=30^\circ$: Axial strain, volumetric strain, and deviatoric stress.

$\sigma_3 = 0$ MPa			$\sigma_3 = 4$ MPa			$\sigma_3 = 10$ MPa		
ϵ_a	ϵ_v	$\sigma_1 - \sigma_3$	ϵ_a	ϵ_v	$\sigma_1 - \sigma_3$	ϵ_a	ϵ_v	$\sigma_1 - \sigma_3$
(%)	(%)	(MPa)	(%)	(%)	(MPa)	(%)	(%)	(MPa)
0	0	0	0	0	0	0	0	0
0.013	0.012	1.38	0.014	0.0148	0.96	0.0255	0.027	2.28
0.0308	0.0273	2.673	0.025	0.0236	1.71	0.041	0.0385	3.97
0.046	0.0394	3.97	0.038	0.0333	3.11	0.062	0.0542	6.37
0.0623	0.0486	5.28	0.049	0.0408	4.21	0.0849	0.0674	8.48
0.078	0.058	6.66	0.073	0.0533	6.67	0.107	0.0778	10.61
0.094	0.0677	8.09	0.10	0.0663	9.37	0.128	0.0876	12.64
0.111	0.0753	9.57	0.115	0.0735	10.89	0.152	0.096	14.85
0.127	0.0808	10.98	0.128	0.0783	12.12	0.17	0.102	16.47
0.143	0.0855	12.3	0.15	0.086	14.13	0.192	0.109	18.42
0.158	0.0884	13.55	0.173	0.0933	16.21	0.214	0.114	20.22
0.189	0.0892	14.83	0.195	0.0994	18.11	0.237	0.12	22.14
0.187	0.075	15.27	0.212	0.102	19.55	0.256	0.124	23.65
0.219	-0.128	4.77	0.23	0.105	20.93	0.28	0.129	25.36
0.236	-0.213	1.96	0.252	0.109	22.71	0.298	0.132	26.68
			0.288	0.105	24.67	0.323	0.135	28.38
			0.318	0.0179	17.31	0.34	0.138	29.48
			0.374	-0.027	12.79	0.373	0.142	31.56
						0.387	0.143	32.37
						0.41	0.144	33.63
						0.427	0.146	34.59
						0.467	0.146	36.59
						0.501	0.138	38.0
						0.51	0.037	26.51
						0.56	0.009	21.21

Table 4.3(c). Experimental data for $\theta=45^\circ$: Axial strain, volumetric strain, and deviatoric stress.

$\sigma_3 = 0$ MPa			$\sigma_3 = 4$ MPa			$\sigma_3 = 10$ MPa		
ϵ_a	ϵ_v	$\sigma_1 - \sigma_3$	ϵ_a	ϵ_v	$\sigma_1 - \sigma_3$	ϵ_a	ϵ_v	$\sigma_1 - \sigma_3$
(%)	(%)	(MPa)	(%)	(%)	(MPa)	(%)	(%)	(MPa)
0	0	0	0	0	0	0	0	0
0.023	0.024	1.68	0.022	0.024	1.46	0.024	0.025	2.24
0.0459	0.044	2.81	0.037	0.037	2.36	0.047	0.0416	4.08
0.069	0.065	4.04	0.056	0.052	3.52	0.070	0.0554	5.89
0.096	0.082	5.61	0.073	0.066	4.54	0.0948	0.07	7.59
0.120	0.0976	7.05	0.093	0.079	5.78	0.122	0.0823	9.45
0.143	0.109	8.39	0.11	0.0903	6.81	0.142	0.0918	10.87
0.167	0.119	9.87	0.146	0.11	9.12	0.172	0.103	12.79
0.192	0.128	11.37	0.187	0.13	11.77	0.193	0.111	14.20
0.217	0.136	12.77	0.205	0.138	12.87	0.215	0.121	15.73
0.239	0.144	14.06	0.222	0.145	13.93	0.239	0.128	17.25
0.266	0.150	15.50	0.257	0.159	15.97	0.265	0.136	18.90
0.301	0.150	17.24	0.29	0.172	18.04	0.291	0.144	20.45
0.312	-0.255	9.06	0.331	0.179	20.02	0.335	0.155	22.90
0.344	-0.420	3.01	0.374	0.196	22.08	0.385	0.167	25.50
			0.415	0.202	23.61	0.43	0.177	27.6
			0.440	0.091	14.99	0.482	0.185	29.74
			0.563	-0.006	14.16	0.525	0.192	31.26
						0.592	0.206	33.13
						0.624	0.113	22.79
						0.675	0.09	20.86
						0.76	0.068	20.30

Table 4.3(d). Experimental data for $\theta=60^\circ$: Axial strain, volumetric strain, and deviatoric stress.

$\sigma_3 = 0$ MPa			$\sigma_3 = 4$ MPa			$\sigma_3 = 10$ MPa		
ϵ_a	ϵ_v	$\sigma_1 - \sigma_3$	ϵ_a	ϵ_v	$\sigma_1 - \sigma_3$	ϵ_a	ϵ_v	$\sigma_1 - \sigma_3$
(%)	(%)	(MPa)	(%)	(%)	(MPa)	(%)	(%)	(MPa)
0	0	0	0	0	0	0	0	0
0.061	0.051	2.38	0.0408	0.0355	2.47	0.037	0.039	2.28
0.094	0.0757	3.86	0.0629	0.051	3.68	0.0625	0.054	4.38
0.124	0.095	5.35	0.083	0.066	4.89	0.0936	0.072	6.74
0.155	0.113	6.98	0.106	0.081	6.31	0.125	0.086	8.85
0.188	0.128	8.77	0.124	0.092	7.36	0.158	0.098	10.88
0.218	0.141	10.41	0.168	0.112	9.98	0.190	0.111	12.80
0.25	0.152	12.11	0.212	0.129	12.44	0.222	0.121	14.7
0.28	0.162	13.65	0.25	0.144	14.47	0.252	0.131	16.35
0.312	0.17	15.25	0.292	0.156	16.59	0.288	0.141	18.32
0.342	0.177	16.63	0.337	0.167	18.66	0.318	0.149	19.83
0.374	0.184	18.01	0.377	0.177	20.31	0.355	0.157	21.61
0.405	0.189	19.32	0.42	0.184	21.98	0.385	0.163	22.93
0.45	0.195	20.89	0.461	0.191	23.45	0.443	0.173	25.37
0.472	-0.837	2.59	0.505	0.196	24.81	0.512	0.181	27.94
			0.544	0.20	26.044	0.574	0.184	29.97
			0.604	0.202	27.40	0.63	0.1846	31.68
			0.635	0.037	12.43	0.703	0.180	33.53
						0.77	0.172	35.1
						0.859	0.145	36.8
						0.894	-0.049	22.06
						0.958	-0.104	21.49

Table 4.3(e). Experimental data for $\theta=90^\circ$: Axial strain, volumetric strain, and deviatoric stress.

$\sigma_3 = 0$ MPa			$\sigma_3 = 4$ MPa			$\sigma_3 = 10$ MPa		
ϵ_a	ϵ_v	$\sigma_1 - \sigma_3$	ϵ_a	ϵ_v	$\sigma_1 - \sigma_3$	ϵ_a	ϵ_v	$\sigma_1 - \sigma_3$
(%)	(%)	(MPa)	(%)	(%)	(MPa)	(%)	(%)	(MPa)
0	0	0	0	0	0	0	0	0
0.076	0.067	2.21	0.037	0.04	2.82	0.055	0.059	1.50
0.118	0.099	4.15	0.075	0.071	5.089	0.106	0.099	2.83
0.177	0.132	7.32	0.128	0.105	7.99	0.161	0.142	5.42
0.239	0.159	10.66	0.183	0.134	10.87	0.209	0.177	7.66
0.296	0.178	13.50	0.236	0.158	13.47	0.278	0.221	10.80
0.356	0.194	16.11	0.289	0.183	15.88	0.318	0.245	12.59
0.419	0.206	18.6	0.343	0.203	18.17	0.38	0.278	15.38
0.474	0.213	20.39	0.396	0.220	20.26	0.428	0.302	17.42
0.543	0.217	22.33	0.452	0.237	22.35	0.485	0.327	19.73
0.594	0.218	23.61	0.503	0.250	24.1	0.536	0.350	21.66
0.660	0.215	25.05	0.56	0.261	25.9	0.594	0.375	23.73
0.727	0.205	26.33	0.62	0.271	27.62	0.649	0.393	25.54
0.849	0.152	28.13	0.717	0.281	30.15	0.692	0.407	26.88
0.887	-2.09	2.95	0.826	0.285	32.58	0.749	0.427	28.56
			0.935	0.279	34.6	0.857	0.456	31.45
			1.069	0.246	36.38	0.961	0.482	33.90
			1.144	-0.159	14.36	1.061	0.50	35.93
						1.173	0.521	37.71
						1.299	0.558	39.01
						1.39	0.637	36.41
						1.493	0.542	26.03

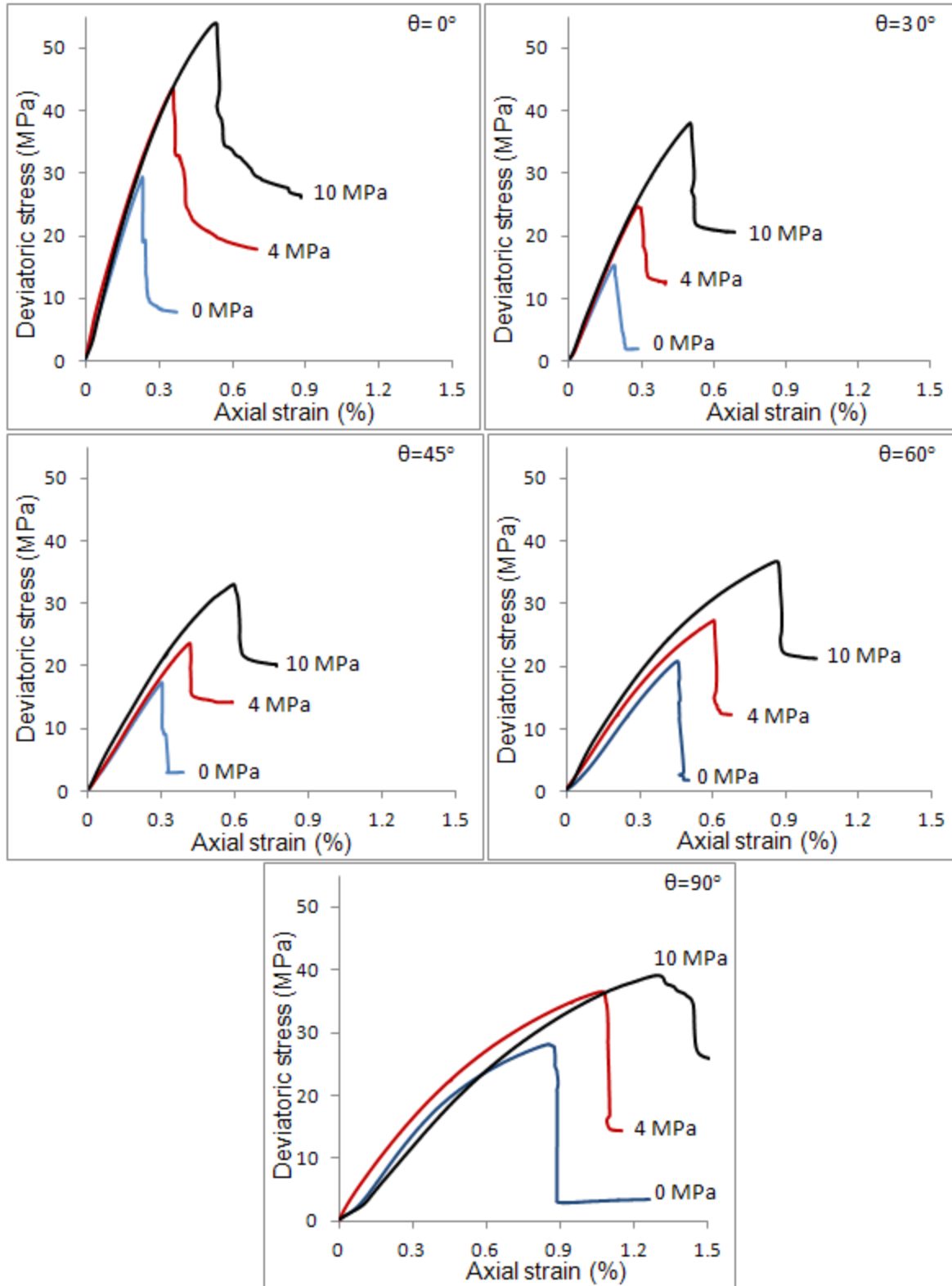


Figure 4.19. Deviatoric stress-axial strain curves for different values of θ and three different confining pressures: 0, 4, and 10 MPa (wet samples).

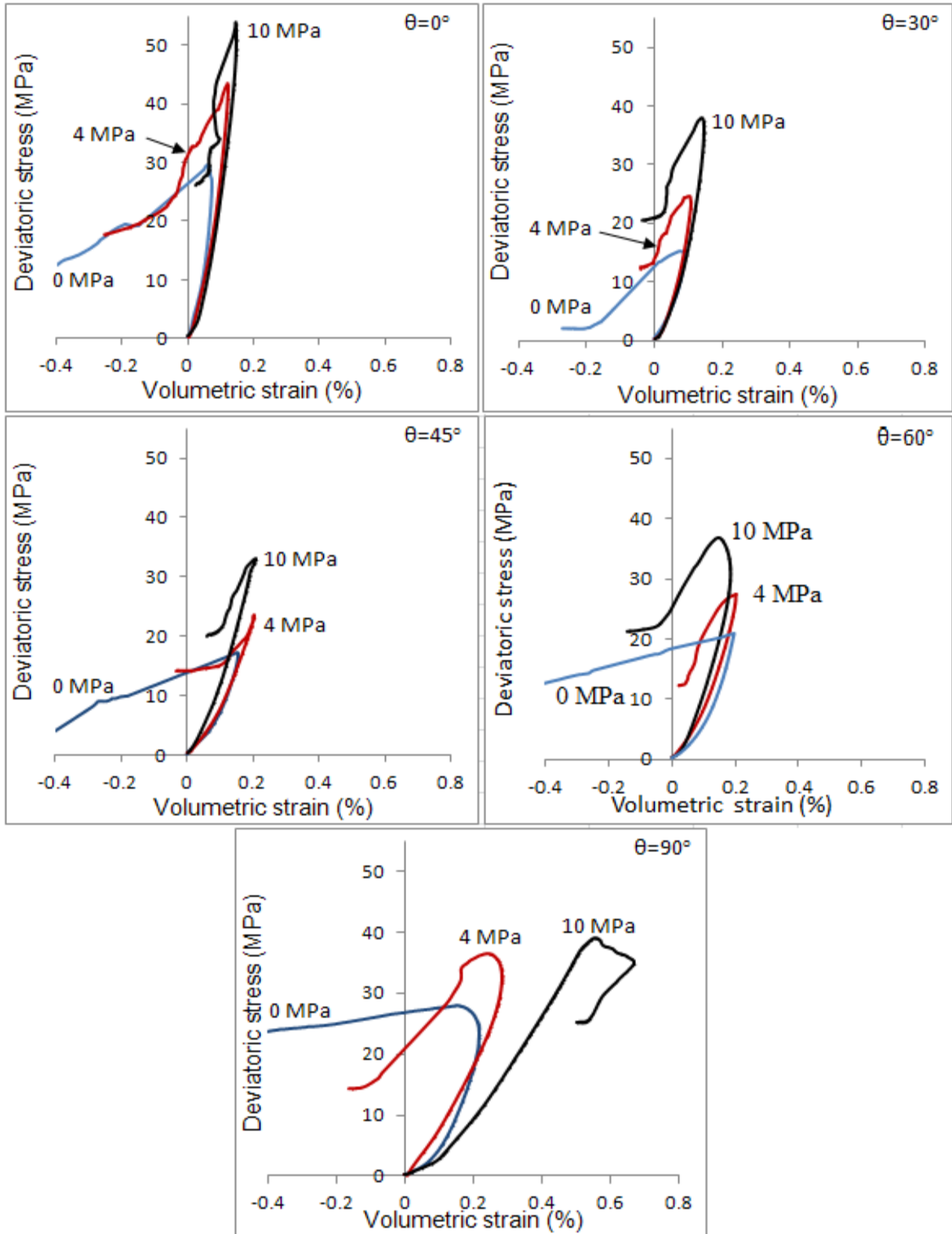


Figure 4.20. Deviatoric stress-volumetric strain curves for different values of θ and three different confining pressures: 0, 4, and 10 MPa.

4.6.8. Unconfined and confined cyclic compression tests

Unconfined and confined cyclic compression tests were carried out on wet specimens obtained from depths ranging from 2.2 m to 5.75 m. Five different loading orientations are considered: $\theta=0^\circ$, 30° , 45° , 60° , and 90° . A confining pressure of 10 MPa was applied in the confined cyclic tests. The main objective of running these cyclic tests was to investigate the cyclic behaviour of the Tournemire argillite and to determine whether the development of plastic strains would cause some degradation in the elastic properties of the rock.

The obtained stress-strain curves are shown in Figs. 4.21 and 4.22 for unconfined cyclic compression tests, and in Figs. 4.23 and 4.24 for confined cyclic compression tests. In most experiments, the unloading-reloading cycles were imposed at various stress levels before the peak strength was reached. There were also a few cyclic tests conducted, whenever it was possible, in the post-peak region. The number of cycles varied from one test to another. For a given loading-unloading cycle, a strain rate was used to set the upper limit of the cycle and a pre-determined stress value was used to set the lower limit of the cycle. In some tests, the data was not captured completely after the peak by the MTS machine as in the case for $\theta=0^\circ$.

The experimental data indicate that the cyclic behaviour of the material depends on whether cycling is carried out before or after the peak strength. Cycling before the peak strength resulted in the following observations:

- More plastic strain develops within the specimen with each additional cycle. The amount of plastic strain varies between $\frac{1}{3}$ and $\frac{1}{2}$ of the total strain, and depends on θ and confining pressure.
- The degradation to the elastic modulus increases with the increase in plastic strain. The degree of degradation depends on θ and confining pressure.
- Each cycle of loading and unloading causes more contraction in the specimen.

The experimental observations indicate that the orientation and the number of the developed fractures within the rock specimen is mainly influenced by the loading orientation angle. After the peak strength, the specimen breaks into two or more blocks (depending on the loading

orientation angle) and each additional cycle results in more sliding of one block on another and more dilation of the specimen.

In this small number of experiments with a limited number of load cycles and loading range, it was observed that the load cycling has no significant effect on the strength and failure mode of the rock. Small hysteresis was observed in these tests. The hysteresis phenomenon is the result of viscous properties of the argillite. The compressibility-dilatancy transition occurs near the peak strength as in uniaxial tests. The plastic strain is anisotropic as indicated by the results.

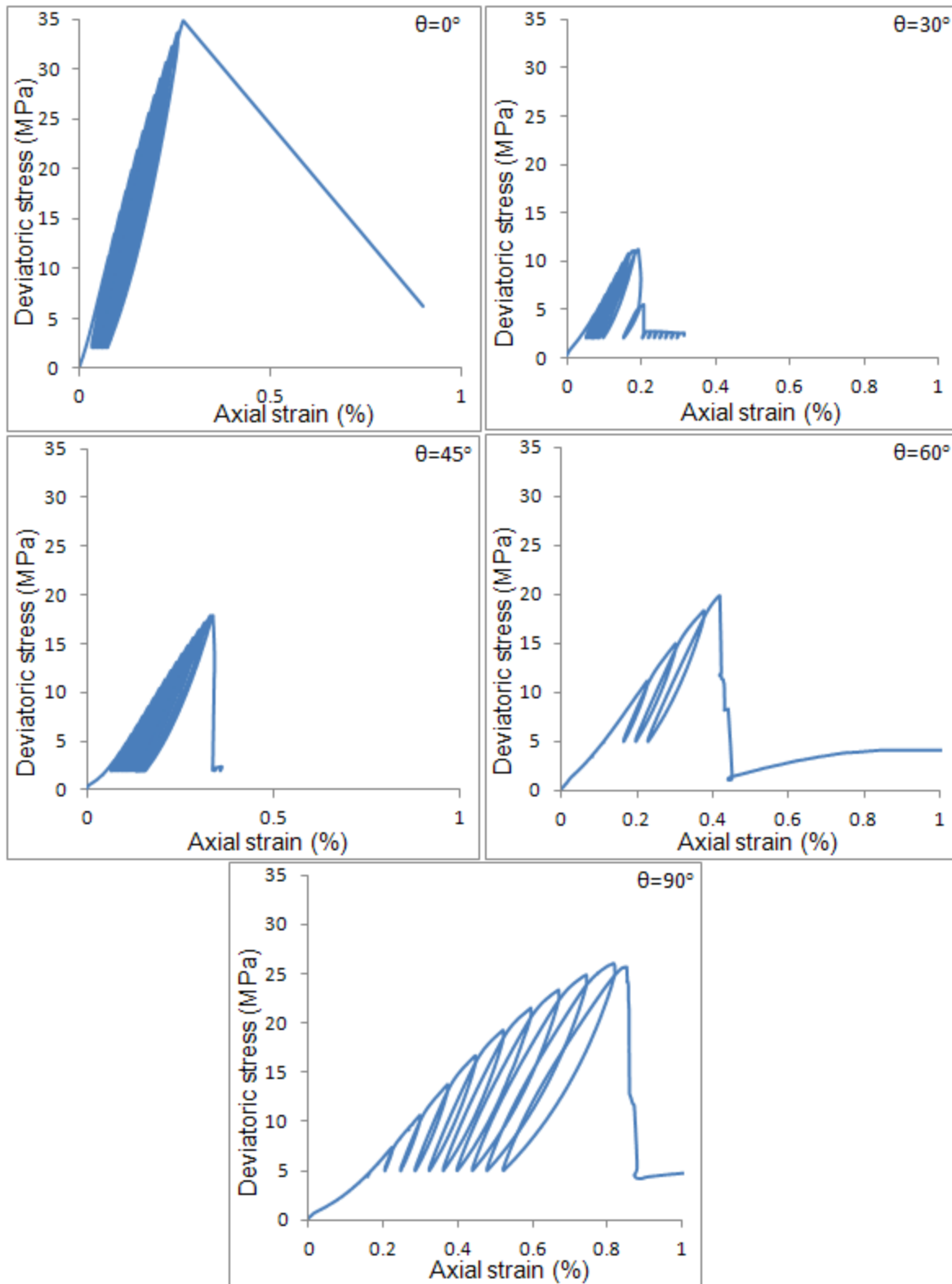


Figure 4.21. Deviatoric stress-axial strain curves for various values of θ (unconfined tests wet samples).

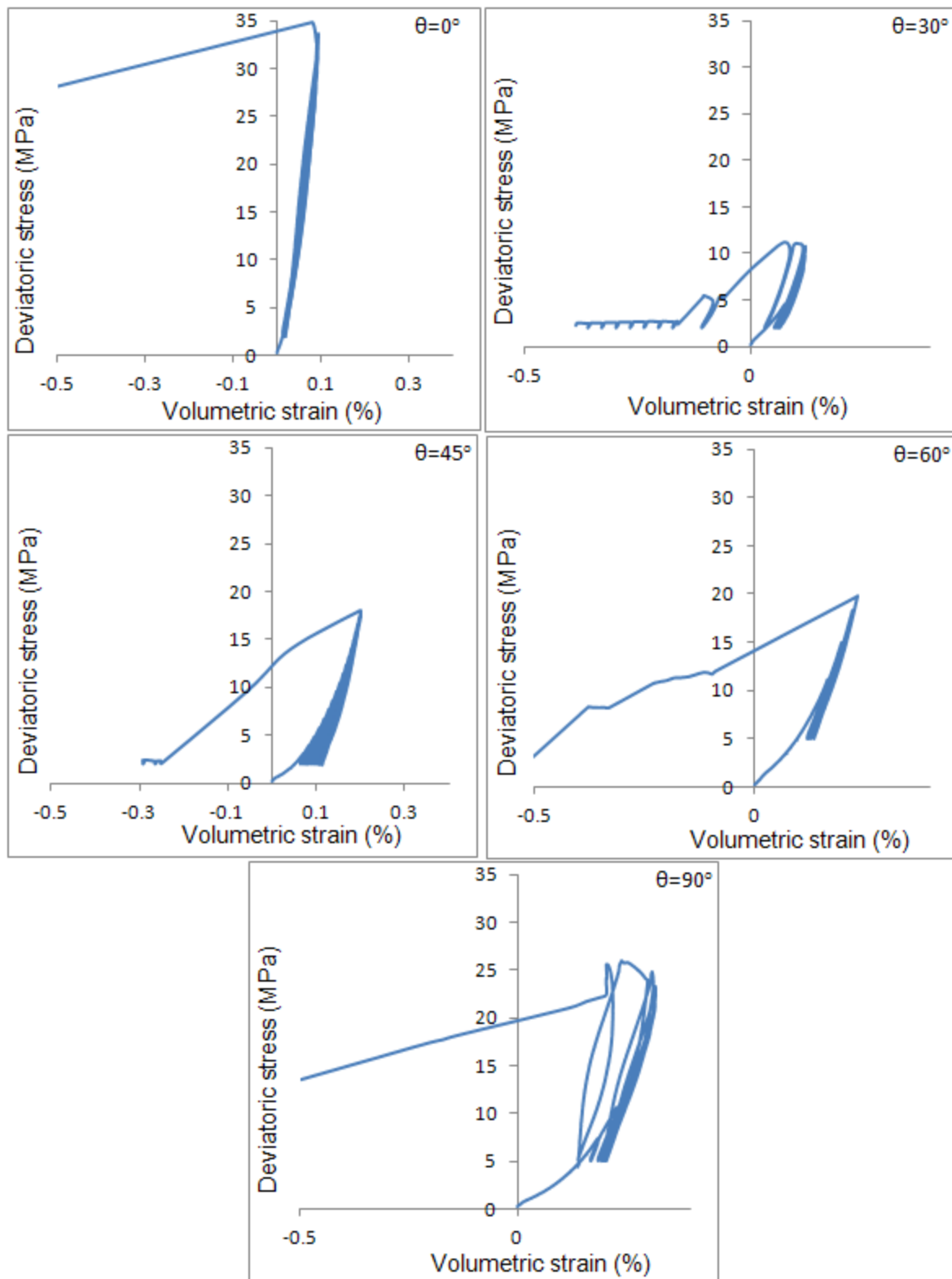


Figure 4.22. Deviatoric stress-volumetric strain curves for various values of θ (unconfined tests).

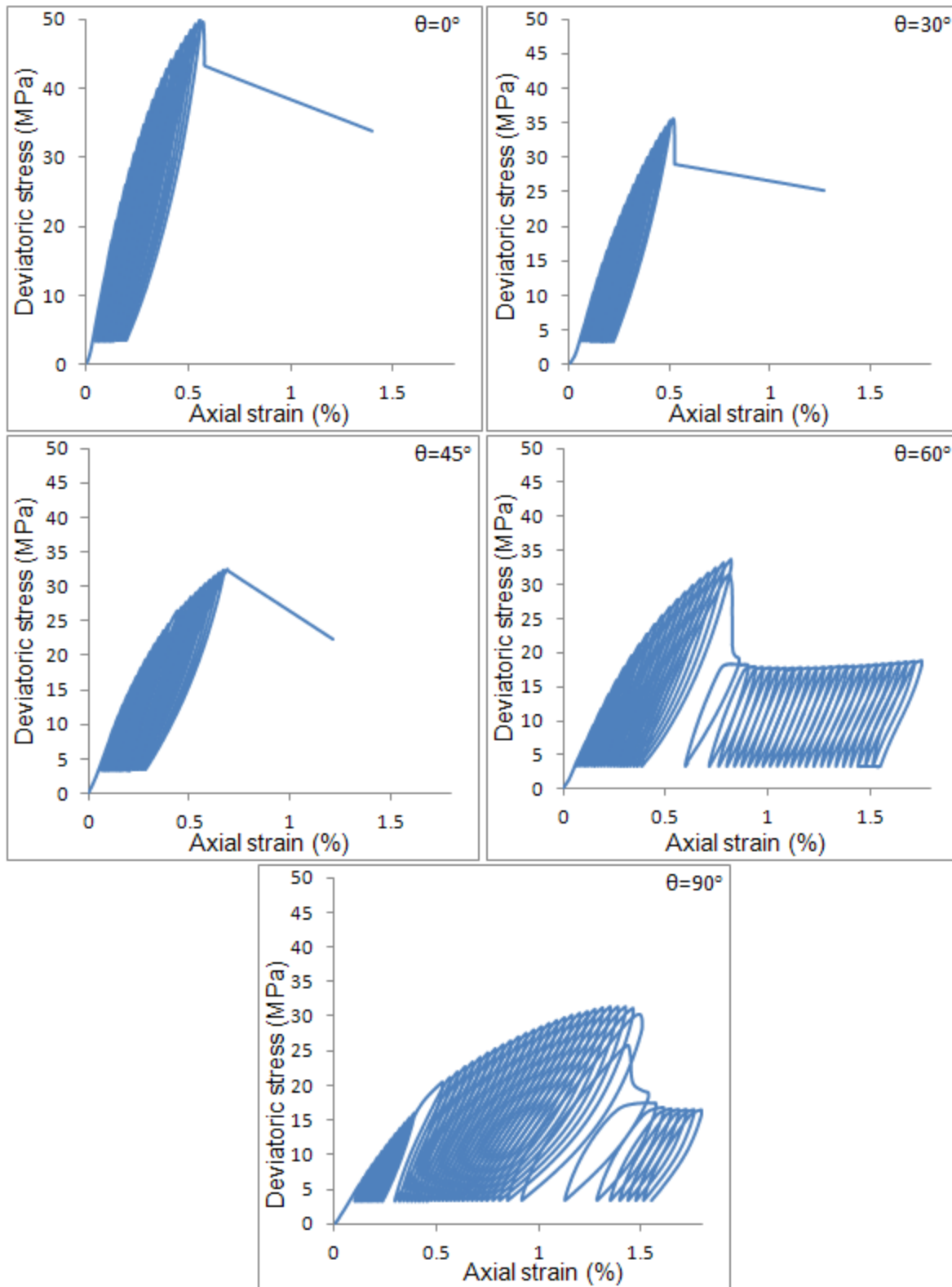


Figure 4.23. Deviatoric stress-axial strain curves for various values of θ and a confining pressure of 10 MPa (wet samples).

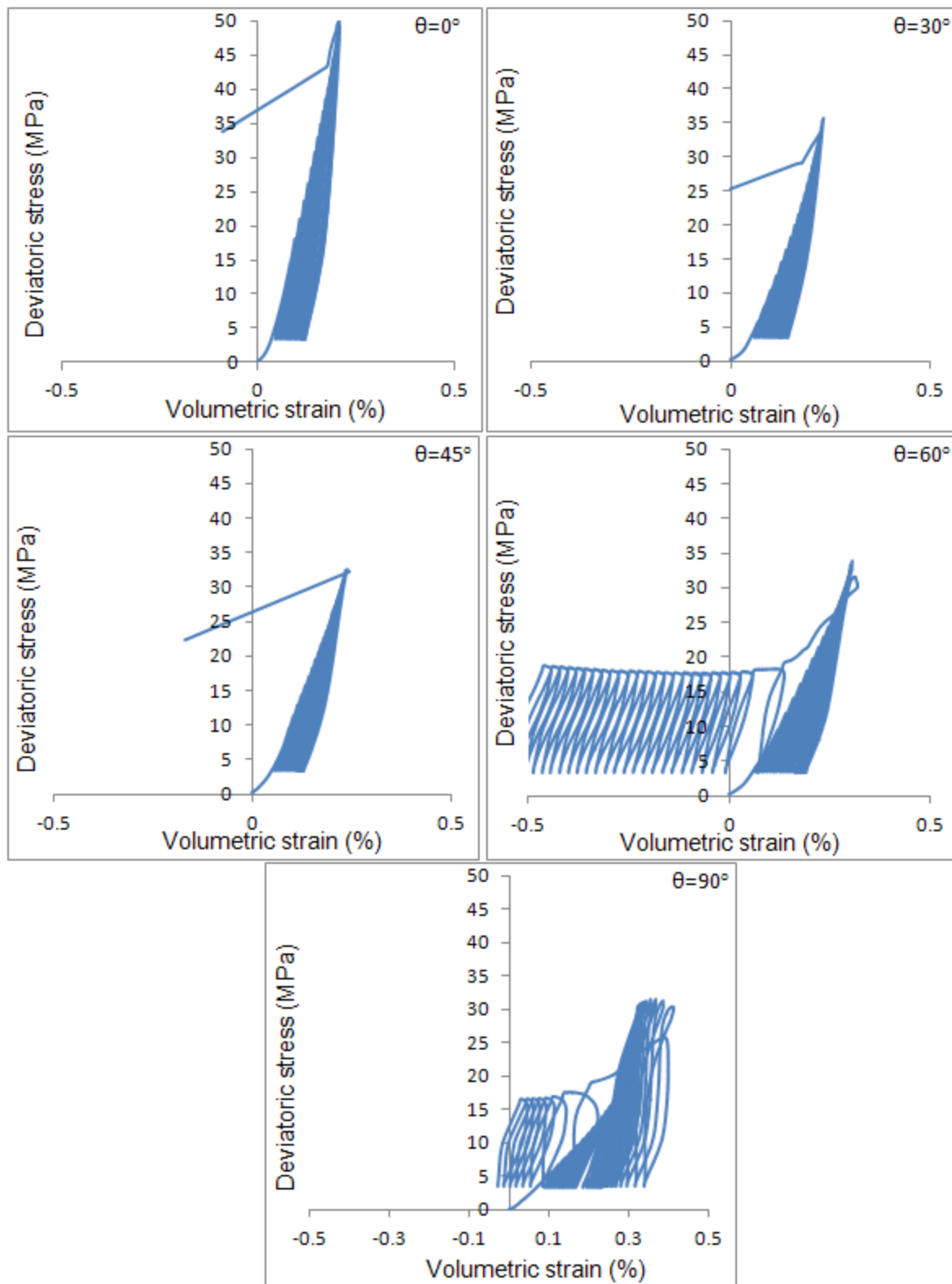


Figure 4.24. Deviatoric stress-volumetric strain curves for various values of θ and a confining pressure of 10 MPa.

4.6.9. Creep tests

Argillaceous formations may exhibit creep behaviour especially along bedding planes which may be filled with different type of minerals. Here, a series of short-term creep tests have been carried out to investigate the creep behaviour of the Tournemire argillite. The specimens with natural water content used in these tests were obtained from five different boreholes ($\theta=0^\circ$, 30° , 45° , 60° , and 90°). The depths of the specimens vary from 4.7 m to 5.55 m. In each test, first the deviatoric stress is increased to a predefined stress level followed by a 1-hour period of waiting time in which the deviatoric stress is kept constant and the axial strain is measured. After, the deviatoric stress is increased to another predefined stress level followed by a 1-hour period of waiting time in which the deviatoric stress is kept constant and the axial strain is also measured. This process is repeated until the failure state is reached.

The obtained deviatoric stress-axial strain curves and axial strain-time curves are shown in Figs 4.25 and 4.26 for all loading orientations, respectively. As indicated above, the crack initiation level (stable crack growth) and the crack damage level (unstable crack growth) occur at stress levels of about $\sigma_{ci} = 70\% \sim 75\%$ of the peak stress and $\sigma_{cd} = 85\% \sim 90\%$ of the peak stress, respectively. The experimental results indicate that the results of the creep tests can be interpreted in terms of σ_{cd} . If the deviatoric stress is below σ_{cd} , only a small increase in axial strain is measured during the creep period before a stable state (no increase in axial strain with time) is reached. When the deviatoric stress exceeds the σ_{cd} , the increase in axial strain during creep period becomes more significant. After 1 hour period of time, the axial strain still increasing and this could be due to unstable crack growth under constant stress. At this stress level, a longer period of time can cause the failure of the material. It seems the period of time applied in these tests (1 hour in each step) is not enough and it should be increased especially when the deviatoric stress becomes greater than the σ_{cd} . Compared to the results of uniaxial tests, larger axial strains are measured at peak in creep tests.

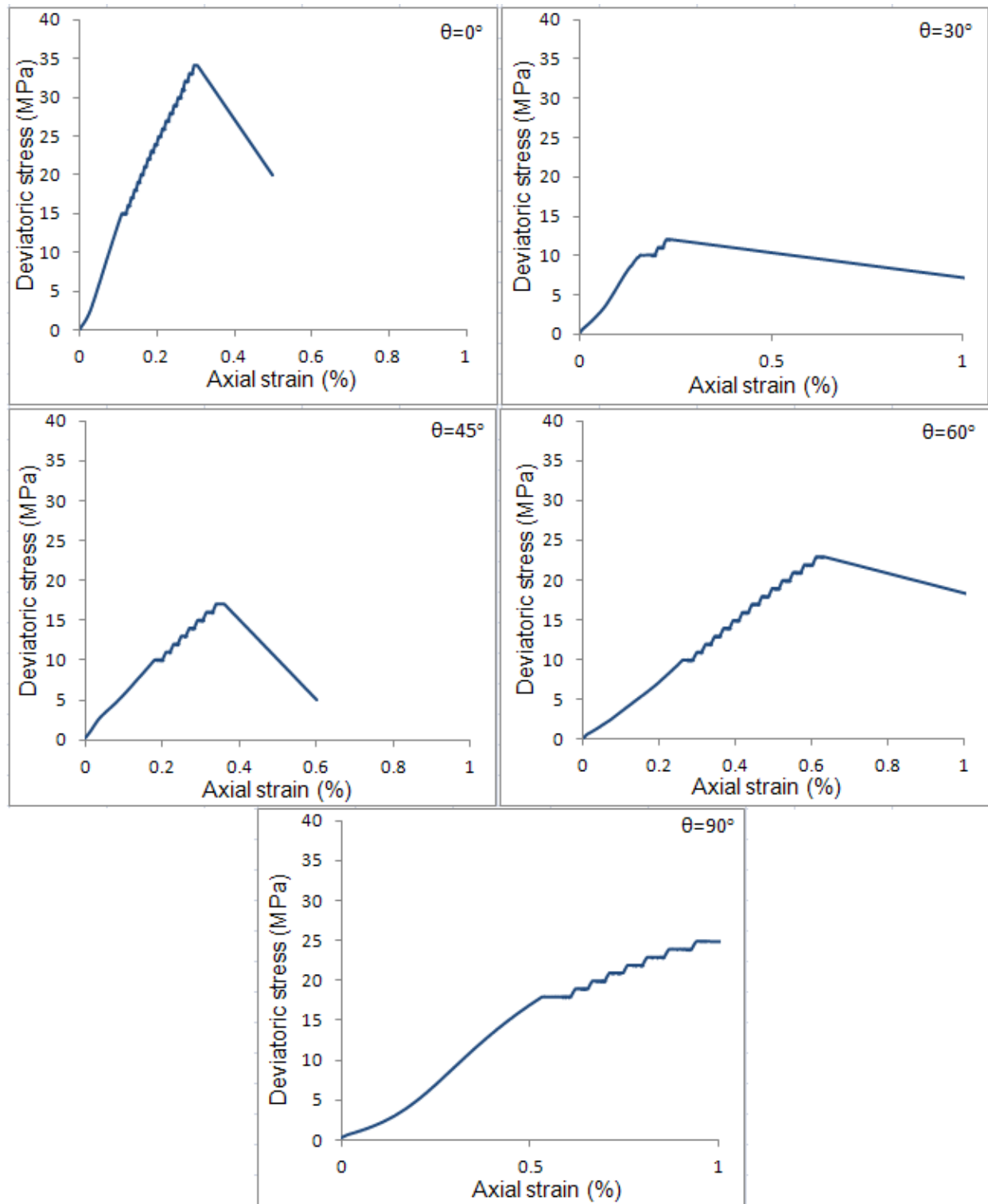


Figure 4.25. Deviatoric stress-axial strain curves for different loading orientations (wet samples).

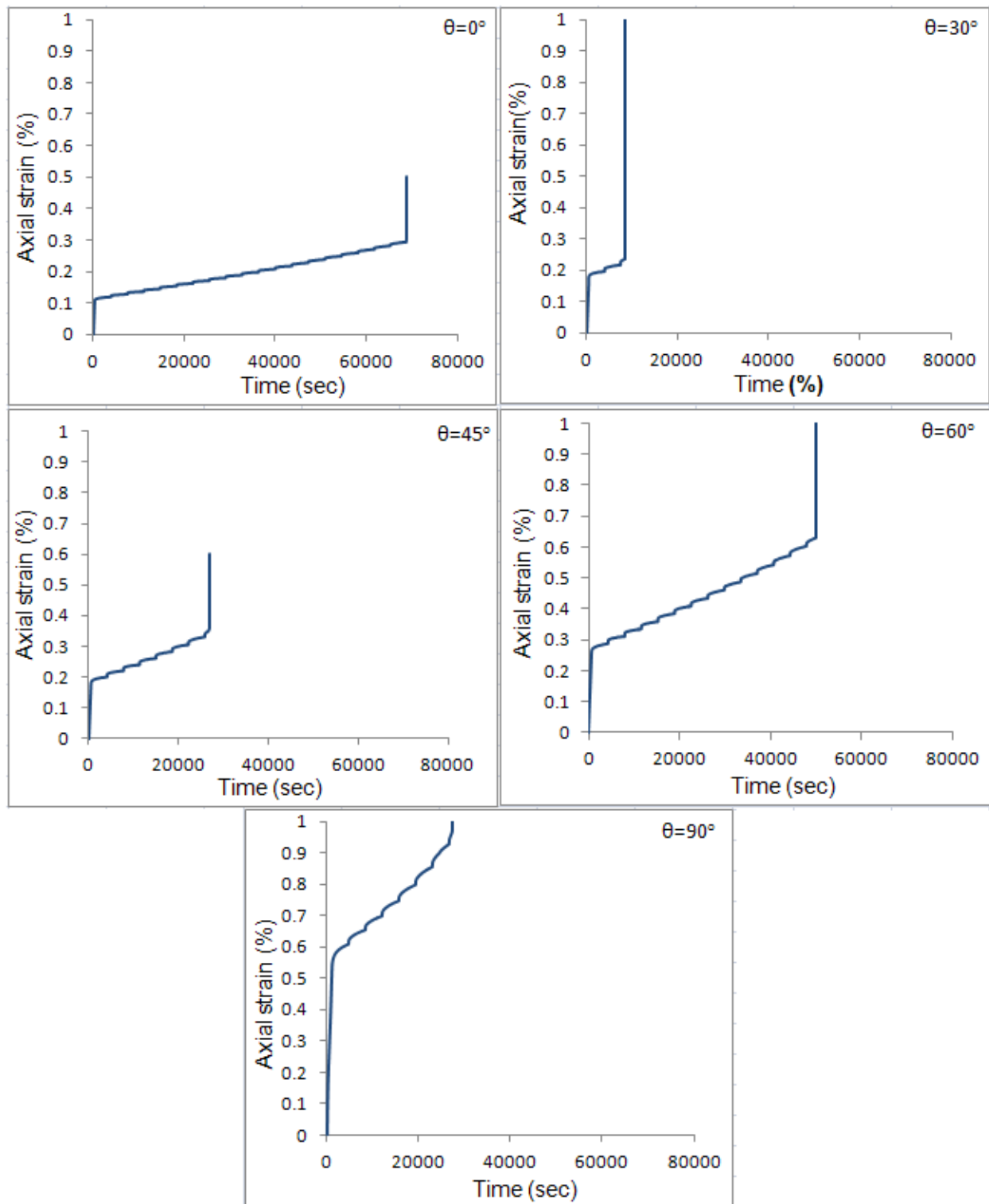


Figure 4.26. Axial strain-time curves for different loading orientations.

CHAPTER 5

INTERPRETATION OF THE EXPERIMENTAL RESULTS

5.1. Background

The main purpose of this Chapter is to interpret the experimental results described in Chapter 4. This includes crack initiation and propagation within the argillite, evaluation of the elastic and elastoplastic response of the argillite, failure behaviour of the argillite, establishing strength parameters, and comparing the results with previously published experimental data.

Many parameters such as loading rate, sample size, water content, capabilities of the testing machine, testing protocols, etc, can affect the results of the experiments. Therefore, comparison of the present experimental data with previously published experimental data is not a straightforward process. However, the same kind of trends in present test results is expected.

5.2. Analysis of the Mechanical Behaviour of the Tournemire argillite

In this section: first the crack closure threshold, crack initiation threshold, and crack damage threshold of the rock are shown. Subsequently, the interpretation of the experimental results is described.

5.2.1. Closure, initiation, and propagation of microcracks

Figure 5.1 shows the crack closure zone, crack initiation stress level, and crack damage stress level for $\theta=90^\circ$. This figure also shows volume change, acoustic emission events along the stress-strain relationship, crack closure volume, and the volume of newly developed microcracks. In this test, the closure of the initial microcracks is achieved at a stress level of about 10% of the peak strength. For other loading orientations, the closure of the initial microcracks is achieved at stresses less than 10% of the peak strength. The initiations of new microcracks start at a stress level of about 70% ~ 75% of the peak strength. The crack damage stress level is reached at about 85% ~ 90% of the peak strength. Similar behaviour is observed for other loading orientations.

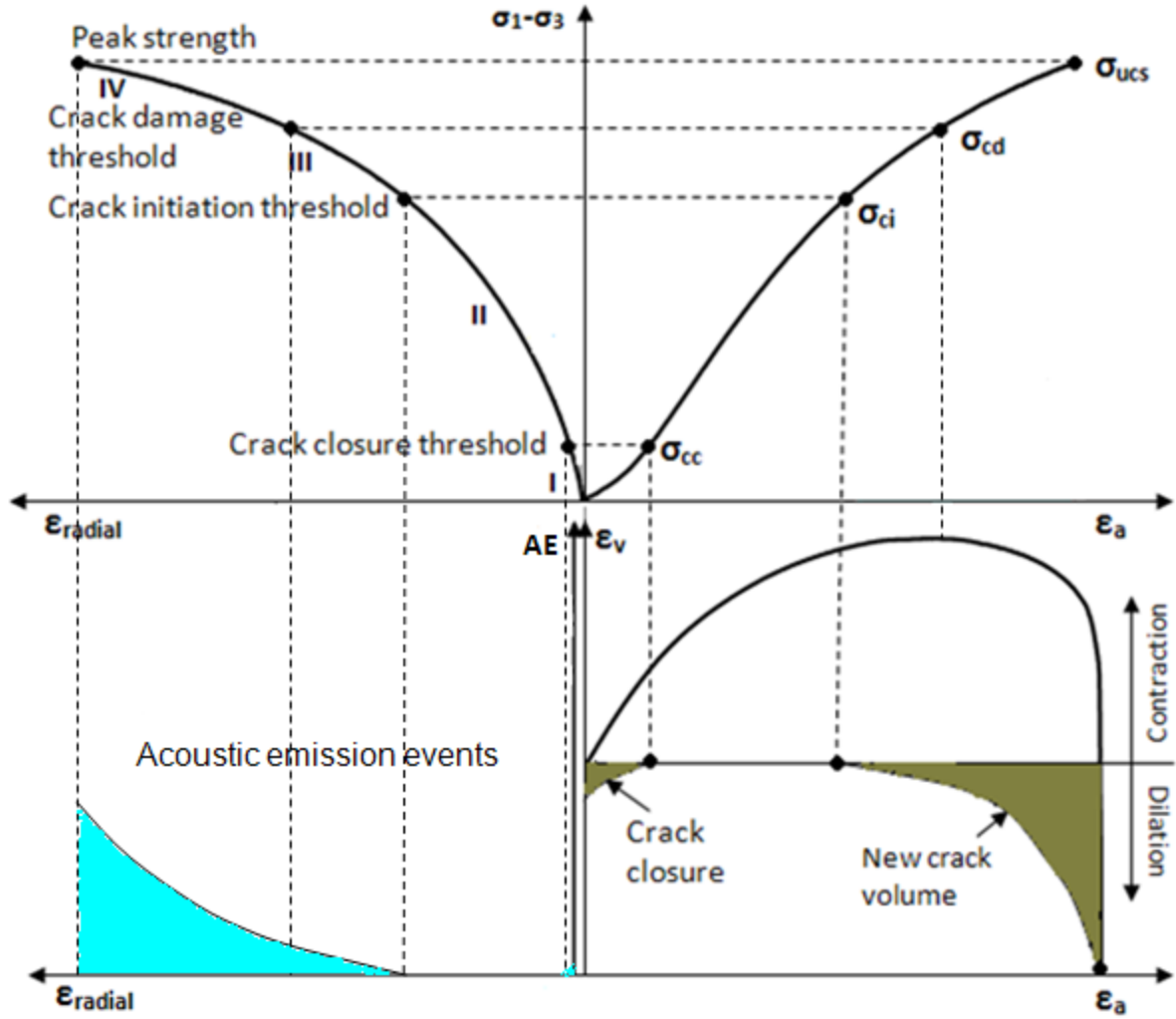


Figure 5.1. Closure, initiation, and propagation of microcracks for the Tournemire argillite.

5.2.2. Stress-strain relationship

Literature survey shows that, for the purpose of developing constitutive models for hard rocks, the experimentally obtained deviatoric stress-axial strain curves are generally divided into four zones: microcrack/bedding closure zone, elastic zone, elastoplastic zone with hardening, and a post-peak zone where the stress level sharply reduces to a low value as shown in Fig. 5.2(a). Loading and unloading in the elastic zone would follow the same curve without producing any plastic strains. Somewhat similar deviatoric stress-axial strain curves were plotted for the

Tournemire argillite. However, cyclic test results show that a purely elastic zone does not exist during monotonic loading in the case of Tournemire argillite, and the corresponding four zones are shown in Fig. 5.2(b). Plastic strains start to develop right from the beginning of the stress-strain curve. Therefore, the parameters of elastic stress-strain relation must be determined from the results of cyclic tests. A purely elastic behaviour of the argillite exists only during the unloading-reloading cycles.

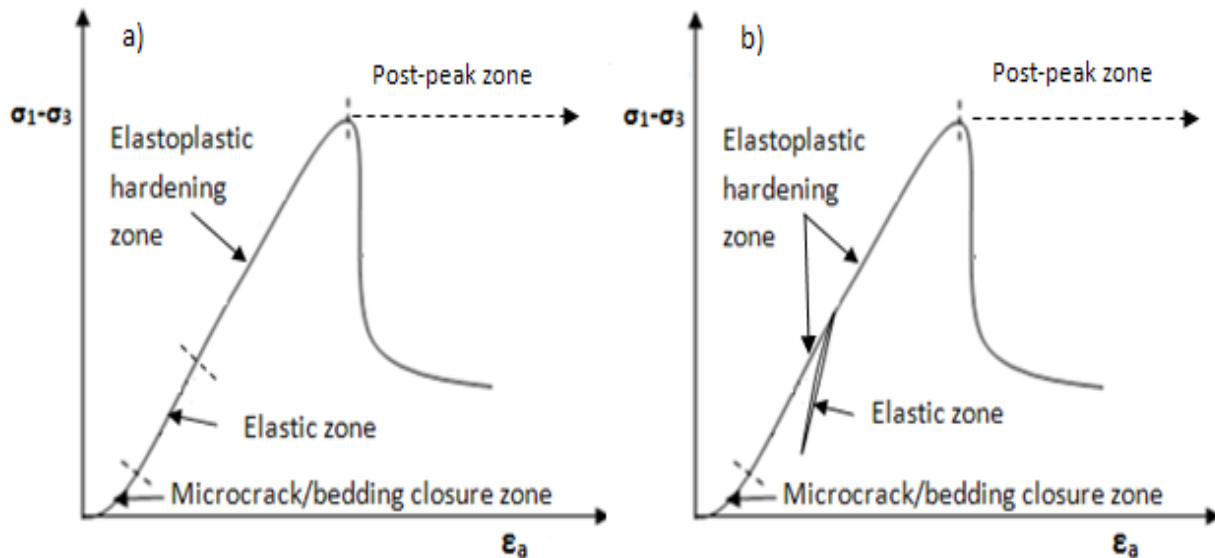


Figure 5.2: Schematic description of the stress-strain behaviour: a) hard rock, and b) Tournemire argillite.

5.2.2.1. Microcrack/bedding closure zone

Sampling, transportation, and storage of rock specimens may cause an increase in the size and number of the microcracks and opening of bedding planes. During testing, the initial increase in the stress level causes the closure of the existing microcracks and bedding planes. The closure of microcracks and the compression of the bedding planes is more pronounced at $\theta=90^\circ$ than those in other loading orientations. However, such a “closure zone” in the stress-strain curves is less significant in the experiments with higher confining pressures as most initial microcracks and bedding planes are closed during the application of the confining pressure. The volume change in this zone is mainly compressive.

5.2.2.2. Elastic zone in cyclic tests

The elastic parameters of the Tournemire argillite were obtained from a straight line connecting the points representing the beginning of the unloading and reloading portions of the first cycle. Experimental data show that these parameters are largely influenced by the loading orientation angle, θ . The Tournemire argillite can be classified as a transversely isotropic material, and its elastic behaviour can be described by the following constitutive relation.

$$\begin{bmatrix} d\varepsilon_1 \\ d\varepsilon_2 \\ d\varepsilon_3 \\ d\gamma_{23} \\ d\gamma_{31} \\ d\gamma_{12} \end{bmatrix} = \begin{bmatrix} \frac{1}{E_1} & -\frac{\nu_{12}}{E_1} & -\frac{\nu_{12}}{E_1} & 0 & 0 & 0 \\ -\frac{\nu_{12}}{E_1} & \frac{1}{E_2} & -\frac{\nu_{23}}{E_2} & 0 & 0 & 0 \\ -\frac{\nu_{12}}{E_1} & -\frac{\nu_{23}}{E_2} & \frac{1}{E_2} & 0 & 0 & 0 \\ 0 & 0 & 0 & \frac{1}{G_{23}} & 0 & 0 \\ 0 & 0 & 0 & 0 & \frac{1}{G_{31}} & 0 \\ 0 & 0 & 0 & 0 & 0 & \frac{1}{G_{12}} \end{bmatrix} * \begin{bmatrix} d\sigma_1 \\ d\sigma_2 \\ d\sigma_3 \\ d\tau_{23} \\ d\tau_{31} \\ d\tau_{12} \end{bmatrix} \quad [5.1]$$

where the axis 1 is perpendicular to bedding planes, and axes 2 and 3 are parallel to bedding planes.

The above elastic compliance matrix depends on five independent parameters: E_1 and ν_{12} that can be obtained from tests performed at $\theta=90^\circ$, E_2 and ν_{23} that can be obtained from tests performed at $\theta=0^\circ$, and the shear modulus G_{12} that can be obtained from a test performed at an angle $0^\circ < \theta < 90^\circ$ and using the following equation (Niandou et al., 1997):

$$G_{12} = \left(\frac{1}{E_\theta \sin^2 \theta \cos^2 \theta} - \frac{\cos^2 \theta}{E_2 \sin^2 \theta} - \frac{\sin^2 \theta}{E_1 \cos^2 \theta} + \frac{2 \nu_{12}}{E_1} \right)^{-1} \quad [5.2]$$

where E_θ is the elastic modulus which is determined from a test carried out in the θ orientation.

Elastic parameters

The unconfined cyclic tests are used to estimate the elastic parameters. In each test, the first unloading-reloading cycle is considered for estimating the parameters. From the cyclic test ($\theta=90^\circ$) on a rock specimen from borehole M9, the following two elastic parameters are obtained:

$$E_1 = 12.5 \text{ GPa}$$

$$\nu_{12} = 0.115$$

From the cyclic test ($\theta=0^\circ$) on a rock specimen from borehole M13, the following two elastic parameters are obtained:

$$E_2 = 21 \text{ GPa}$$

$$\nu_{23} = 0.15$$

In the framework of the theory of elasticity, the symmetry of the elastic matrix should be verified, e.g., $\frac{\nu_{12}}{E_1} = \frac{\nu_{21}}{E_2}$. This gives $\nu_{21} = 0.19$. Using Equation [5.2] and the value of $\theta=45^\circ$ ($E_\theta = 12 \text{ GPa}$) result in $G_{12} = 4.5 \text{ GPa}$.

The experimental data showed that the elastic modulus in the direction of loading (E_z) varied with the stress level at which the unloading–reloading cycle was applied. Figure 5.3 shows the variation of E_z with the normalized deviatoric stress. In order to see the effect of loading orientation on E_z , the variation of E_z , estimated at 70% $\sigma_d/\sigma_{d\text{-peak}}$, with θ is shown in Fig. 5.4. Even though there is no big difference in the results of tests conducted at $\theta=30^\circ$, 45° , and 60° , the experimental data indicate that E_z is decreasing with the increase in θ .

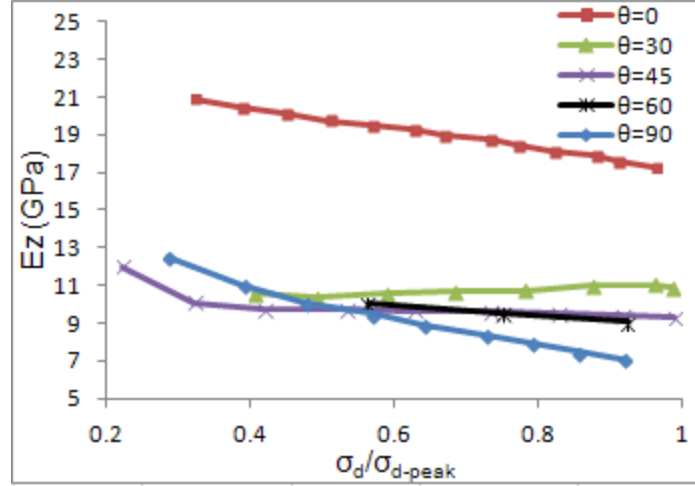


Figure 5.3. Variation of E_z , estimated from unloading-reloading cycles, with σ_d/σ_{d-peak} .

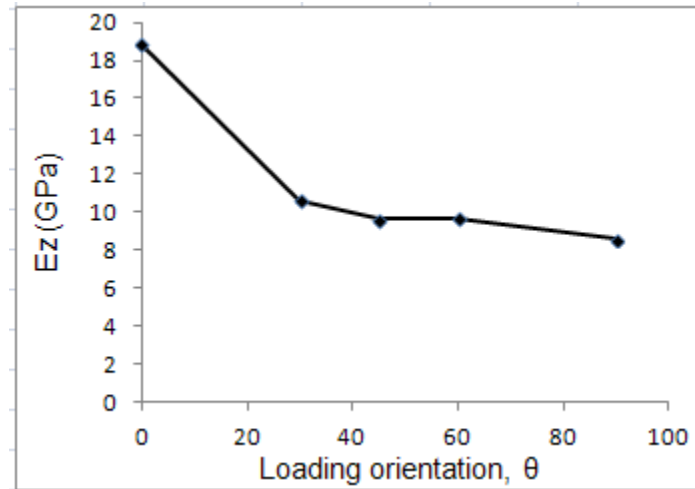


Figure 5.4. Variation of E_z , estimated at 70% σ_d/σ_{d-peak} , with θ .

5.2.2.3. Elastoplastic hardening zone

For the Tournemire argillite, the elastoplastic hardening zone represents the stress-strain curve from the end of the “microcrack/bedding closure zone” up to the peak strength. The amount of plastic strain that can develop in this zone depends on the stress level, loading orientation angle, and confining pressure. The volumetric strain in this zone is mainly contraction. AE data indicate that microcracks initiate in this section, at $\sigma_{ci} = 70\% \sim 75\%$ of the σ_{peak} . Above σ_{ci} , the formation and propagation of microcracks accelerate with the increase in stress level.

Figure 5.5 shows the variation of the axial strain at the peak strength with θ for different values of confining pressure. The axial strain at the peak strength increases with the increase in confining pressure. Small values of θ approximately less than 30° ($\theta < 30^\circ$) have no significant effect on the values of the axial strain at the peak strength. However, larger values of θ ($> 30^\circ$) have a significant effect on the axial strain at the peak strength. The axial strain at the peak strength increases with the increase in θ . These experimental results indicate that the compressibility of the bedding planes is higher than the compressibility of the rock outside the bedding planes. The effect of the compressibility of bedding planes on the deformation behaviour of the argillite can be summarized as follows.

- For $\theta \leq 30^\circ$: In this case, the bedding planes are not exposed directly to the applied stress. Most of the load is supported by the rock outside the bedding planes and the contribution of the deformation of bedding planes to the deformation behaviour of the rock is minimal.
- For $\theta > 30^\circ$: In this case, the bedding planes are exposed to the applied stress. The contribution of the deformation of bedding planes to the deformation behaviour of the rock becomes important. As a result, larger axial strains are measured at the peak strength compared to the values measured in the first case, i.e. $\theta \leq 30^\circ$.

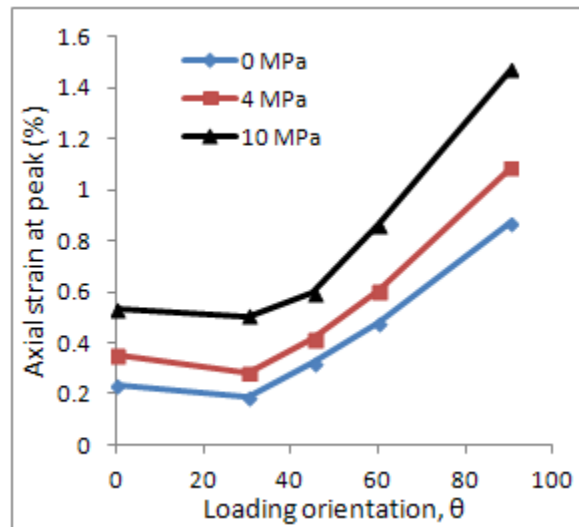


Figure 5.5: Variation of the axial strain at the peak strength with θ for different values of confining pressure: 0, 4, and 10 MPa.

The above results could be explained as shown in Fig. 5.6:

- For $\theta \leq 30^\circ$ (Fig. 5.6.a), the bedding planes are not exposed directly to the applied stress and their effect on the deformation behaviour of the material is minimal.
- For $\theta > 30^\circ$ (Fig. 5.6.b), the bedding planes are exposed directly to the applied stress and their effect on the deformation behaviour of the material is important.

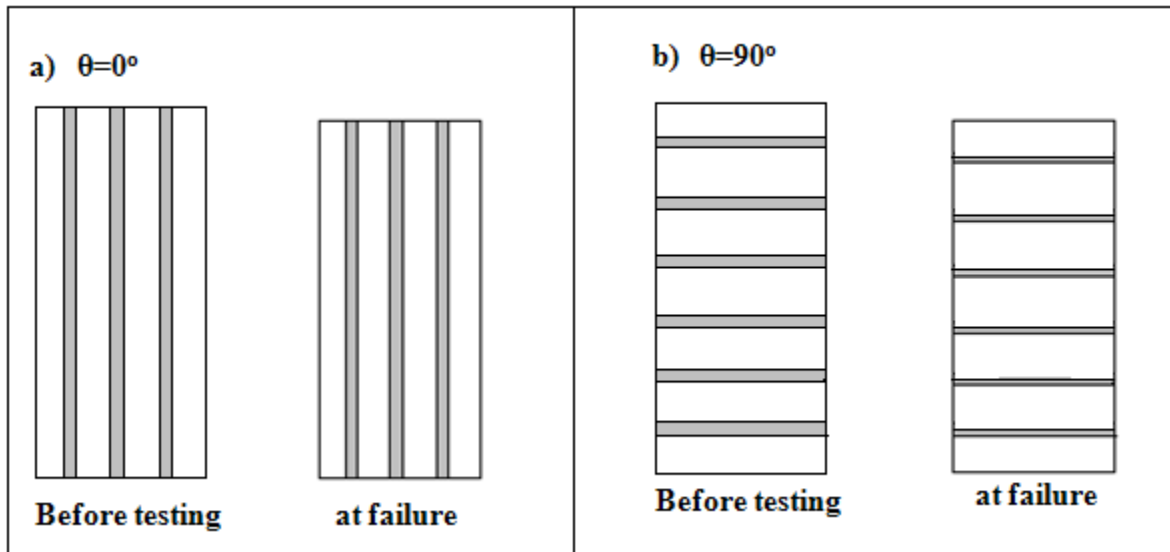


Figure 5.6. The effect of the compressibility of the bedding planes on the deformation behaviour of the argillite.

Figure 5.7 shows the values of the peak strength obtained in uniaxial and triaxial tests. The peak strength depends on the loading orientation angle, θ , and confining pressure. For the range of confining pressure considered, the maximum strength was obtained in tests performed at $\theta=0^\circ$. The strength obtained in tests performed at $\theta=90^\circ$ was smaller than the strength at $\theta=0^\circ$. For the uniaxial tests, the minimum strength is obtained at $\theta=30^\circ$. The increase in confining pressure causes a shift in this angle to the right, towards $\theta=45^\circ$. The results indicate that the compressive strength is highly sensitive to the load inclination and the shear strength characteristics of bedding planes.

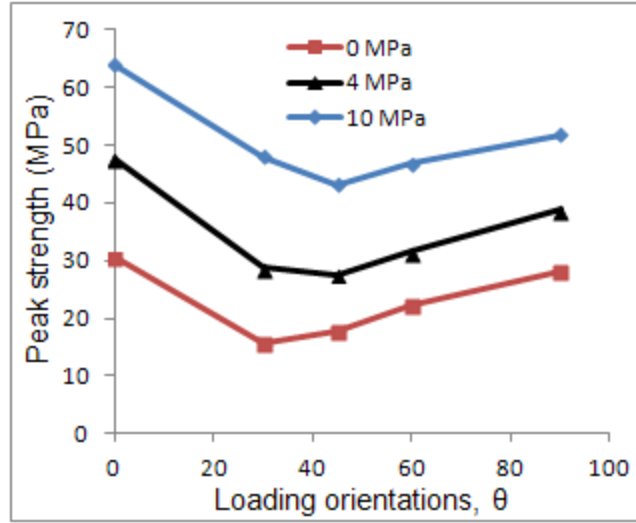


Figure 5.7: Variation of the peak strength with θ for different values of confining pressure: 0, 4, and 10 MPa.

5.2.2.4. *Post-peak zone*

The Tournemire argillite behaves as a brittle material. A sudden collapse of the material, e.g. moving from peak strength towards a small value of strength measured in the post-peak region, is observed in uniaxial, triaxial, and cyclic tests. Figure 5.8 shows the values of strength in the post-peak region obtained in uniaxial and triaxial tests. These values are a function of the loading orientation angle, θ , and confining pressure. The highest value is measured in tests performed at $\theta=0^\circ$, whereas the lowest values are measured in tests performed at $30^\circ \leq \theta \leq 60^\circ$.

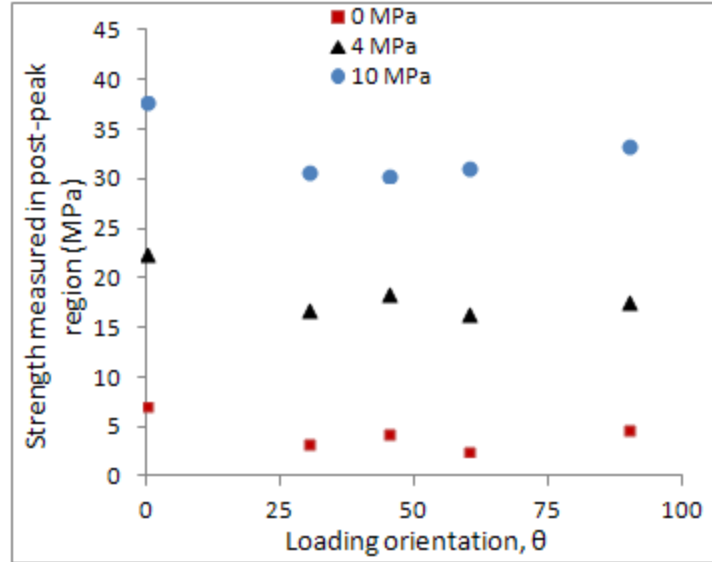


Figure 5.8: Variation of the strength measured in the post-peak region with θ for different values of confining pressure: 0, 4, and 10 MPa.

5.3. Effect of confining pressure on the elastic modulus

Figure 5.9 shows a couple of cycles of unconfined cyclic tests and confined cyclic tests ($\sigma_3 = 10$ MPa). The results indicate that the range of confining pressure considered in this testing program does not have an effect on the elastic modulus of the rock. However, the combination of confining pressure and inclination of bedding planes has some effect. For example, the confining pressure has some effect at $\theta = 30^\circ$ and 45° . Only a negligible effect is observed at $\theta = 0^\circ$, 60° , and 90° .

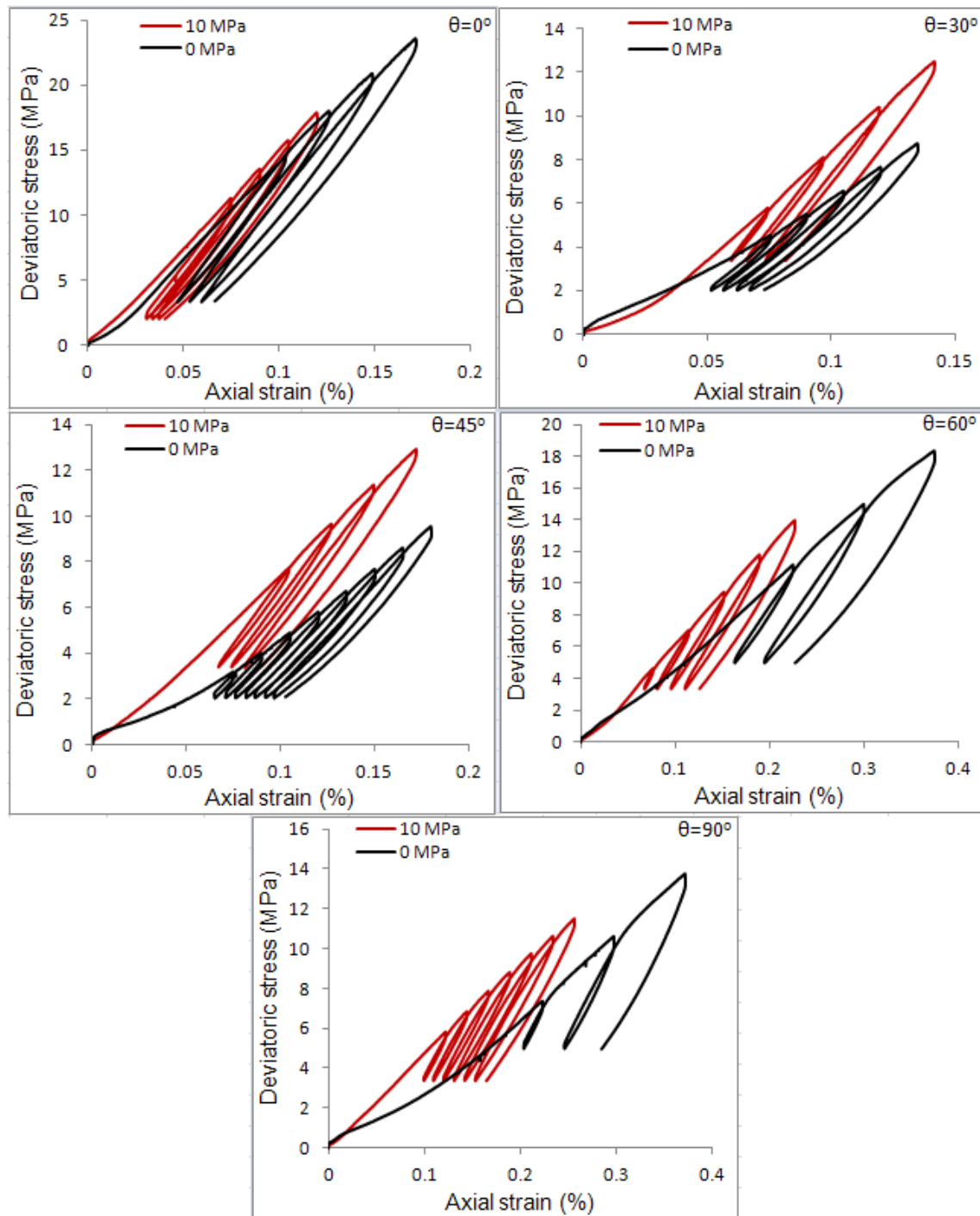


Figure 5.9. Effect of confining pressure on the elastic modulus.

5.4. Development of plastic strains

Unloading-reloading provided an opportunity to calculate the plastic strain. Figure 5.10 describes the development of plastic strain during loading rock specimens at different loading orientation angle and for two different confining pressures: 0 MPa and 10 MPa. The plastic strain represents between two-thirds ($2/3$) and one-half ($1/2$) of the total strain (depending on the loading orientation angle). The plastic strain increases with the increase in the loading orientation angle, θ , as well as with the increase in confining pressure.

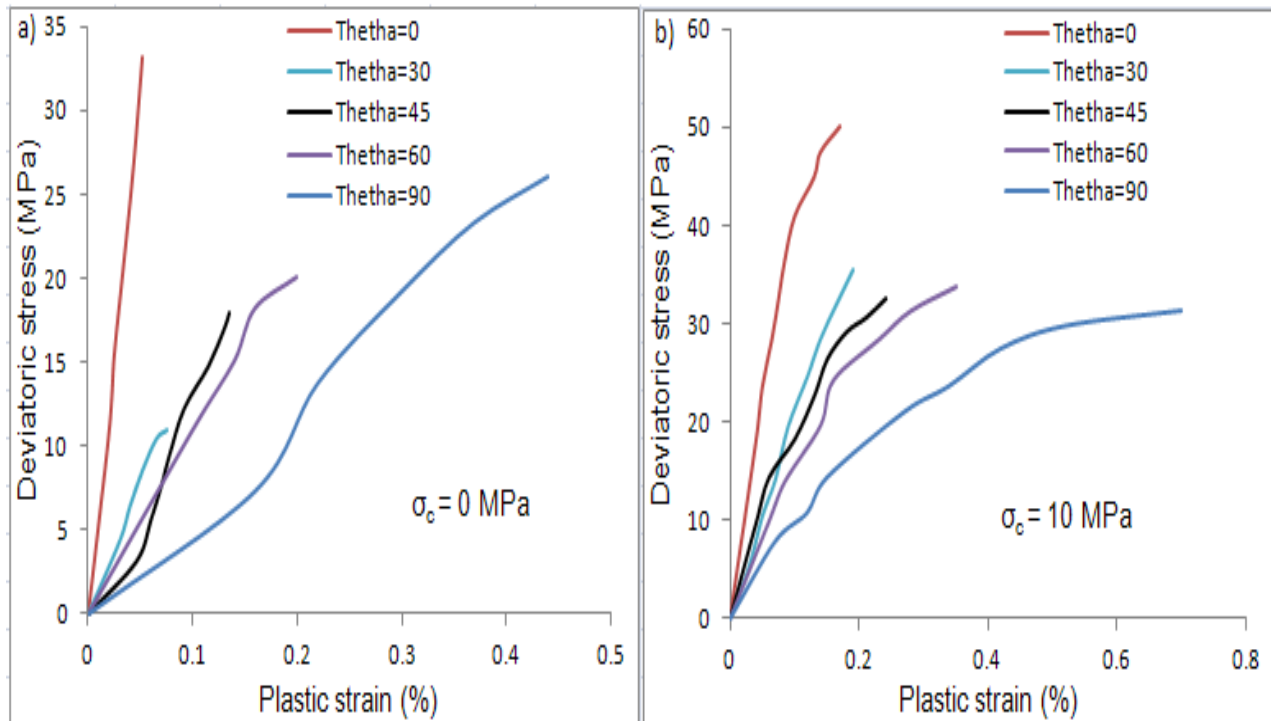


Figure 5.10. Development of plastic strains during loading rock specimens at different loading orientation angle, θ , and confining pressures: 0 MPa and 10 MPa.

5.5. Strength parameters measured at the peak strength and in the post-peak region

The measured strength parameters, cohesion and friction angle, at peak strength and in the post-peak region are listed in Table 5.1 for different loading orientations. The highest values of friction and cohesion are obtained in tests carried out at $\theta=0^\circ$. The results indicate that the strength parameters are sensitive to the inclination of bedding planes. Figure 5.11 describes the strength parameters measured at the peak strength. Figure 5.12 describes the strength parameters measured in the post-peak region.

Table 5.1: Friction angle and cohesion at the peak strength and in the post peak region for different loading orientations, θ .

Loading orientation θ	Friction angle at peak strength φ (degrees)	Cohesion at peak strength c (MPa)	Friction angle in post peak region φ_{pp} (degrees)	Cohesion in post peak region c_{pp} (MPa)
0°	32.68	8.65	30.5	2.32
30°	31.26	4.4	27.7	1.01
45°	26.6	5.3	25.8	1.11
60°	27.2	6.43	26.1	1.15
90°	31.46	6.8	29	1.48

Note: Subscript “pp” means post-peak.

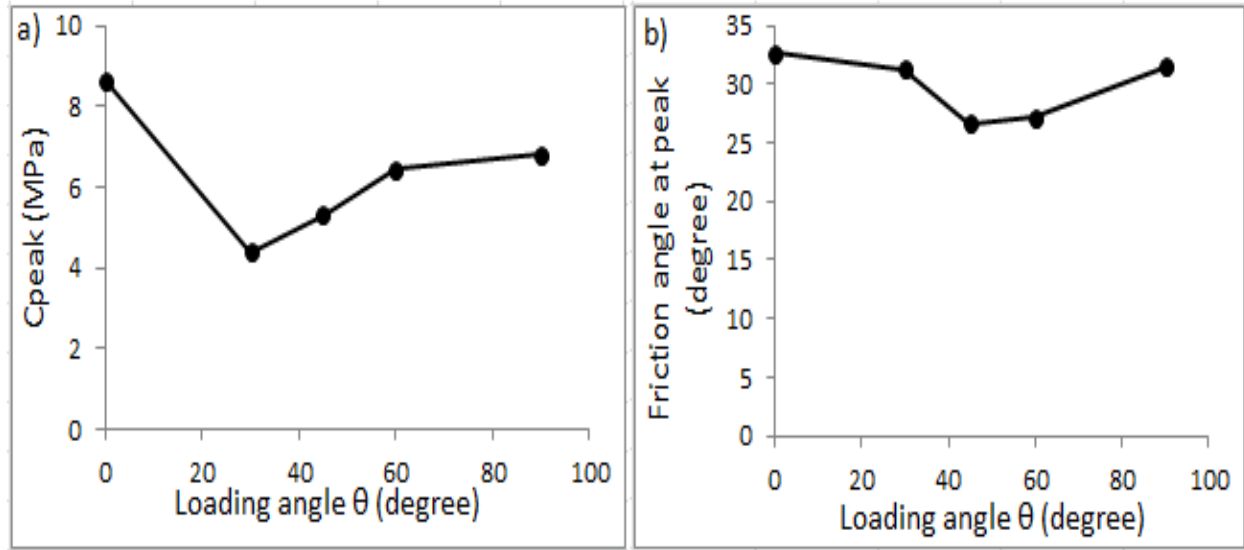


Figure 5.11. Strength parameters at peak strength: a) Cohesion and b) Friction angle.

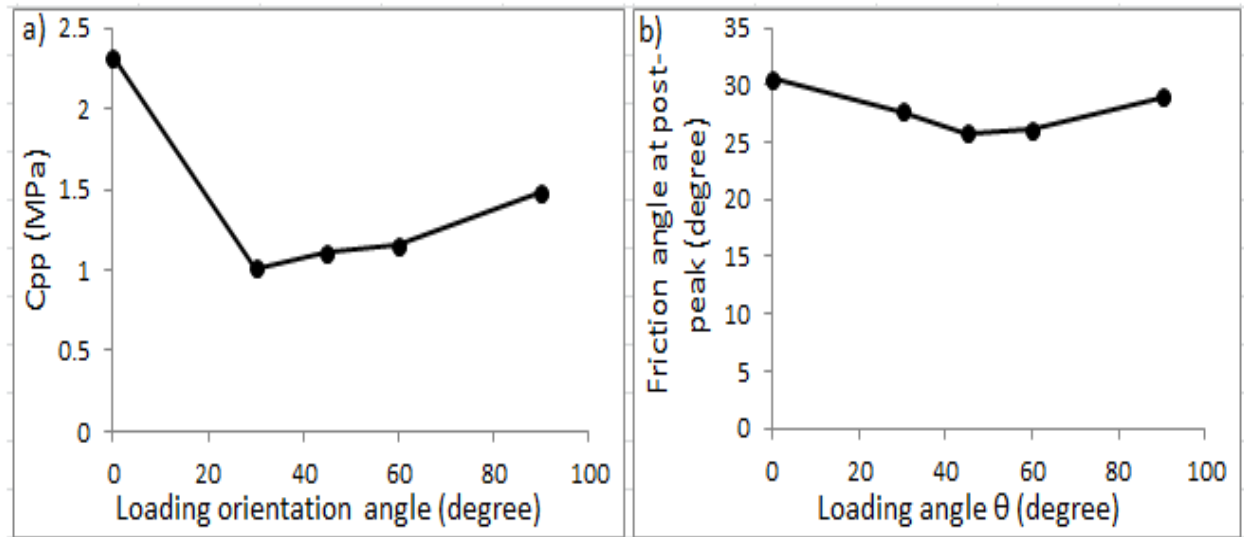


Figure 5.12. Strength parameters measured in the post-peak region: a) Cohesion and b) Friction angle.

5.6. Strength anisotropy

The evolution of the degree of anisotropy is investigated by defining two parameters as follows:

$$k_1 = \frac{(\sigma_1 - \sigma_3)_{\theta=0}}{(\sigma_1 - \sigma_3)_{\theta=90}} \quad \text{and} \quad k_2 = \frac{(\sigma_1 - \sigma_3)_{max}}{(\sigma_1 - \sigma_3)_{min}}$$

In these equations, the parameter k_1 defines the ratio between the principal stress difference at the peak strength when $\theta=0^\circ$ and the principal stress difference at the peak strength when $\theta=90^\circ$. The parameter k_1 must be determined from the tests conducted at the same confining pressure. The parameter k_2 defines the ratio between the maximum and minimum peak strengths for the same confining pressure. Figure 5.13 describes the experimental results obtained for both parameters. The value of the parameter k_1 is nearly equal to unity and increases slightly with the increase in confining pressure. This indicates that the strength anisotropy between the two principal axes, $\theta=0^\circ$ and 90° , is small. The value of the parameter k_2 is nearly equal to two and it decreases with the increase in confining pressure. This indicates that the strength anisotropy becomes smaller for high confining pressures. Similar observations have been made by Niandou et al. (1997).

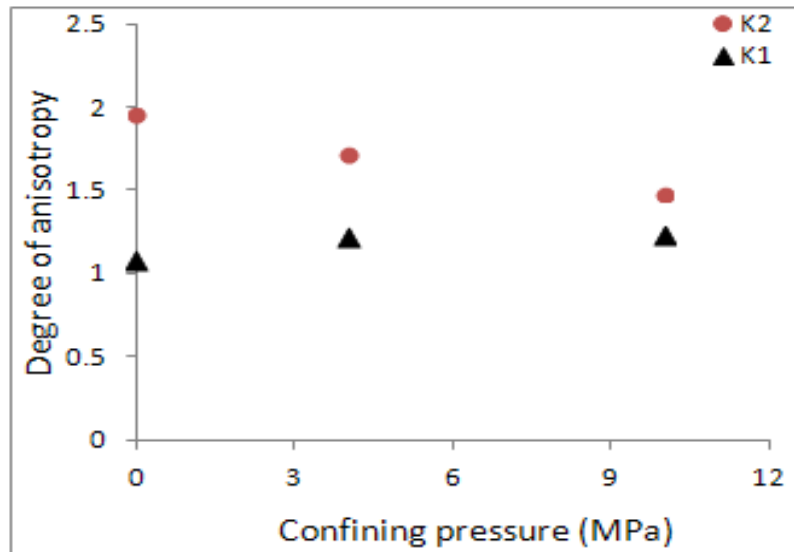


Figure 5.13: Variation of the degree of anisotropy with confining pressure.

5.7. Experimental failure surface in p-q plane

The data of failure stress from various tests are presented in p-q plane as shown in Fig. 5.14. The failure surface is a function of the loading orientation angle, θ . For the range of the confining pressure considered in testing (0, 4, and 10 MPa), it is clear that the experimental failure surfaces are of a linear form.

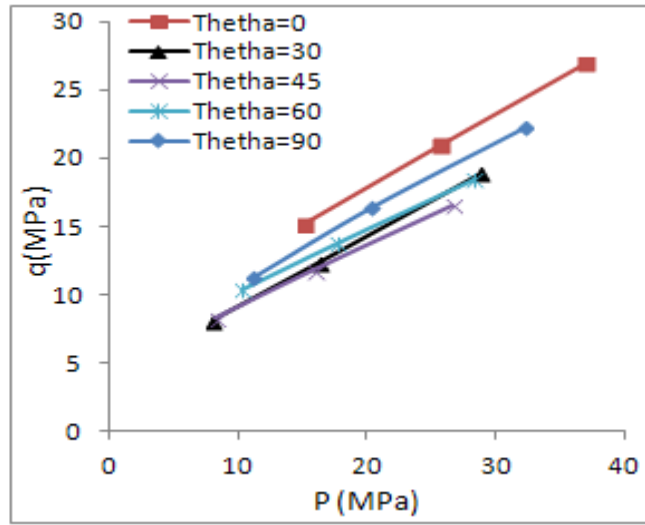


Figure 5.14. Experimental failure surface in p-q plane for different values of θ .

5.8. Main failure modes of the argillite

Fractures observed on failed samples indicate that the failure occurs in three principal modes: extension (splitting), shearing, and a combination of the extension and shearing. The mode of failure is a function of the loading orientation angle, θ , as shown in Fig. 5.15. The range of confining pressure considered in this investigation has little effect on the failure mode.

1. Testing at $\theta=0^\circ$

The failure mode is an extension type. In this particular test, two fractures (almost vertical) were developed and the sample was split into three blocks.

2. Testing at $\theta=30^\circ$, 45° , and 60°

The failure mode is shearing. It manifested itself by the opening and sliding of bedding planes. The orientation of the developed fracture is nearly equal to θ .

3. Testing at $\theta=90^\circ$

The failure mode is an extension type at the top of the rock specimen. The fracture cuts through the argillite sample perpendicular to the bedding planes. In the lower half of the specimen, the failure mode becomes shearing. The sample broke into three blocks. It is possible that the development of shear fractures is caused by the constraints imposed on the lower end of the specimen by the loading platen at the base.

The experimental observations agree with the modeling results reported in Brady and Brown (2006).

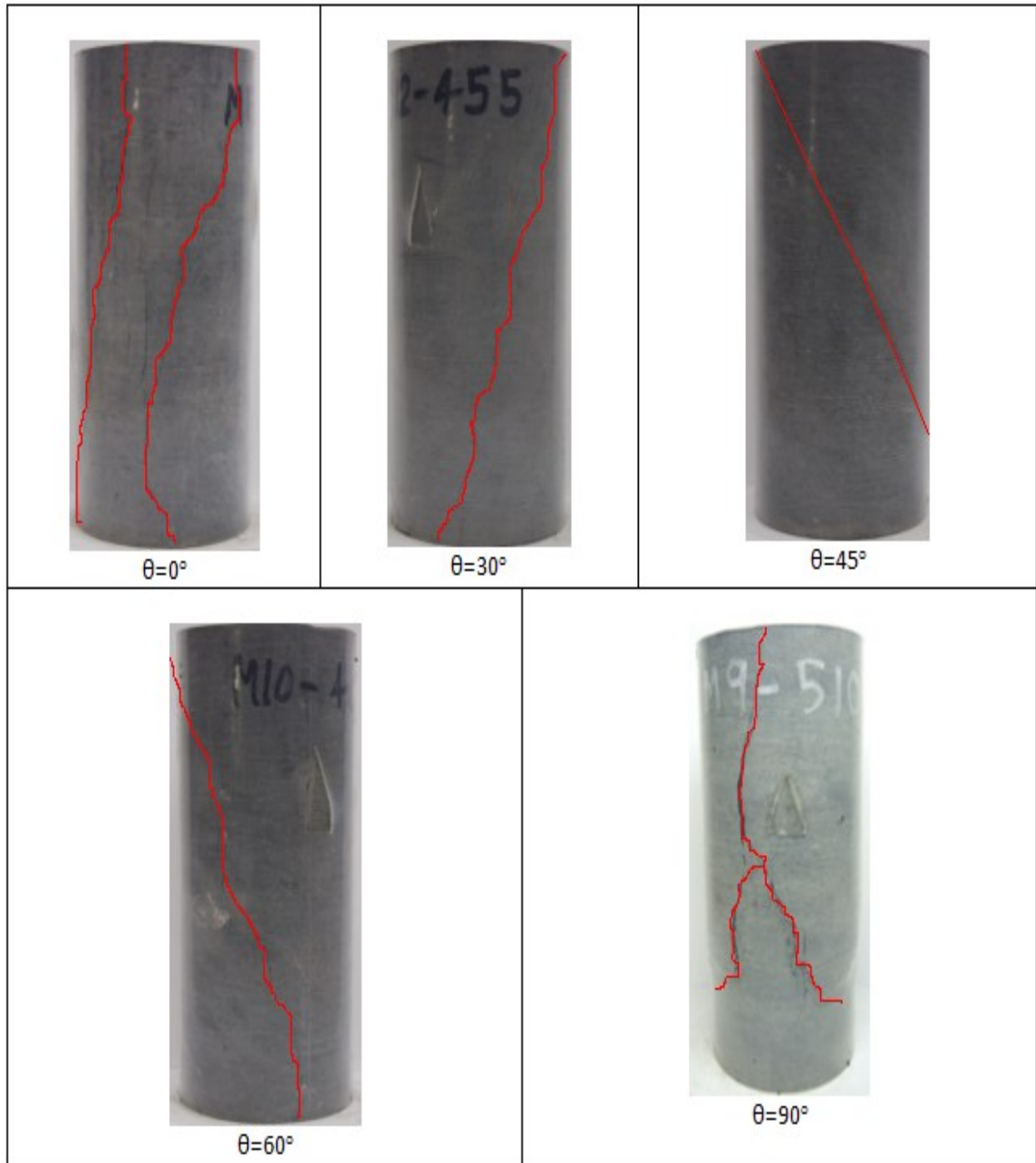


Figure 5.15: Failure modes for different values of θ .

5.9. Creep behaviour of sedimentary rocks

Sedimentary rocks exhibit creep behaviour. Construction of DGRs in sedimentary rocks results in the development of an EDZ around shafts, tunnels, and emplacement rooms. The initial size of the EDZ depends mainly on the state of stress, geometry and orientation of the underground opening, mineralogical composition of the rock, excavation method, mechanical and hydraulic properties of the rock, and the geological structure of the rock mass. The EDZ evolve with time due to strength degradation, porewater pressure dissipation, and development of wet-dry cycles (Millard et al., 2009).

The results of the creep tests described in Chapter 4 indicate that the time-dependency of the mechanical behaviour of the Tournemire argillite is quite important. If the magnitudes of the newly developed stresses around the underground openings are sufficiently high relative to the rock strength, greater than 85% of the peak strength, the creep can cause an increase in the size of the EDZ and even the failure of the rock. Other parameters may speed up the rate of the creep such as repeated wetting/drying cycles as these cycles can cause softening of the rock.

5.10. Comparison with previous experimental data

Many parameters such as loading rate, sample size, and water content can affect the results of the experiments. The scale effects on the mechanical properties of rocks have been investigated by many researchers (Thuro et al., 2001; Simon and Deng, 2009; Heuze, 1980) and it was found that, as an example, the uniaxial compressive strength and the deformation modulus tend to decrease as size of the sample increases. The water content also affects the mechanical properties of the rocks (Chapter 4). Therefore, comparison of the present experimental data with previously published experimental data is not a straightforward process. However, the same kind of trends in present test results is expected.

The mechanical behaviour of the Tournemire argillite was investigated by Niandou et al. (1997). The size of the specimen used by those authors was about 37 mm in diameter and 75 mm in height, compared to, respectively, 61.3 mm and 133 mm, used in the present investigation. The

large size of the sample used in this investigation would capture the effect of anisotropy. In the present study, all tests are displacement-controlled type except for the Brazilian tests. Here, comparisons of some key results are given.

The mechanical properties of the Tournemire argillite are anisotropic in both strength and deformation. The peak strength, post-peak strength, axial strain, plastic strain, and volumetric strain are influenced by the loading orientation angle, θ , and confining pressure. The elastic parameters are also influenced by θ . Similar observations were made by other authors.

In terms of the magnitude, the compressive strength values obtained in this research are around two-thirds ($2/3$) of the compressive strength values reported by other authors. Also, the axial and volumetric strains measured in the present investigation are lower than those obtained by Niandou et al. (1997). In the cyclic tests of the present study, after the removal of the deviatoric stress the plastic strain represents between two-thirds ($2/3$) and one-half ($1/2$) of the total strain and the other authors reported a value of one-half ($1/2$). However, the values of the elastic modulus in the two main directions and shear modulus ($E_1=12.5$ GPa, $E_2=21$ GPa, and $G_{12}=4.57$ GPa) are very close to the corresponding values reported by other authors ($E_1=11$ GPa, $E_2=20$ GPa, and $G_{12}=4.9$ GPa). With respect to Poisson's ratio, the results of the present study are slightly less than those reported by Niandou et al. (1997).

The mechanical properties of the Tournemire argillite can also be compared to the mechanical properties of many clay-stones. As an example, the Opalinus clay has been investigated as potential host rock for radioactive waste deposits; see for example, Bossart et al. (2004). The Opalinus clay is also anisotropic due to the presence of bedding planes and its mechanical properties also depend on the loading orientation angle, θ . As an example, the uniaxial compressive strength ranges from 12 to 18 MPa, the tensile strength ranges from 0.4 to 2.2 MPa, and the elastic modulus ranges from 3 to 8 GPa. The stress-strain relationship for the Opalinus clay is also nonlinear.

5.11. General observations

The development of a constitutive relationship for the Tournemire argillite could be based on the classical framework of strain hardening-strain softening plasticity.

CHAPTER 6

MODELING OF THE MECHANICAL BEHAVIOUR OF THE TOURNEMIRE ARGILLITE

6.1. Background

Two kinds of inelastic behaviour might be observed during excavation in a rock mass: the plastic deformation inside the rock matrix due to the dislocation movement of the rock particles and rock damage induced by the initiation and growth of microcracks. The microcracking process affects the hydraulic and mechanical properties of the rock (Martin, 1993; Rutqvist et al., 2009). It induces softening and in some cases can be the main cause of instability problems. Continuum damage mechanics concepts have been applied to different materials to account for the effect of microcracking (Kachanov, 1958; Lemaitre, 1992; Salari et al., 2004; Dragon and Mroz, 1979).

The stress level at which the new microcracks start to develop within the rock depends on several factors, such as its clay content; as an example, the Tournemire argillite has a high clay content (55%) which makes the argillite to behave less brittle than a hard rock and the new microcracks start to develop at a stress level of about 70% of the peak strength (Chapter 4). For hard rock such as granite, the new microcracks start to develop within the rock at a stress level of about 40% of the peak strength (Martin, 1993).

Laboratory investigation and prediction of the mechanical behavior of the rock within the EDZ is essential for the development of DGRs. A large testing program was carried out to characterize the mechanical behaviour of the Tournemire argillite within the EDZ (Chapter 4). The results of the laboratory experiments indicate that the mechanical properties of the argillite are anisotropic, the plastic strain starts to develop right at the beginning of the stress-strain relationship, the plastic strain affects the elastic properties of the argillite, and its mechanical behaviour is highly nonlinear.

The objective of this Chapter is to develop and validate an elastoplastic-damage model for describing the mechanical behaviour of the Tournemire argillite. More specifically, the objectives are: (1) to develop a damage model and determine its damage parameters, (2) to validate the proposed concept of mobilized strength parameters (Chapter 4), (3) to combine the damage model and the concept of mobilized strength parameters with the theory of elastoplasticity, (4) to numerically simulate the first part of the results of laboratory experiments,

(5) to interpret the results of the numerical simulations, (6) to combine the general equations developed in the step (5) with the classical theory of elastoplasticity, and (7) to evaluate the capabilities of the model by simulating the second part of the results of laboratory experiments.

6.2. Assumptions made in this Chapter

The following two assumptions have been considered in this Chapter:

- **Assumption 6.1:** The initiation and propagation of microcracks throughout the sedimentary rocks is assumed to be uniformly distributed everywhere and to be independent of loading orientation.
- **Assumption 6.2:** The Tournemire argillite is transversely isotropic material. Since a widely accepted anisotropic failure criterion that is easy to calibrate and apply for sedimentary rocks has yet to be established (Gao et al., 2010), the isotropic failure criterion is used. The effect of inherent anisotropy is minimized by selecting appropriate strength parameters (loading-orientation dependent) of the rock.

6.3. A general concept for formulating the elastoplastic-damage model

The concept consists of four components:

- *Elastic properties of the argillite:* They are estimated from the results of cyclic tests (Chapter 4).
- *Damage model:* It is used to estimate the amount of damage caused to the elastic properties of the argillite at any stress level and for any loading orientation angle, θ .
- *Concept of mobilized strength parameters (proposed in Chapter 4):* It is used to estimate mobilized strength parameters at any stress level and for any loading orientation angle, θ . These mobilized strength parameters are supplied to the yield/failure criterion.
- *Classical theory of elastoplasticity:* For calculating stresses and strains.

The interaction between the four components of the model is shown in Fig. 6.1.

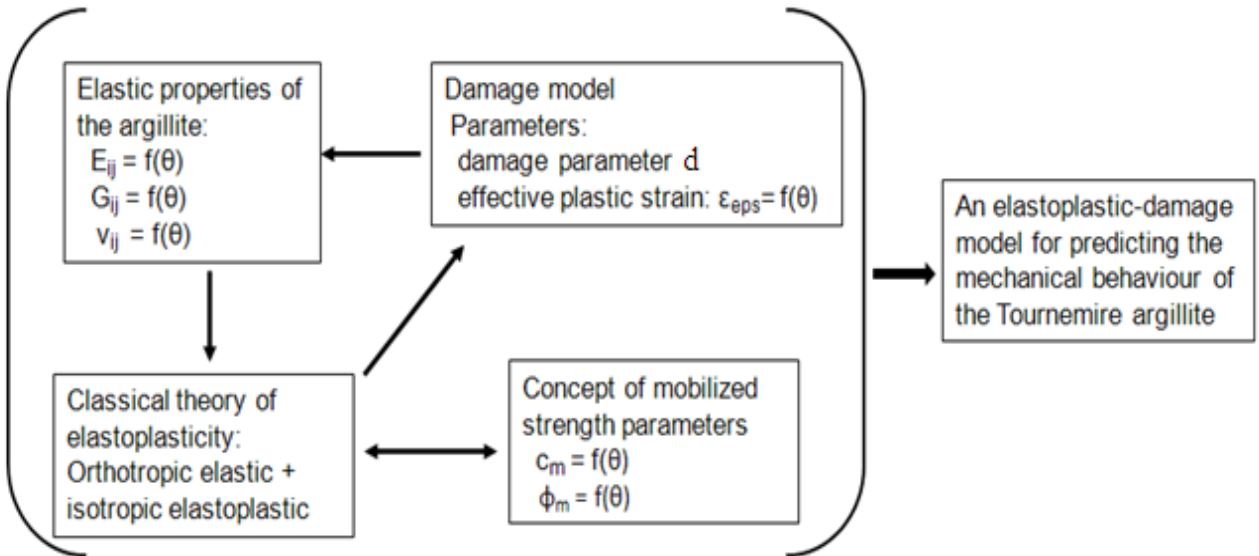


Figure 6.1. The interaction between the four components of the model.

6.4. Mathematical formulation of the elastoplastic damage model

The elastoplastic damage model is the result of combining a damage model, the concept of mobilized strength parameters, and the theory of elastoplasticity. Model development requires numerical simulations of laboratory experiments. Therefore, the experimental results of the laboratory testing program (Chapter 4) are divided into two parts. The first part of the results is used to develop the model. Best-fit with experimental data helps in understanding the mechanical behaviour of the argillite and developing the required relations. The second part of the experimental results is used to validate the model.

Orthotropic-elastic isotropic-plastic combination (see Assumption 6.2) is used to simulate the three-dimensional mechanical behaviour of the argillite. The peak strength parameters used in the numerical simulations of the experiments are the ones measured in the experiments in the direction of loading. The Mohr-Coulomb yield/failure criterion is used in the elastoplastic part. The Finite Element Code COMSOL Multiphysics v 4.3 is used for simulating the experiments. The argillite may contain pre-existing microcracks, small fractures, and may crack under mechanical loading. Such cracks and fractures cannot be simulated using continuum mechanics-based Finite Element Code such as COMSOL. However, it is expected the results of the numerical simulations will agree with the results of the experiments.

6.4.1. Damage evolution law

Different approaches have been developed to measure directly the damage or to formulate models for estimating the amount of damage. In 1958, Kachanov introduced a damage parameter d (mechanical) as illustrated in Fig. 6.2. In his model, the damage is assumed to be isotropic and the damage parameter can be treated as a scalar quantity and completely characterizes the three-dimensional damage state. According to Kachanov, the stress-strain relationship can be expressed as follows and the damage parameter d can be expressed in terms of stress or strain.

$$\varepsilon = \frac{\sigma}{E(1-d)} \quad [6.1]$$

In this research project, the Kachanov model is adopted to estimate the amount of damage that can be caused to the elastic properties of the argillite due to the inelastic deformations. Poisson's ratio is defined as the negative ratio of transverse strain to axial strain. The damage is assumed to be isotropic (see Assumption 6.1) and its effect on Poisson's ratio is minimal and can be neglected (Chiantoni et al., 2012). However, the effect of the damage on the elastic modulus and shear moduli of the argillite can be expressed as a function of the developed inelastic deformations during loading.

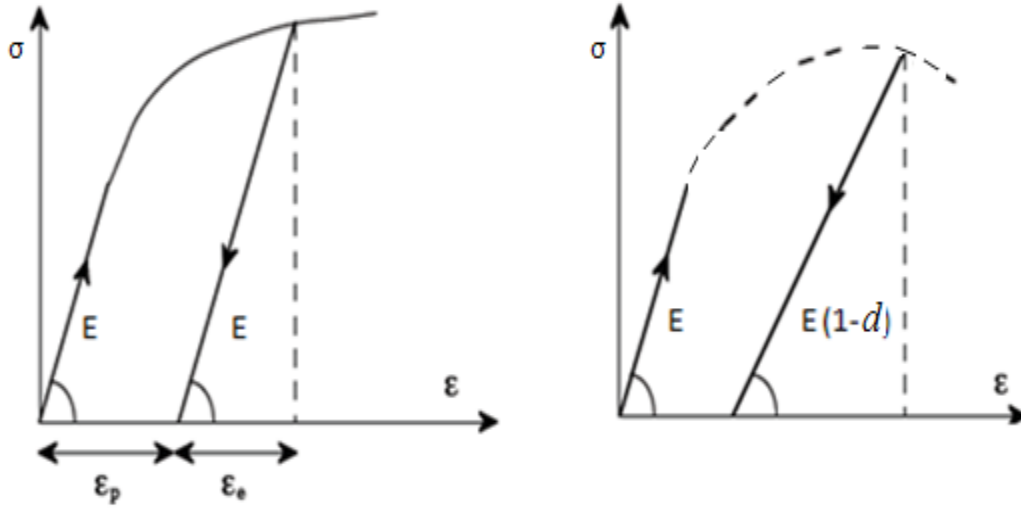


Figure 6.2. Degradation in the elastic modulus of the material (d is the scalar damage variable).

Strains such as plastic strains and equivalent deviatoric strains have been used as a measure of damage; see for example, Shirazi and Selvadurai (2005) and Nguyen (2007). Here, the effective plastic strain (ϵ_{eps}) is used as a measure of damage. The ϵ_{eps} is a monotonically increasing scalar value which is calculated incrementally as a function of the plastic components of the rate of deformation tensor.

$$\epsilon_{eps} = \sqrt{\frac{2}{3} \epsilon p_{ij} \epsilon p_{ij}} \quad [6.2]$$

Based on the Kacahnov concept, an equation has been proposed for estimating the amount of damage that can be caused to the elastic modulus of the argillite. For every axis-direction (x, y, or z), the elastic modulus is expressed according to the following equation.

$$E_i = E_{i0}(1 - d) = E_{i0}(1 - \chi \varepsilon_{eps}) \quad [6.3]$$

The ε_{eps} is a function of the loading orientation angle, θ , and stress level. χ is a constant and it can be determined by trial and error to give best agreement between calculated and measured results. E_{i0} is the initial elastic modulus along axis i that can be obtained from the first cycle in a cyclic test.

Even though there is no available experimental data that can be used to verify the effect of damage on the shear moduli of the argillite, similar equation has been proposed for estimating the amount of damage caused to the shear moduli of the argillite.

$$G_{ij} = G_{ij0}(1 - d) = G_{ij0}(1 - \chi \varepsilon_{eps}) \quad [6.4]$$

G_{ij0} is the initial shear moduli in direction j on the plane whose normal is in direction i .

6.4.2. Concept of mobilized strength parameters

Figure 6.3 describes the proposed concept of mobilized strength parameters, which is based on the results of the acoustic emission records. The concept indicates that the mobilization of strength parameters under increasing stress can be approximated as follows. Upon loading the specimen, the mobilized cohesion starts to increase very rapidly with the increase in stress. The rate of increase in the mobilized cohesion tends to slow down when the stress level increases beyond crack initiation stress, about 70% of the peak strength. The mobilized cohesion reaches a maximum value at the peak strength. However, as the stress increases beyond its value corresponding to the peak strength, the cohesion reduces rapidly to a negligible value. On the other hand, the mobilized friction angle is small at very low stress levels. Initially, the rate of its increase is also slow. When the stress level increases beyond crack initiation stress, the increase in the mobilized friction angle becomes large and its value reaches a maximum value at the peak strength. In the post peak region, the frictional resistance becomes the only component of the

post-peak strength. The peak strength parameters, Point D in Fig. 6.3.a, are influenced by the loading orientation angle, θ . The mobilized strength parameters, between Point A and Point D in Fig. 6.3.a, are influenced by the loading orientation angle and the confining pressure.

The peak strength parameters are obtained from conventional laboratory tests. The laboratory tests are displacement controlled and as the displacement increase/decrease in increments both mobilized strength parameters, c_m and ϕ_m , also increase/decrease according to Fig. 6.3.a. In the numerical simulations, the values of mobilized strength parameters are selected to achieve best fit with the experimental data. As will be shown later, the numerical results are used to develop general equations for estimating mobilized strength parameters for any loading orientation angle, θ , and at any stress level.

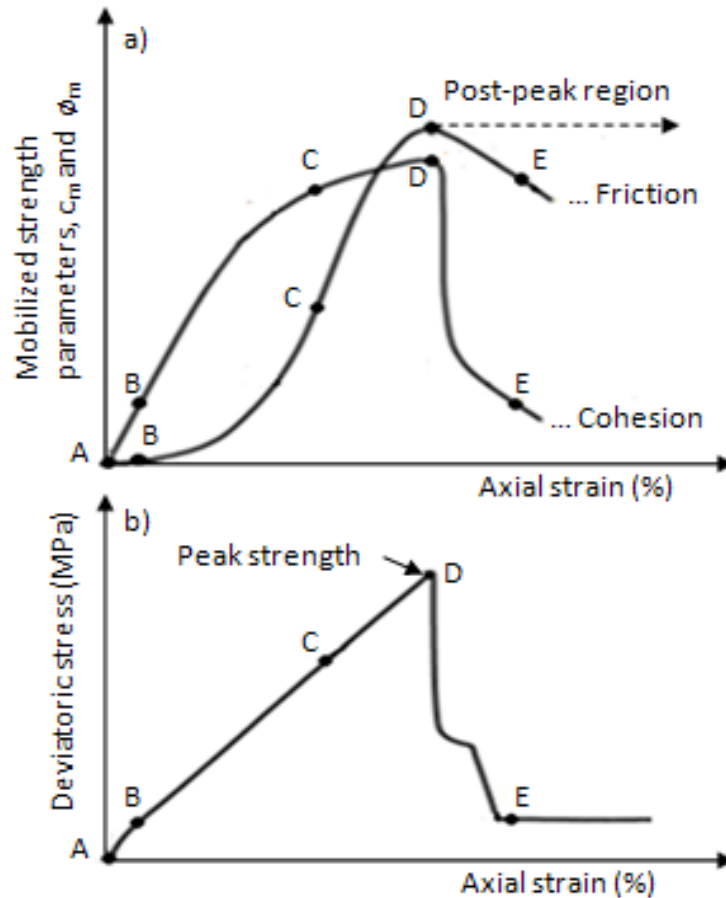


Figure 6.3. Mobilization of strength parameters during loading: a) c_m and ϕ_m as a function of axial strain, b) typical stress-strain curve for the Tournemire argillite.

The concept shown in Fig. 6.3 indicates that the mobilized cohesion of the material controls the location of the yield surface at low stress levels. At high stress levels the friction has more effect. Figure 6.4 describes the movement of the yield envelope as a function of mobilized strength parameters for a uniaxial test ($\theta=45^\circ$).

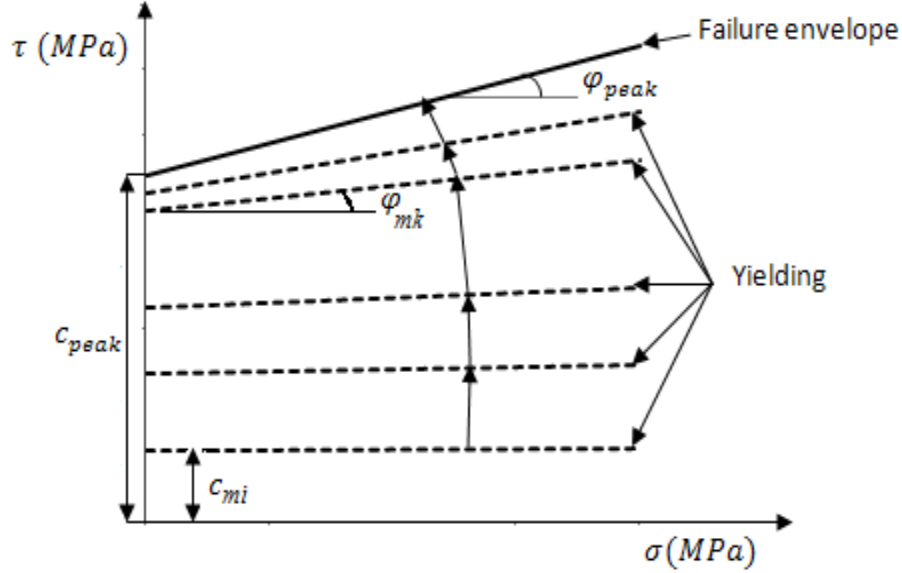


Figure 6.4. Translation of the yield surface as a function of the mobilized strength parameters for a uniaxial test ($\theta=45^\circ$). Note: c_{mi} indicates the mobilized cohesion at stress level i. Similarly, φ_{mk} indicates the mobilized friction angle at stress level k.

6.4.3. Elastoplastic model

The mechanical behaviour of the Tournemire argillite can be modeled using Hooke's law. This law can be expressed as follows.

$$d\varepsilon_{ij} = A_{ijkl} d\sigma_{kl} \quad [6.5]$$

The A_{ijkl} is the compliance tensor.

If we consider the matrix representation of tensors (ε_{ij} , σ_{kl} , and A_{ijkl}) in an arbitrary coordinate system (x, y, z), the Equation [6.5] can be rewritten as follows.

$$d\varepsilon_{ij} = D_{ijkl}^{ep} d\sigma_{kl} \quad [6.6]$$

The elastoplastic constitutive matrix $[D^{ep}]$ can be expressed as follows (Zienkiewicz and Taylor, 2005; Brady and Brown, 2006).

$$D_{ijkl}^{ep} = D_{ijkl}^e - \frac{D_{ijpq}^e \frac{\partial f}{\partial \sigma_{pq}} (\frac{\partial f}{\partial \sigma_{pq}})^T D_{ijpq}^e}{A + (\frac{\partial f}{\partial \sigma_{pq}})^T D_{ijpq}^e \frac{\partial f}{\partial \sigma_{pq}}} \quad [6.7]$$

$$A = - \frac{\partial f}{\partial H} (\sigma_{ij})^T \frac{\partial f}{\partial \sigma_{pq}} \quad [6.8]$$

where H is the hardening parameter, D^e is the elastic constitutive matrix, and f is the yield/failure criterion.

The Mohr-Coulomb yield criterion is used in this analysis. The mobilized strength parameters are used instead of peak strength parameters. The general form of the criterion is as follows.

$$\frac{\bar{\sigma}}{g(\beta)} - \alpha \sigma_m - \eta = 0 \quad [6.9]$$

$$\text{where } g(\beta) = \frac{3 - \sin \varphi_m}{2\sqrt{3} \cos \beta - 2 \sin \beta \sin \varphi_m}, \quad \alpha = \frac{2\sqrt{3} \sin \varphi_m}{3 - \sin \varphi_m}, \quad \eta = \frac{2\sqrt{3} c_m \cos \varphi_m}{3 - \sin \varphi_m}, \quad \bar{\sigma} = \sqrt{J_2};$$

$$\sigma_m = -\frac{1}{3} I_1, \text{ and } \beta = \frac{1}{3} \arcsin\left(\frac{3\sqrt{3} J_3}{2\sqrt{J_2^3}}\right).$$

β is the Lode angle, c_m and φ_m are the mobilized strength parameters, I_1 is the first invariant of the stress tensor σ_{ij} , J_2 is the second invariant of the stress deviator s_{ij} , and J_3 is the third invariant of the stress deviator s_{ij} .

Differentiating f with respect to σ_{ij} result in:

$$\frac{\partial f}{\partial \sigma_{ij}} = \frac{\partial f}{\partial \sigma_m} \frac{\partial \sigma_m}{\partial \sigma_{ij}} + \frac{\partial f}{\partial \bar{\sigma}} \frac{\partial \bar{\sigma}}{\partial \sigma_{ij}} + \frac{\partial f}{\partial \beta} \frac{\partial \beta}{\partial \sigma_{ij}} \quad [6.10]$$

The mobilized strength parameters, c_m and ϕ_m , are calculated for each increment of strain according to the concept described in Fig. 6.3.a and supplied to the yield/failure criterion. Thus, the yield surface translates in $\tau - \sigma$ space as a function of the mobilized strength parameters.

6.4.4. Coupling damage with elastoplastic model

The experimental results indicate that the elastic properties of the argillite are affected by the damage. Incorporating the Equations [6.3] and [6.4] into the stress-strain relationship, the elastic constitutive matrix D^e becomes as follows.

$$D^e = \begin{bmatrix} \frac{1}{E_1(1-\chi \varepsilon_{eps})} & -\frac{\nu_{21}}{E_2(1-\chi \varepsilon_{eps})} & -\frac{\nu_{31}}{E_3(1-\chi \varepsilon_{eps})} & 0 & 0 & 0 \\ -\frac{\nu_{12}}{E_1(1-\chi \varepsilon_{eps})} & \frac{1}{E_2(1-\chi \varepsilon_{eps})} & -\frac{\nu_{32}}{E_3(1-\chi \varepsilon_{eps})} & 0 & 0 & 0 \\ -\frac{\nu_{13}}{E_1(1-\chi \varepsilon_{eps})} & -\frac{\nu_{23}}{E_2(1-\chi \varepsilon_{eps})} & \frac{1}{E_3(1-\chi \varepsilon_{eps})} & 0 & 0 & 0 \\ 0 & 0 & 0 & \frac{1}{G_{23}(1-\chi \varepsilon_{eps})} & 0 & 0 \\ 0 & 0 & 0 & 0 & \frac{1}{G_{31}(1-\chi \varepsilon_{eps})} & 0 \\ 0 & 0 & 0 & 0 & 0 & \frac{1}{G_{12}(1-\chi \varepsilon_{eps})} \end{bmatrix} \quad [6.11]$$

Thus, the three-dimensional damage state is represented. The effect of the damage is considered in the four zones of the stress-strain relationship from the beginning of loading to the failure of the specimen.

The Equations [6.2], [6.3], [6.4], [6.6], [6.7], [6.8], [6.9], and [6.11] are used together in the next section to simulate the results of the first part of the laboratory experiments. The values of the mobilized strength parameters are selected to achieve best fit with the experimental data.

6.5. Numerical simulations of the results of the laboratory experiments

In this section, the first part of the results of the laboratory experiments is simulated in order to develop the necessary equations. A total of five uniaxial tests, five triaxial tests with confining pressure of 10 MPa, and five unconfined cyclic tests are simulated. In each category of the tests, five different loading orientations are considered: $\theta=0^\circ$, 30° , 45° , 60° , and 90° .

6.5.1. Finite element model and boundary conditions

The specimen of the argillite is of diameter 61.3 mm and height of 133 mm. Only a quarter of the specimen is considered in the analysis as shown in Fig. 6.5. Boundary conditions are as follows: the boundary A is subjected to prescribed increments of displacement (compression), the boundary E is subjected to confining pressure, and boundaries B, C, and D are set as rollers.



Figure 6.5. Finite element model and boundary conditions.

6.5.2. Elastic properties of the argillite

The mechanical behaviour of the argillite is orthotropic. The elastic parameters required for the constitutive matrices, Equation [6.11], are estimated from the experimental results and shown in Table 6.1. The elastic parameters depend on the loading orientation angle, θ .

Table 6.1. Elastic parameters required for the constitutive matrices.

θ	E_1 (GPa)	E_2 (GPa)	E_3 (GPa)	ν_{12}	ν_{23}	ν_{13}	G_{12} (GPa)	G_{31} (GPa)	G_{23} (GPa)
0°	21	12.5	21	0.15	0.115	0.15	9.1	9.1	5.4
30°	10.5	10	21	0.06	0.13	0.06	4.95	4.56	4.42
45°	11.9	11.9	21	0.11	0.11	0.11	5.36	5.17	5.36
60°	10	10.5	21	0.13	0.06	0.13	4.42	4.34	4.95
90°	12.5	21	21	0.115	0.15	0.115	4.5	4.5	9.1

6.5.3. Calculation procedure

The incremental form is adopted to model the nonlinear behaviour of the argillite. The initial values of mobilized strength parameters are around 5% of the peak strength parameters. The elastic parameters are described in Table 6.1. For every loading orientation angle, θ , the following steps are performed:

- An increment of displacement is applied to the specimen.
- The resulting strains and stresses are calculated using the classical theory of elastoplasticity.
- The calculated effective plastic strains are used as a measure of damage.
- The elastic properties of the argillite are updated according to the damage model (Equations [6.3] and [6.4]).

- The concept of mobilized strength parameters is used to estimate the amount of mobilized cohesion and friction angle generated by the increment of displacement. These values of mobilized strength parameters are supplied to the yield/failure criterion in the next increment of displacement.
- The above steps are repeated for each increment of displacement.

Upon loading the specimen of the argillite, the closure of existing microcracks cannot be simulated using continuum mechanics-based toolsets such as COMSOL Multiphysics. A correction factor is introduced to include the volumetric strain that can result from such process into the calculations. Based on the experimental results (Chapter 4), the correction factor is defined as follows.

$$c_f = 0.0009 \theta + 0.02 \quad [6.12]$$

Units: θ [degrees]; $c_f=0.02$ for $\theta=0^\circ$; $c_f=0.06$ for $\theta=45^\circ$.

The volume change is calculated as follows.

$$\varepsilon_v = (1 + c_f) \varepsilon_{vc} \quad [6.13]$$

where: ε_{vc} is the volume change obtained from numerical simulations.

6.5.4. Numerical simulations of uniaxial and triaxial tests

The deviatoric stress-strain curves obtained from numerical simulations are shown together with the corresponding curves obtained from the experiments. Figure 6.6 shows the deviatoric stress-axial strain curves for uniaxial and triaxial tests.

Figure 6.7 shows the deviatoric stress-volumetric strain curves. In the experiments, the reversal in direction from contraction to dilation occurs at a stress level σ_r equal to the peak deviatoric stress. In the numerical simulations, the reversal in direction from contraction to dilation occurs at σ_r which is less than the peak deviatoric stress. The level of this stress depends on confining pressure and loading orientation angle, θ .

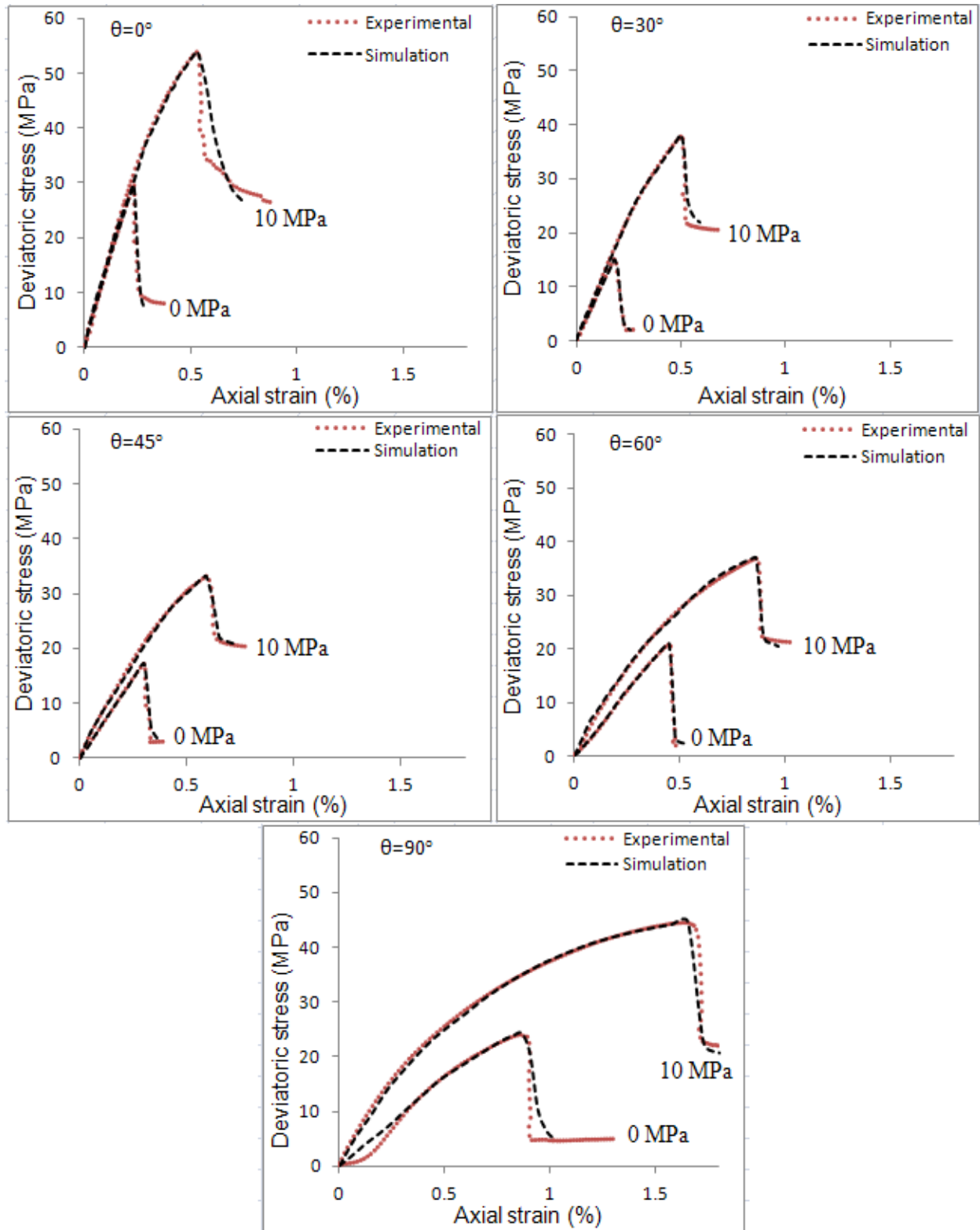


Figure 6.6. Deviatoric stress-axial strain curves for uniaxial and triaxial tests, obtained from experiments and numerical simulations.

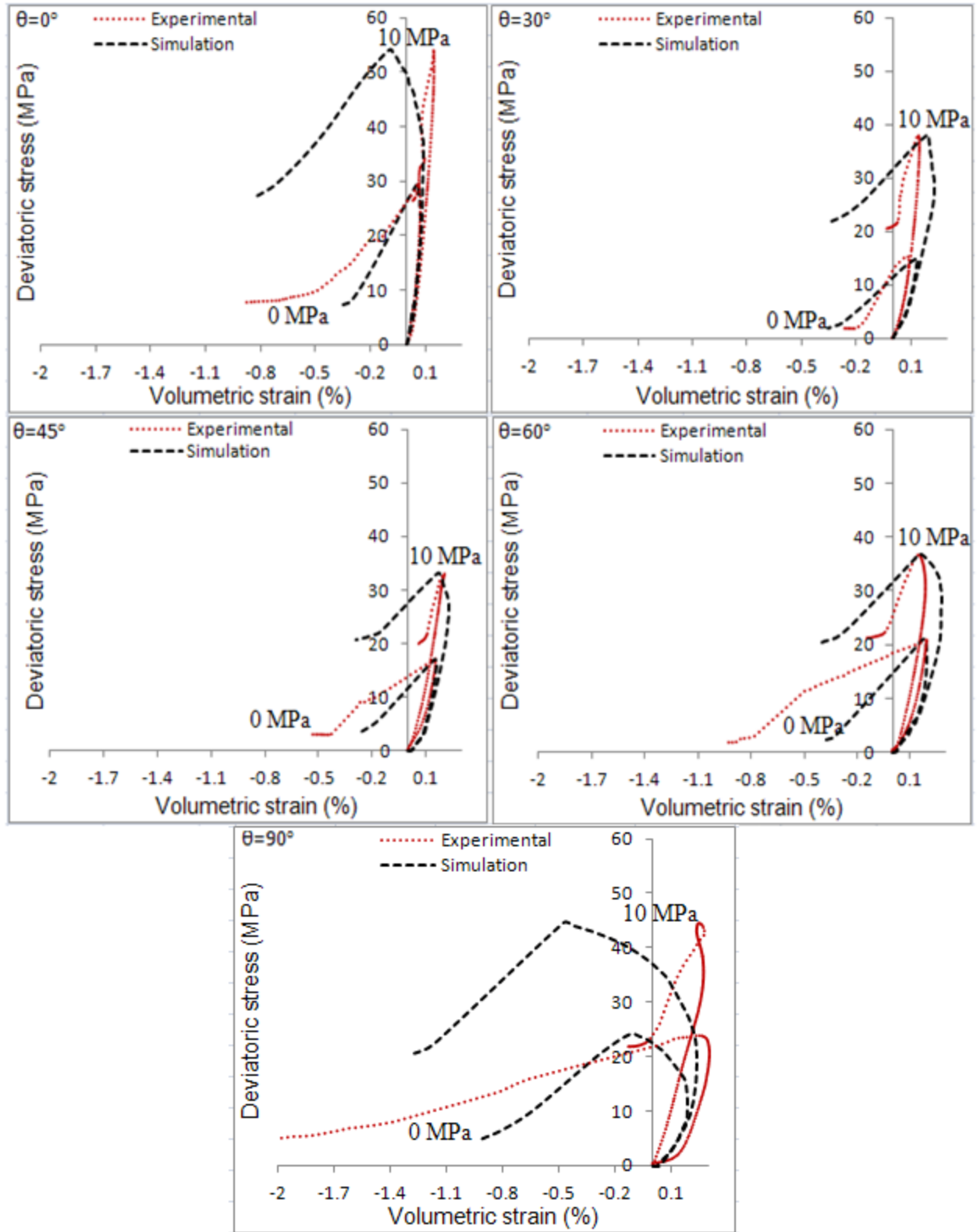


Figure 6.7. Deviatoric stress-volumetric strain curves for uniaxial and triaxial tests, obtained from experiments and numerical simulations.

6.5.5. Numerical simulations of unconfined cyclic tests

In each test, only a couple of cycles are numerically simulated. The deviatoric stress-strain curves obtained from numerical simulations are shown together with the corresponding curves obtained from the experiments. The deviatoric stress-axial strain curves are shown in Fig. 6.8. The numerical approach reproduces the stress-strain curves including unloading-reloading.

The deviatoric stress-volumetric strain curves are shown in Fig. 6.9. In the experiments, as explained above, the reversal in direction from contraction to dilation occurs at σ_r equal to the peak deviatoric stress. In the numerical simulations, the reversal in volumetric strain direction from contraction to extension occurs at a stress level σ_r less than the peak deviatoric stress. In both the experiments and numerical simulations, if cycling is below σ_r the specimen experiences only a small change in volumetric strain with each additional cycle (contraction). If cycling is above σ_r , the specimen experiences a significant change in volumetric strain, moving from contraction state to the extension state, with each additional cycle.

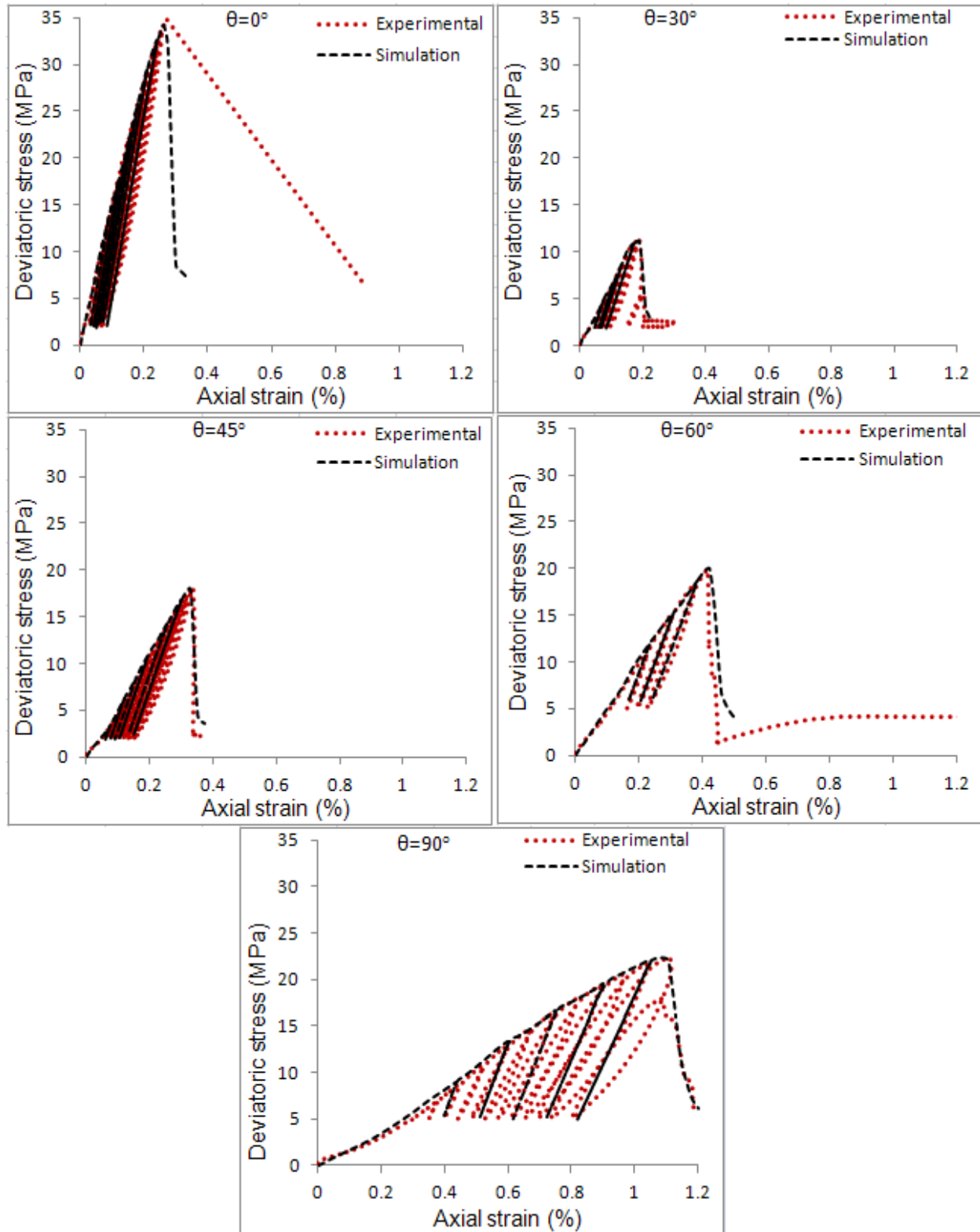


Figure 6.8. Deviatoric stress-axial strain curves for unconfined cyclic tests, obtained from experiments and numerical simulations.

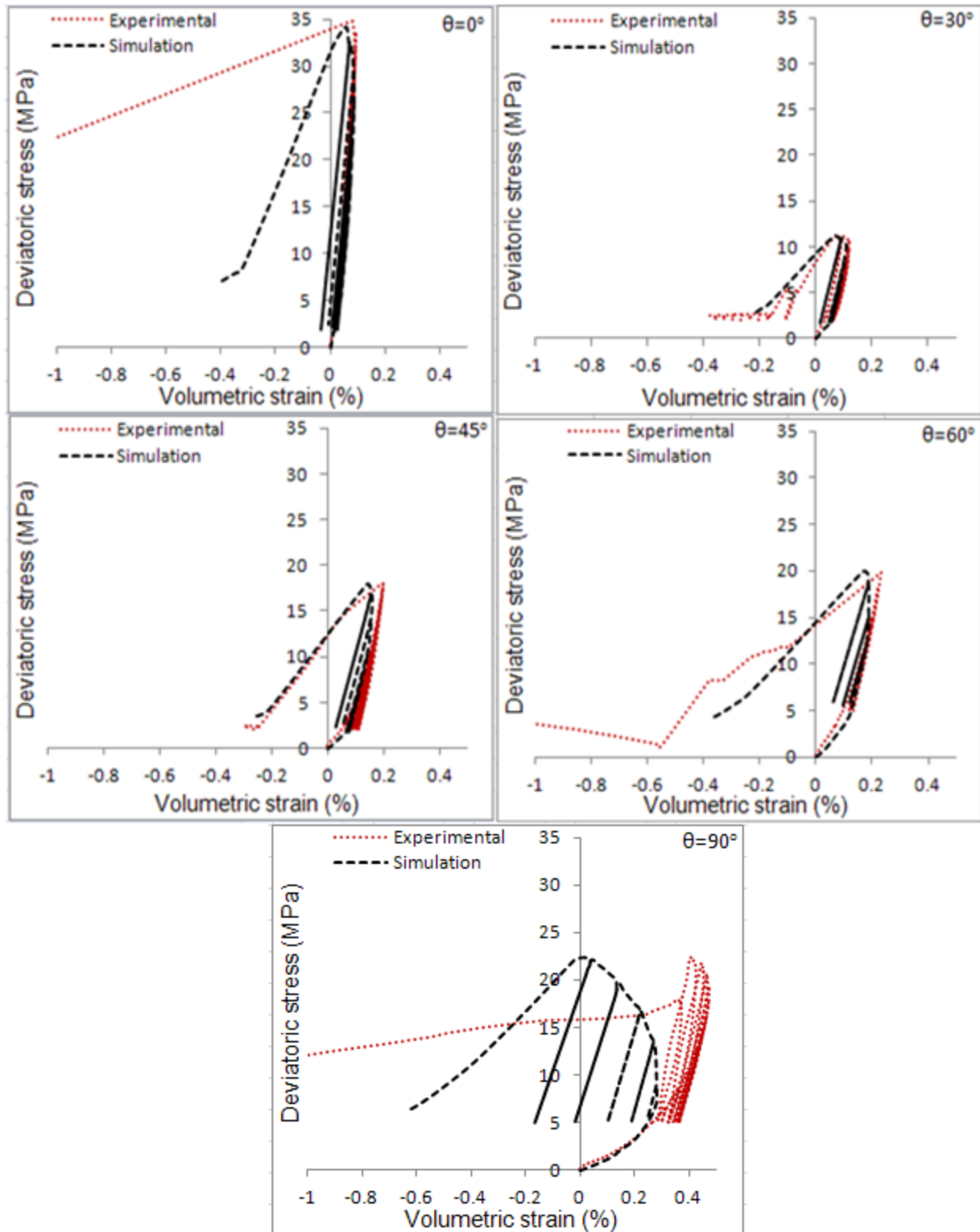


Figure 6.9. Deviatoric stress-volumetric strain curves for unconfined cyclic tests, obtained from experiments and numerical simulations.

6.6. Interpretation of the numerical results

In this section, the numerical results are interpreted and the objective is to develop general equations for estimating mobilized strength parameters and the amount of damage at any stress level and for any loading orientation angle, θ .

6.6.1. Development of a general equation for estimating mobilized cohesion c_m

A comparison of the numerical results and experimental data (Figs 6.6, 6.7, 6.8, and 6.9) confirmed the validity of the proposed concept of mobilized strength parameters (Fig. 6.3). Figure 6.10 describes the variation of the mobilized cohesion, c_m , with deviatoric stress for five different values of loading orientation angle, θ , and two different values of confining pressure (0 and 10 MPa). The c_m is a function of θ , stress level, and cohesion at the peak strength.

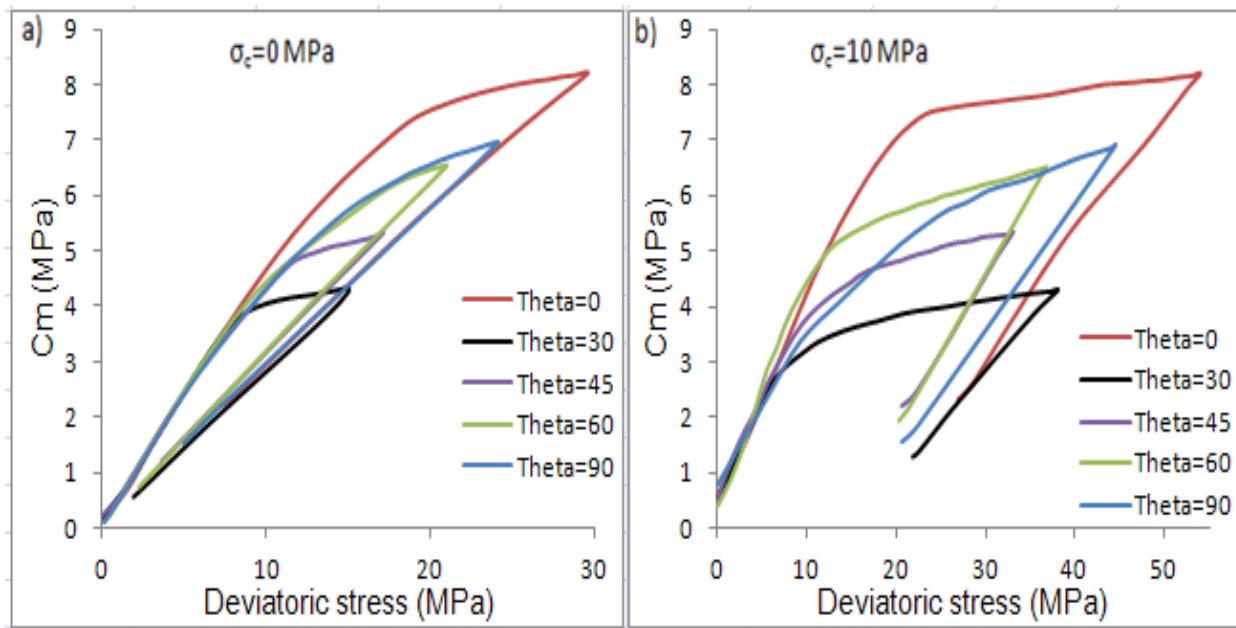


Figure 6.10. Mobilized cohesion (c_m) versus $(\sigma_1 - \sigma_3)$ for five different values of θ and two different values of confining pressure: 0 and 10 MPa.

Based on the numerical results, a general equation is proposed for estimating the parameter c_m at any stress level and for any loading orientation angle, θ . The c_m is expressed as a function of the stress level, cohesion at the peak strength (c_{peak}), and peak strength $((\sigma_1 - \sigma_3)_{peak})$.

$$c_m = c_{peak} \sqrt{\frac{\sigma_1 - \sigma_3}{(\sigma_1 - \sigma_3)_{peak}}} \quad [6.14]$$

c_{peak} and $(\sigma_1 - \sigma_3)_{peak}$ are a function of the loading orientation angle, θ .

c_{peak} :

The values of c_{peak} are obtained from the results of laboratory experiments for five different loading orientation angles and shown in Fig. 6.11.

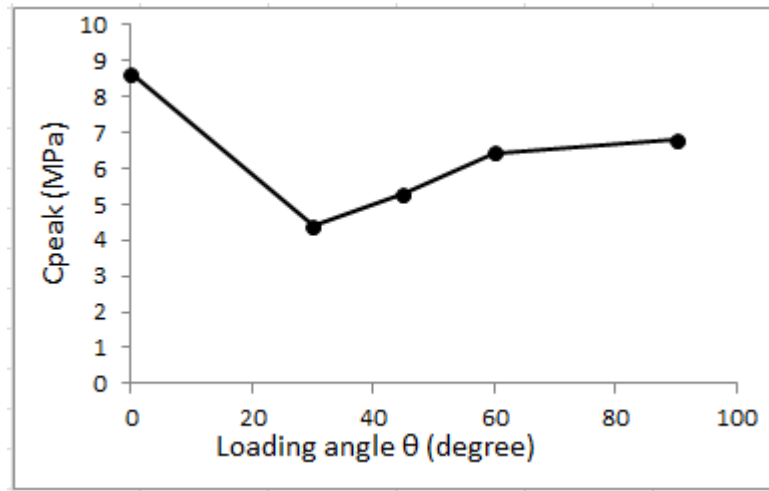


Figure 6.11. Cohesion at peak strength (c_{peak}) versus loading orientation angle (θ).

$(\sigma_1 - \sigma_3)_{peak}$:

The values of $(\sigma_1 - \sigma_3)_{peak}$ are obtained from the results of laboratory experiments for five different values of θ and three different confining pressures (σ_3) and shown in Fig. 6.12.

For any loading orientation angle, the value of $(\sigma_1 - \sigma_3)_{peak}$ can be estimated using the microstructure approach (Pietruszczak, 2010) as follows.

$$(\sigma_1 - \sigma_3)_{peak} = \overline{(\sigma_1 - \sigma_3)_{peak}} (1 + A_1(1 - 3 l_2^2) + a_1 A_1^2(1 - 3 l_2^2)^2 + a_2 A_1^3(1 - 3 l_2^2)^3 + a_3 A_1^4(1 - 3 l_2^2)^4) \quad [6.15]$$

$$l_2^2 = \frac{\sigma_3^2 \sin^2(90 - \theta) + \sigma_1^2 \cos^2(90 - \theta)}{2 \sigma_3^2 + \sigma_1^2} \quad [6.16]$$

The coefficients A_1 , a_1 , a_2 , and a_3 in the Equation [6.15] are the coefficients of best fit approximation (Pietruszczak – personal contact). Under uniaxial conditions, the values of the coefficients are as follows.

$$\overline{(\sigma_1 - \sigma_3)_{peak}} = 15.31 \text{ MPa}; A_1 = -0.122; a_1 = 36; a_2 = -243.5; a_3 = 587.5.$$

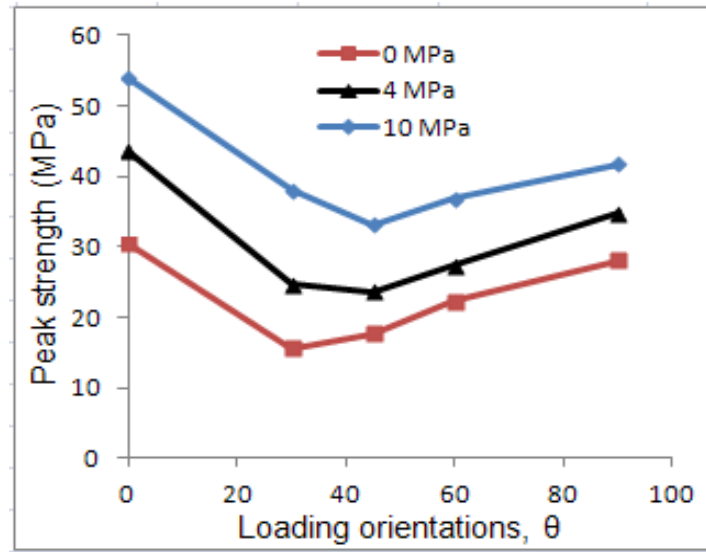


Figure 6.12: $(\sigma_1 - \sigma_3)_{peak}$ versus θ for different values of confining pressure: 0, 4, and 10 MPa.

The equation [6.14] is valid up to the peak strength. In the post-peak strength region, a linear relationship can be established between C_m and deviatoric stress $(\sigma_1 - \sigma_3)$.

6.6.2. Development of a general equation for estimating mobilized friction ϕ_m

Figure 6.13 describes the variation of the mobilized friction, ϕ_m , with deviatoric stress for five different values of loading orientation angle, θ , and two different values of confining pressures (0 and 10 MPa). The ϕ_m is a function of θ , stress level, and friction angle at the peak strength.

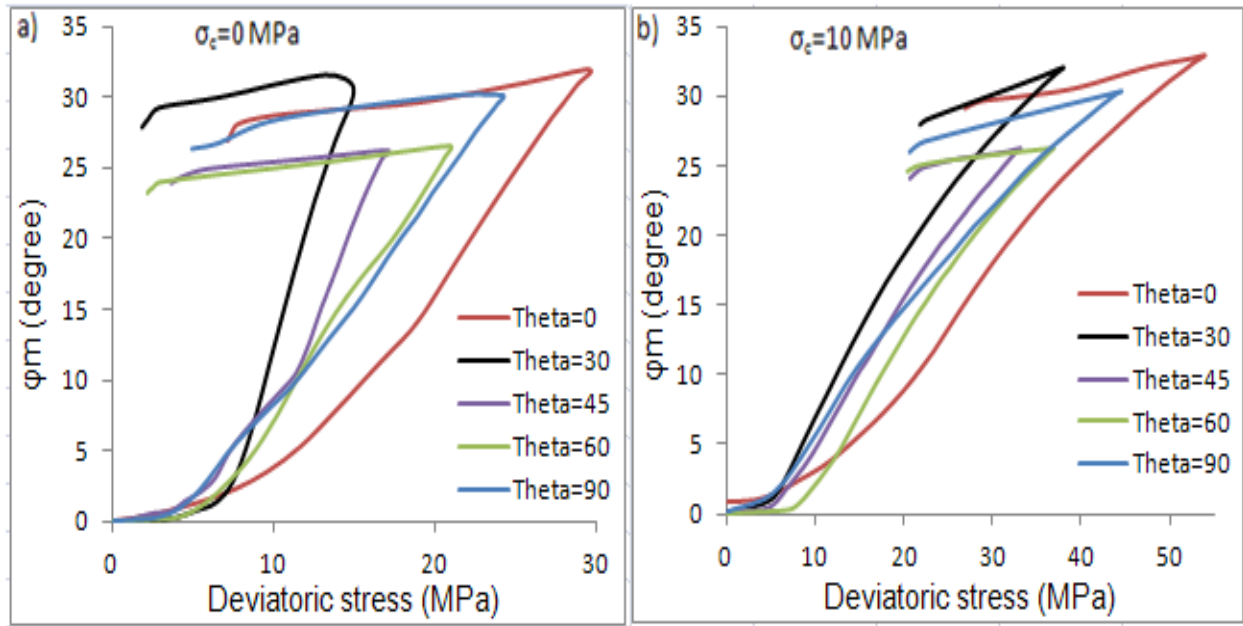


Figure 6.13. Mobilized friction (ϕ_m) versus $(\sigma_1 - \sigma_3)$ for different values of θ and two different values of confining pressure: 0 and 10 MPa.

Based on the numerical results (Figs 6.6, 6.7, 6.8, 6.9, and 6.13), a general equation is proposed for estimating the mobilized friction, ϕ_m , at any stress level and for any θ . The ϕ_m is expressed as a function of stress level, friction angle at peak strength (ϕ_{peak}), and peak strength $((\sigma_1 - \sigma_3)_{peak})$.

$$\phi_m = \phi_{peak} \left(\frac{\sigma_1 - \sigma_3}{(\sigma_1 - \sigma_3)_{peak}} \right) \quad [6.17]$$

ϕ_{peak} and $(\sigma_1 - \sigma_3)_{peak}$ are a function of loading orientation angle, θ .

φ_{peak} :

The values of φ_{peak} are obtained from the results of laboratory experiments for five different loading orientation angles and shown in Fig. 6.14.

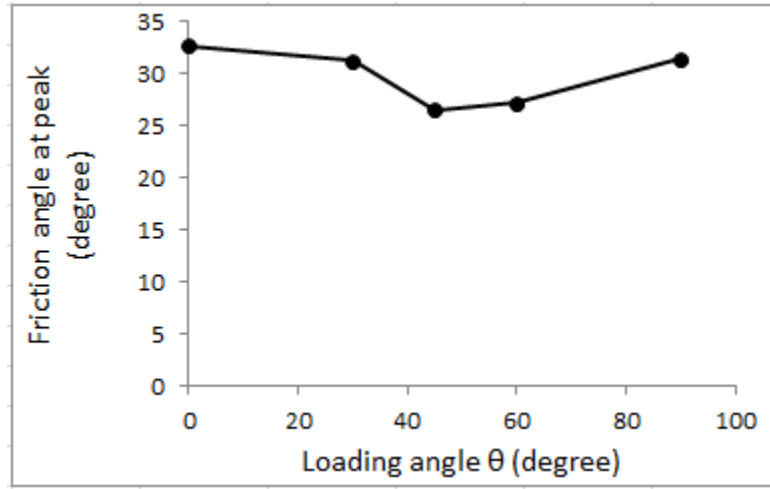


Figure 6.14. Friction angle at peak strength (φ_{peak}) versus loading orientation angle (θ).

The value of $(\sigma_1 - \sigma_3)_{peak}$ at any loading orientation angle can be obtained using Equation [6.15].

The equation [6.17] is valid up to the peak strength. In the post-peak strength region, a linear relationship can be established between φ_m and deviatoric stress $(\sigma_1 - \sigma_3)$.

6.6.3. Development of two general equations for estimating the damage

The numerical results (Figs 6.6, 6.7, 6.8, and 6.9) indicate that the numerical approach reproduces stress-strain curves including unloading-reloading cycles. The amount of damage caused to the elastic properties of the argillite due to the inelastic deformations can be estimated using Equations [6.3] and [6.4]. In these equations, the effective plastic strain (ϵ_{eps}) is used as a measure of damage. Figure 6.15 describes deviatoric stress-effective plastic strain curves for five different values of θ and two different values of confining pressure: 0 MPa and 10 MPa. The ϵ_{eps} is influenced by θ and increases with the increase in confining pressure.

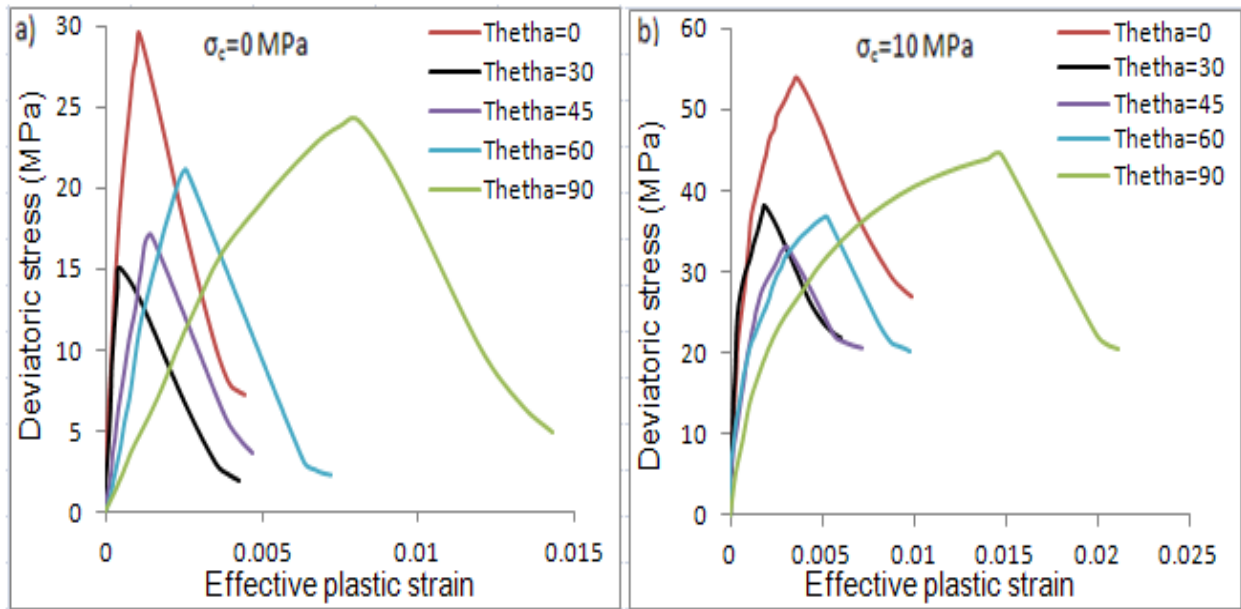


Figure 6.15. Deviatoric stress- ϵ_{eps} curves for five different values of θ and two different values of confining pressure: 0 MPa and 10 MPa.

The numerical results indicate that the value of the material parameter χ in Equations [6.3] and [6.4] is constant and equal to 30. The amount of damage is a function of θ and stress level.

Figure 6.16 shows the effect of damage on the elastic modulus in axial-direction for five different values of θ and two different values of confining pressure: 0 and 10 MPa.

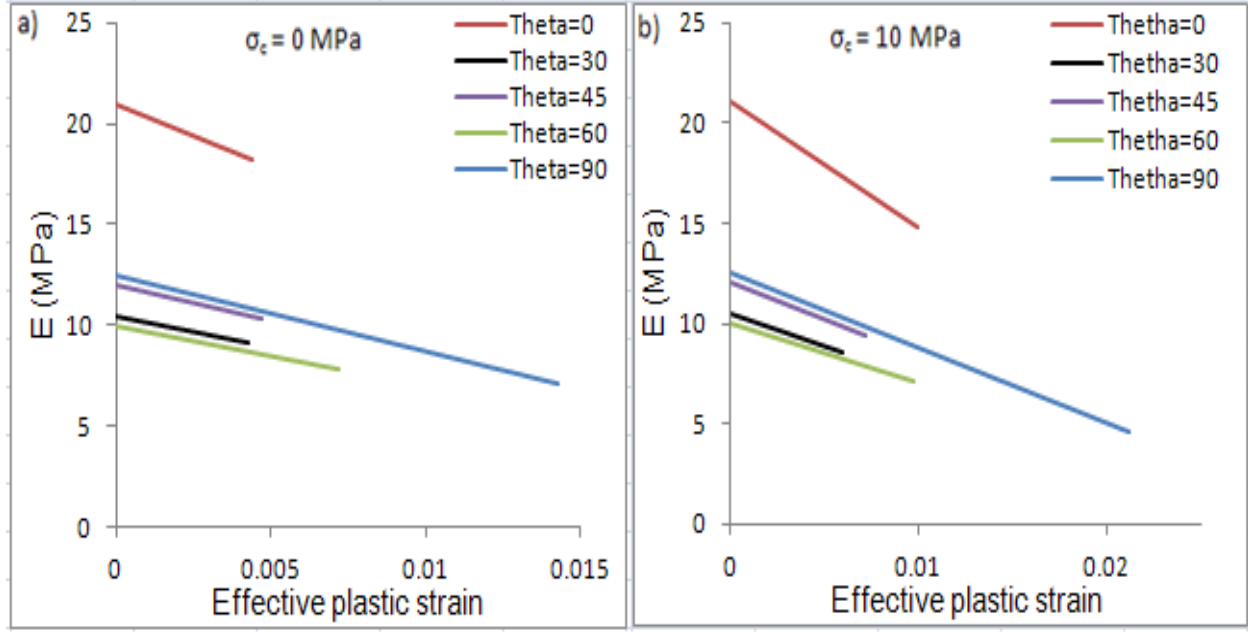


Figure 6.16. Effect of damage on the elastic modulus in axial-direction for two different values of confining pressure: 0 and 10 MPa.

Thus, in any direction x , y , or z the effect of damage on the elastic modulus and shear moduli of the argillite can be estimated using the following two equations.

$$E_i = E_{i0}(1 - \chi \varepsilon_{eps}) \quad [6.18]$$

$$G_{ij} = G_{ij0}(1 - \chi \varepsilon_{eps}) \quad [6.19]$$

where χ is a constant and equal to 30, and ε_{eps} is the effective plastic strain.

6.7. Application of the elastoplastic-damage model to simulate laboratory experiments

The second part of the experimental results (Chapter 4) is used to validate the model. The model is applied to simulate five triaxial tests with confining pressure of 4 MPa and five confined cyclic tests with confining pressure of 10 MPa. In each category of the tests, five different loading orientations are considered: $\theta=0^\circ$, 30° , 45° , 60° , and 90° .

For each increment of strain, the Equations [6.14] and [6.17] are used to calculate the values of mobilized strength parameters, c_m and ϕ_m . These values are supplied to the yield/failure criterion in the next increment of strain. Also, for each increment of strain the Equations ([6.18] and [6.19]) are used to calculate the amount of damage and update the elastic properties of the argillite accordingly. An algorithm for carrying out such calculations is described in Fig. 6.17.

First, the results of the numerical simulations of triaxial tests with confining pressure of 4 MPa are described. Subsequently, the results of the numerical simulations of cyclic tests with confining pressure of 10 MPa are described.

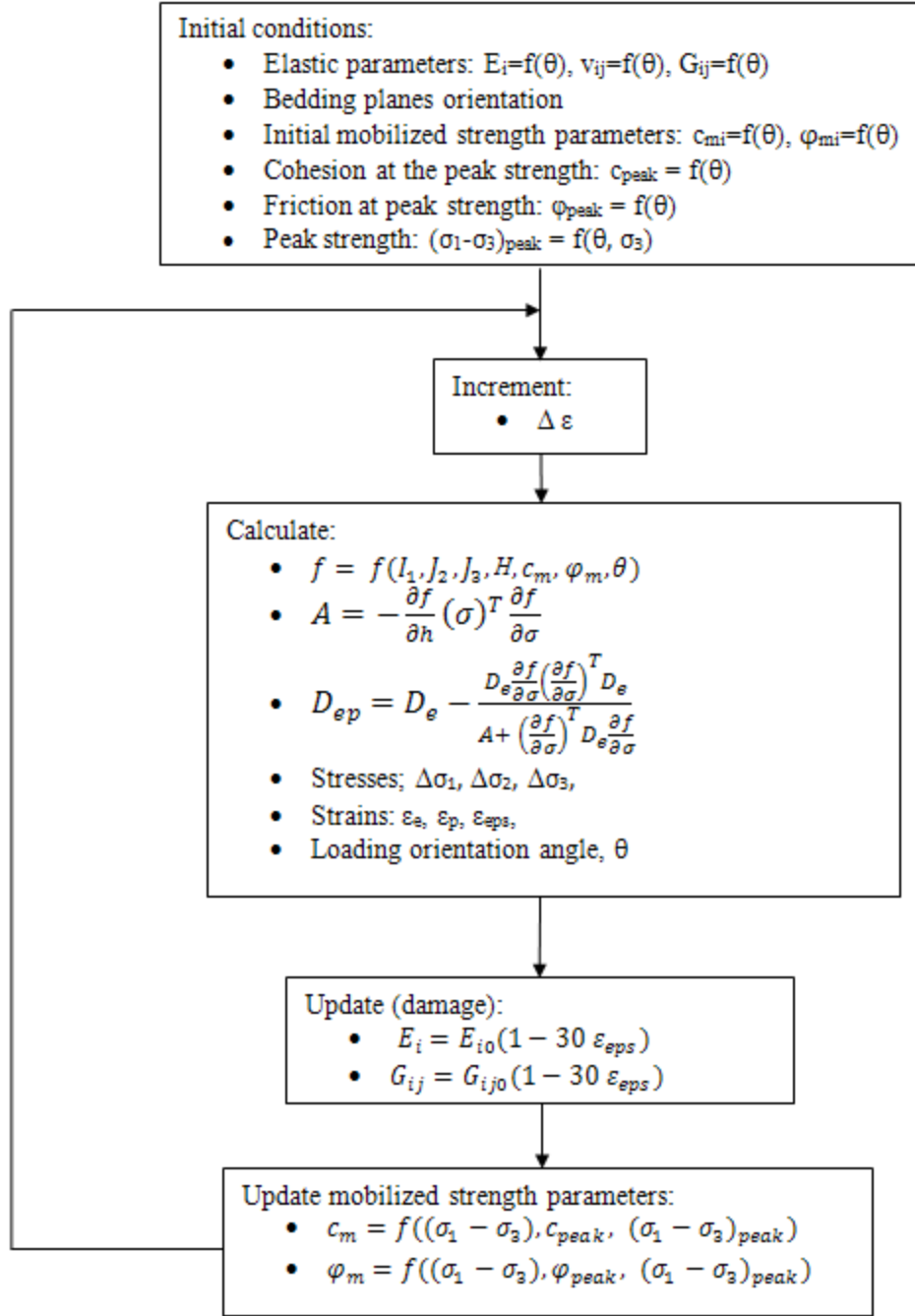


Figure 6.17. An algorithm for carrying out the calculations.

6.7.1. Application of the model to simulate triaxial tests with confining pressure of 4 MPa

The deviatoric stress-strain curves obtained from the modeling of triaxial tests with confining pressure of 4 MPa are shown together with the corresponding curves obtained from the experiments. Figures 6.18 and 6.19 shows the deviatoric stress-axial strain curves and deviatoric stress-volumetric strain curves obtained for five different values of loading orientation angle, respectively.

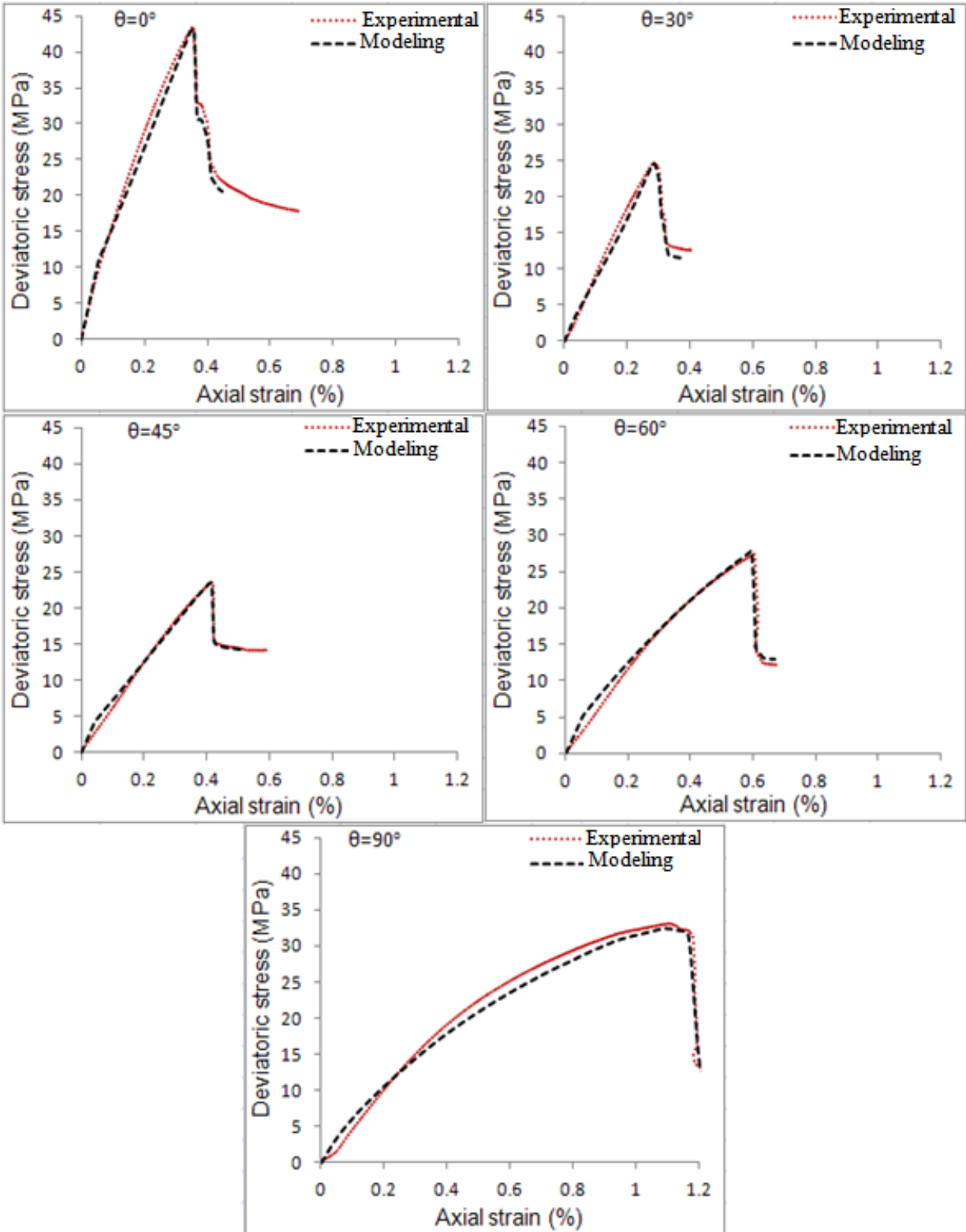


Figure 6.18. Deviatoric stress-axial strain curves obtained from modeling and experiments: triaxial tests with confining pressure of 4 MPa.

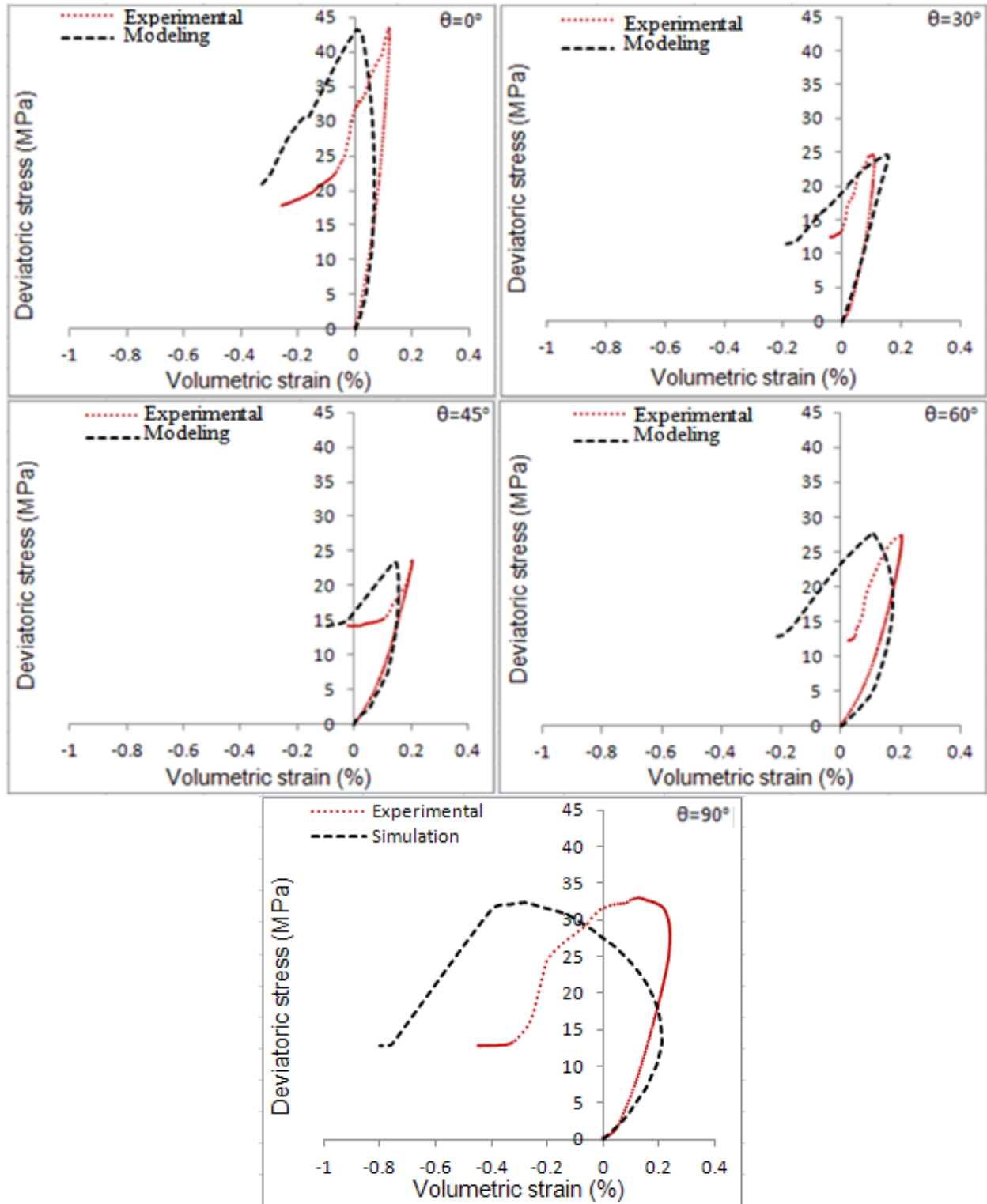


Figure 6.19. Deviatoric stress-volumetric strain curves obtained from modeling and experiments: triaxial tests with confining pressure of 4 MPa.

6.7.2. Application of the model to simulate cyclic tests with confining pressure of 10 MPa

Only a couple of cycles are simulated in each test. The deviatoric stress-strain curves obtained from the modeling of cyclic tests are shown together with the corresponding curves obtained from the experiments. Figure 6.20 shows deviatoric stress-axial strain curves. Figure 6.21 shows the deviatoric stress-volumetric strain curves. In terms of the performance, the model is good but does not capture all the details.

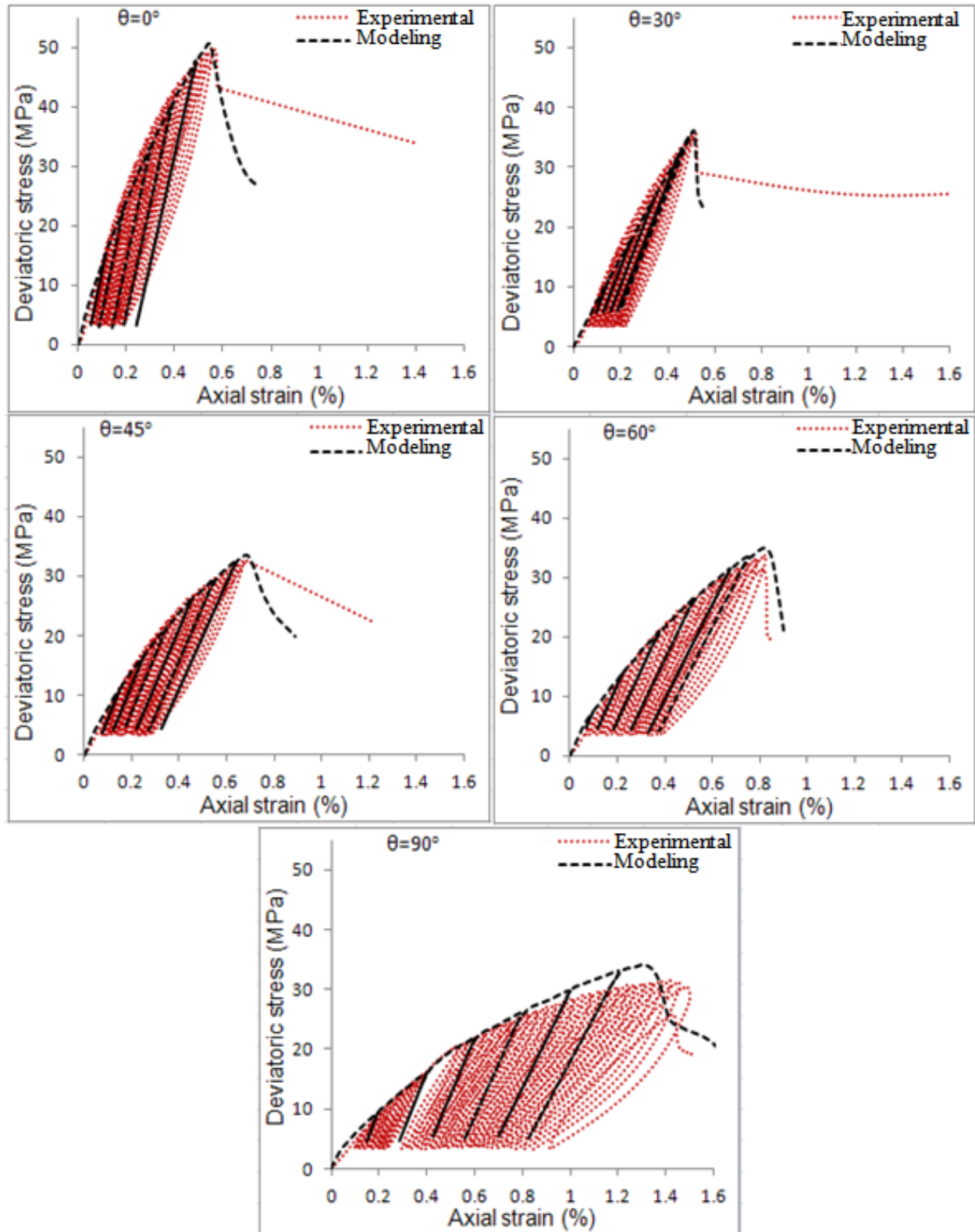


Figure 6.20. Deviatoric stress-axial strain curves obtained from modeling and experiments: confined cyclic tests with confining pressure of 10 MPa.

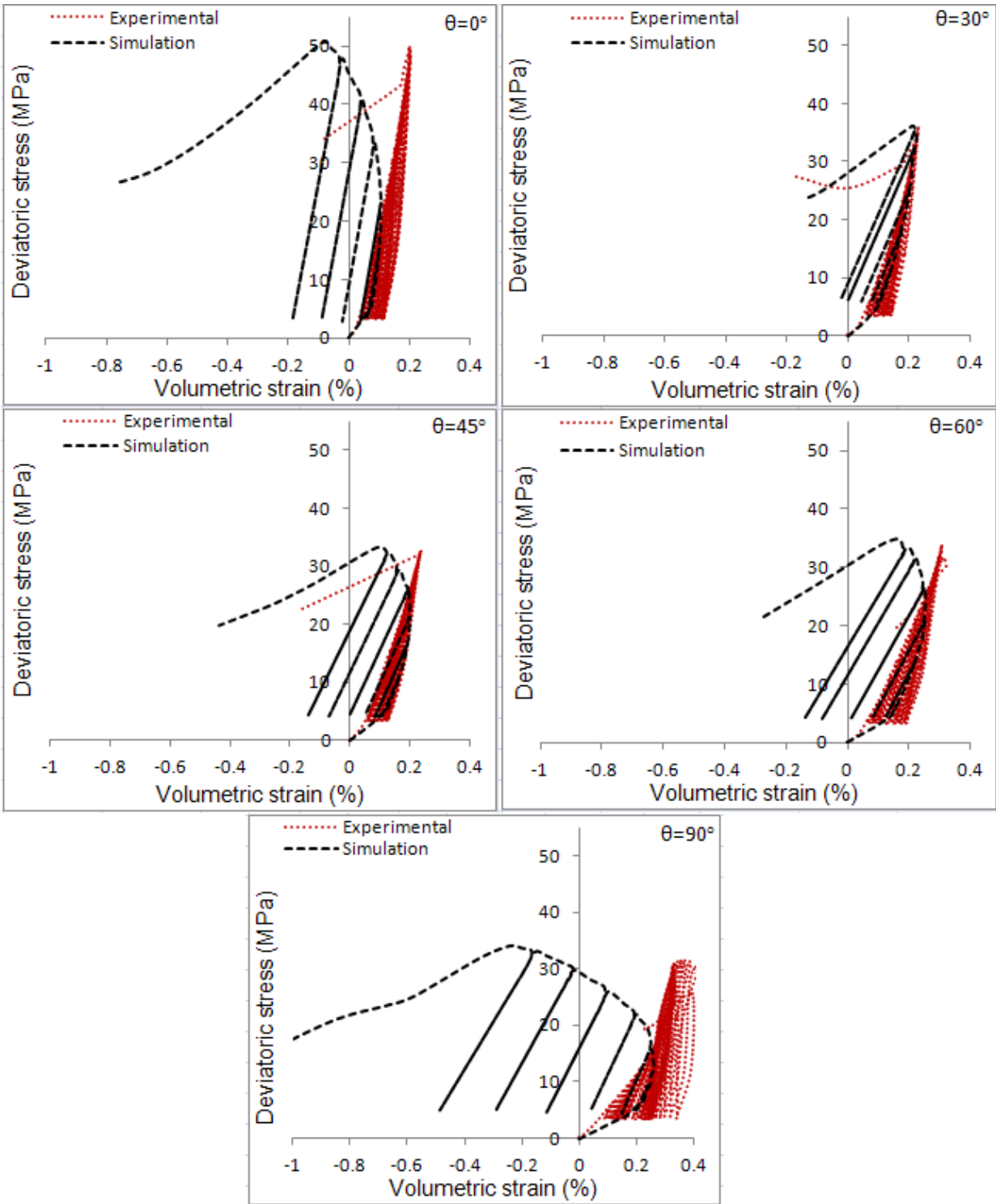


Figure 6.21. Deviatoric stress-volumetric strain curves obtained from modeling and experiments: confined cyclic tests with confining pressure of 10 MPa.

6.8. A comparison of the model to the existing models

Stress-strain relationship for most rocks consists of microcracking closure zone, the elastic part, elastoplastic part, and post-peak region. Many models have been developed to predict the elastoplastic behaviour of sedimentary rocks; see for example, You et al. (2011); Lisjak et al. (2014); and Chen et al. (2010). These models require explicit consideration of bedding planes, the use of tensors of different orders, and/or the use of microstructure tensor (complex mathematical formulations). Here are some examples of the models.

6.8.1. Distinct Lattice Spring Model (DLSM) (You et al., 2011):

The DLSM model is a microstructure based numerical model. It involves explicit considerations of the microstructure of the material. The material is discretized into mass particles linked through distributed bonds as shown in Fig. 6.22. The particles and springs make the whole system which represents the material.

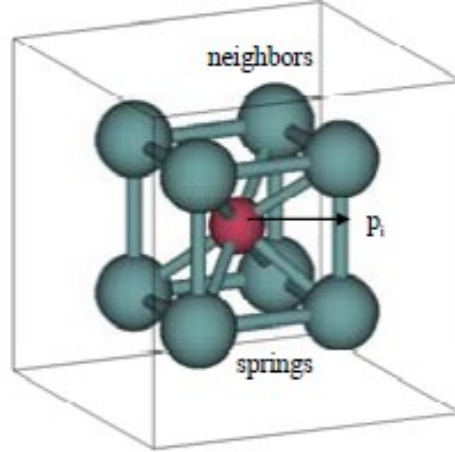


Figure 6.22. The physical model of DLSM (You et al., 2011).

The relationships between the micromechanical parameters and the macro material constants, i.e. the Young's modulus and the Poisson ratio, are as follows:

$$k_n = \frac{3E}{\alpha^{3D}(1-2\nu)} \quad [6.20]$$

$$k_s = \frac{3(1-4\nu)E}{\alpha^{3D}(1-2\nu)(1+\nu)} \quad [6.21]$$

where: k_n is the normal stiffness of the spring, k_s is the shear stiffness, E is the Young's modulus, ν is the Poisson ratio, and α^{3D} is a microstructure geometry coefficient which can be obtained from:

$$\alpha^{3D} = \frac{\sum l_i^2}{V} \quad [6.22]$$

where: l_i is the original length of the i^{th} bond, V is the volume of the geometry model.

The motion of the system can be expressed using the following formulas.

$$[K]u + [C]\dot{u} + [M]\ddot{u} = F(t) \quad [6.23]$$

where: u represents the vector of displacement, $[K]$ is the stiffness matrix, $[M]$ represent the diagonal mass matrix, $[C]$ the damping matrix and $F(t)$ is the vector of external forces on particles.

It was found that the DLSP model can provide a quantitative analysis tool for transverse isotropic materials. But, it needs further improvement on the computational power and on the failure mechanism.

6.8.2. The combined finite/discrete element method (Lisjak et al., 2014):

The combined method consists of two parts: (i) a finite element analysis part for describing the transversely isotropic elasticity (triangular element), and (ii) a discrete element algorithms part for describing the interaction and fractures (cohesive element approach - 4-node crack elements). It is a fully dynamic method (explicit solver) and the external friction is interpreted using Coulomb's law. The elastic part requires the identification of five elastic parameters: E_1 , E_2 , G_{12} , ν_{12} , and ν_{23} .

The modeling approach seems to be able to capture both intra-bedding plane extensional cracks and bedding plane shear failure mechanisms that have been identified as main failure processes

around excavations in Opalinus Clay. However, the model suffers from dimensional limitations, time-dependent behaviour, and the effect of pore water pressure.

6.8.3. Microstructure tensor (Chen et al., 2010) :

The microstructure tensor approach can be used to incorporate the effect of anisotropy in the elastoplastic modeling of transversely isotropic materials. The approach employs a scalar anisotropy parameter(s) which is defined in terms of mixed invariants of stress and microstructure tensors.

A tensor a_{ij} of order 2 can be used to describe the microstructure of the material (Fig. 6.23).

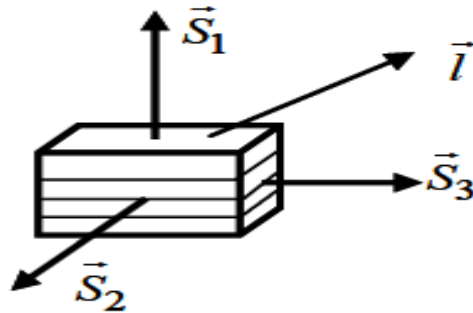


Figure 6.23. Loading vector and axes of the material (Chen et al., 2010).

The spectral decomposition of a_{ij} takes the form:

$$a_{ij} = a_1 S_i^1 S_j^1 + a_2 S_i^2 S_j^2 + a_3 S_i^3 S_j^3 \quad [6.24]$$

In order to define the anisotropy parameter(s), the formulation employs a generalized loading vector that is defined as:

$$L_i = L_j S_i^j; \quad L_j = (\sigma_{j1}^2 + \sigma_{j2}^2 + \sigma_{j3}^2)^{\frac{1}{2}}; \quad (i,j = 1,2,3)$$

Thus the components of L_i represent the magnitudes of traction vectors on the planes normal to the principal material axes.

The unit vector along L_i can be given by:

$$l_i = \frac{L_i}{(L_k L_k)^{\frac{1}{2}}} \quad [6.25]$$

$$L_k L_k = tr \sigma^2 \quad [6.26]$$

The projection of the microstructure tensor on l_i becomes:

$$\eta = a_{ij} l_i l_j \quad [6.27]$$

The scalar variable η referred to as anisotropy parameter, specifies the effect of load orientation relative to material axes.

The equation [6.27] can be rewritten as follows:

$$\eta = \eta_0 (1 + A_{ij} l_i l_j) \quad [6.28]$$

where: $A_{ij} = \frac{1}{\eta_0} a_{ij} - \delta_{ij}$; $\eta_0 = \frac{1}{3} a_{kk}$, and $A_{ij} = \frac{dev(a_{ij})}{\eta_0}$ is a symmetric traceless operator.

The equation [6.28] can be generalized by considering higher order tensors:

$$\eta = \eta_0 (1 + A_{ij} l_i l_j + A_{ijkl} l_i l_j l_k l_l + \dots) \quad [6.29]$$

The failure function can be expressed in the general form:

$$F = F(\sigma_{ij}, a_{ij}) = F(I_1, I_2, I_3, \eta) \quad [6.30]$$

where: I_1 , I_2 , and I_3 are the basic invariants of the stress tensor.

It seems the modeling approach can capture the elastoplastic behaviour of the material. However, the modeling approach involves complex mathematical formulations and it has limitations in terms of practical engineering applications.

The inelastic behaviour of the Tournemire argillite starts at the beginning of the stress-strain relationship and its mechanical behaviour is highly nonlinear. The elastic part exists only during loading-unloading. This behaviour requires a different approach to model its mechanical behaviour. Here, an approach to formulate an elastoplastic-damage model for the argillite has been proposed. The proposed model consists of a damage model, the concept of mobilized strength parameters, and the classical theory of elastoplasticity. The damage model estimates the amount of damage caused to the elastic properties of the argillite at any stress level and for any loading orientation angle. The concept of the mobilized strength parameters controls the plasticity along the stress-strain curve.

The developed elastoplastic-damage model is capable of predicting the mechanical behaviour of the argillite. The mathematical equations have been developed in terms of the loading orientation angle, θ . The model does not capture all the details but it produces acceptable results. The model is easy to implement in a finite element code and apply to practical engineering problems. Compared to the existing models described above and in the literature, the proposed model does not require a large number of laboratory tests to estimate its six parameters (five elastic parameters and one damage parameter) and also does not involve any complex mathematical formulations. The model can be improved in future studies.

6.9. Summary

The experimental results, numerical results, and modeling results indicate that the developed elastoplastic-damage model can predict the mechanical behaviour of the Tournemire argillite under any loading conditions.

The model can be used as follows:

- A Finite Element Code (toolset) that calculates the loading orientation angle (θ) as shown in Fig. 6.22 should be used.
- The strength parameters at the peak strength, strength parameters at the post-peak strength, and peak strength can be estimated from the experimental data.
- Damage: The damage at any point of the domain (Fig. 6.22) can be estimated at the end of each increment of strain using Equations [6.18] and [6.19]. The elastic properties of the argillite can be updated accordingly. The process should be repeated for each increment of strain.
- Strains and stresses: The concept of mobilized strength parameters and the damage model can be used together with the classical theory of elastoplasticity. Knowing the value of θ and stress level, the values of mobilized strength parameters at any point of the domain (Fig. 6.22) can be estimated at the end of each increment of strain using Equations [6.14] and [6.17]. These values can be supplied to the yield criterion during the next increment of strain. Elastic strains, plastic strains, and stresses can be calculated. The process should be repeated for each increment of strain.
- The Equations [6.14], [6.15], [6.17], [6.18], and [6.19] can be added directly to the toolset or using a script, depends on the design of the user interface of the toolset.

Using effective plastic strains as a measure of damage, the amount of damage caused to the elastic properties of the argillite can be estimated. The elastoplastic damage model is the result of combining the proposed concept of mobilized strength parameters, a damage model, the classical theory of elastoplasticity, and experimental data. The model can predict the elastoplastic behaviour of the Tournemire argillite. So far a widely accepted anisotropic failure criterion for

sedimentary rocks is not established yet, but if such criterion is established and used it should result in better results.

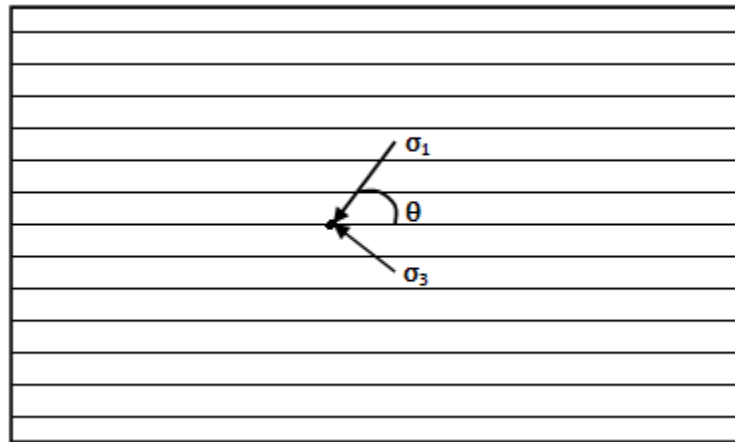


Figure 6.22. A rock mass characterized by the presence of horizontal closely spaced bedding planes such as Tournemire argillite.

CHAPTER 7

CHARACTERIZATION OF EDZ AROUND TWO DGRs IN TWO SEDIMENTARY ROCKS: TOURNEMIRE ARGILLITE AND COBOURG LIMESTONE

7.1. Background

Argillaceous sedimentary formations are characterized with very low permeability, strong capacity for radionuclide retention, and acceptable strength and deformation properties. They are investigated by many research groups for being candidates for hosting deep geologic repositories (DGRs). Development of DGRs within such formations can generate coupled and time-dependent THMC processes within the rock mass. As a result, an EDZ could develop within the vicinity of the excavated areas. More permeable than the undisturbed rock, the EDZ is likely to be a preferential pathway for water and gas flow and it could be a potential exit pathway for the radioactive waste. The size of the EDZ at the end of excavation and its evolution over time needed to be evaluated to ensure the ability of the rock mass to confine the radionuclides.

In recent decades, numerical simulations have been carried out to study the initiation and evolution of the EDZ around underground openings; see for example, Rutqvist et al. (2009) and Wei and Ren (2012). In most of the cases, it was found that the simulation results were in agreement with field measurements. The objective of this Chapter is to use the experimental and modeling results described in Chapters 4, 5, and 6, and data and equations from open literature: (1) to simulate the hydromechanical (HM) behaviour of the Tournemire argillite around a gallery excavated in 2003 at the Tournemire site (France), (2) to simulate the mechanical (M) and hydromechanical (HM) behaviour of the Cobourg limestone around an emplacement room at the Bruce site (Canada), (3) to estimate the initial size of the EDZ around both DGRs, and (4) to discuss the main parameters influencing the initiation of the EDZ and its evolution over time in sedimentary rocks.

Numerical simulations are limited to a period of time of 2 years after the excavation. The effect of ventilation, chemical reactions, and strength degradation are not considered in the numerical analyses. COMSOL Multiphysics v 4.3 toolset is used to carry out the numerical simulations.

7.2. Assumptions made in this Chapter

The following assumption is considered in this Chapter.

Assumption 7.1: Rejeb et al. (2007) indicated that a hydraulic disturbance has been developed within the Tournemire rock mass due to the presence of the century-old tunnel, microcracks, and fractures. The porewater pressure within the rock mass is assumed to be uniformly distributed.

7.3. Damage evolution law and permeability considered in the numerical analyses

The damage evolution law described in Chapter 6 is included into the analysis to account for the effect of damage on the elastic properties of the rock.

$$E_i = E_{i0}(1 - \chi \varepsilon_{eps}) \quad [7.1]$$

$$G_{ij} = G_{ij0}(1 - \chi \varepsilon_{eps}) \quad [7.2]$$

$$\varepsilon_{eps} = \sqrt{\frac{2}{3} \varepsilon_{p_{ij}} \varepsilon_{p_{ij}}} \quad [7.3]$$

where: ε_{eps} is the effective plastic strain. χ is a constant parameter equal to 30 for the Tournemire argillite. E_{i0} is the initial elastic modulus along axis i that can be obtained from the first cycle in a cyclic test. G_{ij0} is the initial shear moduli in direction j on the plane whose normal is in direction i .

The identification of the constant parameter χ for the Cobourg limestone requires the execution of laboratory cyclic tests. The DGR is located at a depth 683 m and the site is not accessible for sampling. Consequently, a value of $\chi = 30$ is assumed and used in the analyses of the mechanical behaviour of the Cobourg limestone.

In order to simulate the increase in permeability due to the damage, similarly to Nguyen (2007), Shirazi and Selvadurai (2005), and Mahyari and Selvadurai (1998), we adopted the Equation

[7.4] (Nguyen, 2007). In this Equation the amount of damage is related to the amount of the equivalent deviatoric strain, ε_d . The ε_d is a function of the mechanical properties of the rock and the state of stress.

$$k = k_{ini} e^h = k_{ini} e^{\psi \varepsilon_d} \quad [7.4]$$

$$\varepsilon_d = \frac{\sqrt{2}}{3} ((\varepsilon_{11} - \varepsilon_{22})^2 + (\varepsilon_{22} - \varepsilon_{33})^2 + (\varepsilon_{11} - \varepsilon_{33})^2 + 6 \varepsilon_{12}^2 + 6 \varepsilon_{13}^2 + 6 \varepsilon_{23}^2)^{\frac{1}{2}} \quad [7.5]$$

where: h is the damage parameter (hydraulic), ε_d is the equivalent deviatoric strain, k_{ini} is the permeability of the undamaged rock, and ψ is a constant parameter (for granite: $\psi=7000$).

7.4. Two-dimensional numerical simulation of a mine-by-test experiment at the Tournemire URL (France)

The mine-by-test experiment around the 2003 gallery at the Tournemire URL has been carried out for nearly two years. The mechanical properties of the Tournemire argillite are anisotropic (Chapter 4) and its hydraulic conductivity is isotropic (Millard and Rejeb, 2008). The natural water content of the argillite is 3.86% and the degree of saturation is around 100% (Chapter 4).

Orthotropic-elastic isotropic-plastic combination is used to simulate the elastoplastic behaviour of the argillite. The effect of the damage on the mechanical and hydraulic properties of the argillite is considered as described above. The elastic properties estimated from laboratory tests (Chapter 5) are used as input to the numerical simulation. The peak strength parameters used into the calculations are the lowest values which obtained at the loading orientation angle $\theta=30^\circ$ (Chapter 5). The Mohr-Coulomb failure criterion is used. The excavation process is simulated by gradually reducing stress and porewater pressure at the boundary of the gallery from the initial state to zero.

7.4.1. Description of the test

During the excavation of the Principal New Gallery in 2003 in the Tournemire argillite, a mine-by-test experiment has been performed, enabling measurements of displacements and stresses at a series of points along the two boreholes (M4 and M5) and porewater pressures at a series of points along the borehole (PH2) as shown in Fig. 7.1 (Millard and Rejeb, 2008). The Main Tunnel is a century-old tunnel. The field measurements were taken at various distances from the gallery wall. They have been recorded during excavation and after, over nearly two years.

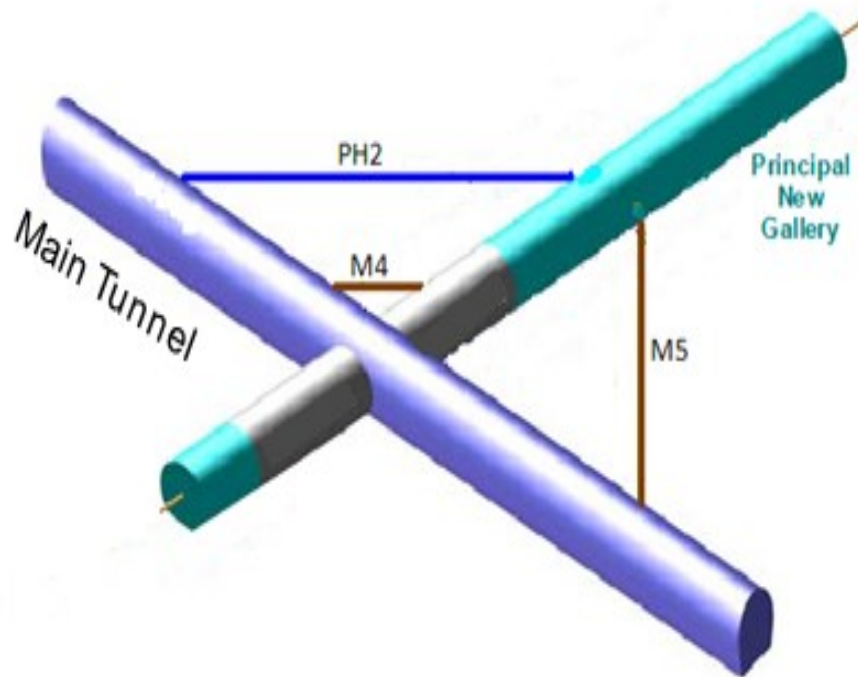


Figure 7.1. A mine-by-test experiment at the Tournemire URL (Millard and Rejeb, 2008).

Due to seasonal effects and presence of local fractures, strong hydraulic disturbances have been developed around the Main Tunnel (Rejeb et al., 2007). The field measurements of the porewater pressure were taken close to the Main Tunnel. Due to non-existence of field measurements of porewater pressure within the rock mass prior to the excavation, the porewater pressure within the rock mass is assumed to be uniformly distributed (Assumption 7.1). Thus, it is expected that the predicted values of porewater pressure would be different from the field measurements. A

coupled hydromechanical analysis of the experiment is carried out with the objective of predicting displacements, stresses, and the general behaviour of the gallery.

7.4.2. Field measurements

The field measurements were obtained from different sources. The following are the details:

1. An initial in-situ stress field of around 4 MPa is measured at the level of the gallery (Rejeb and Cabrera, 2006).
2. The porewater pressure at the level of the gallery is around 0.4 MPa (Rejeb et al., 2007).
3. The field measurements indicate that the elastoplastic behaviour of the rock is dominated by the elastic part (Millard and Rejeb, 2008).
4. The maximum values of stresses developed around the gallery are almost a double of the in-situ stress prior to the excavation (NFPRO Project - Rejeb et al., 2007).
5. Displacements at five points located at various distances from the wall of the gallery (Millard and Rejeb, 2008).
6. Strong hydraulic disturbances have been developed around the Main Tunnel (Rejeb et al., 2007). No field data have been provided of the porewater pressure within the rock mass.
7. Field measurements of the porewater pressure were taken at three points close to the Main Tunnel.
8. The length of the Principal new gallery is about 40 meters (Fig. 7.1).

7.4.3. Mechanical and hydraulic properties of the argillite

The parameters required for the numerical simulation (HM) are extracted from the results of laboratory experiments (Chapter 4). Except for the porosity of the rock which was estimated from the laboratory results (Chapter 4), the rest of the hydraulic properties of the rock were taken from different sources (Rejeb and Cabrera, 2006; Rejeb et al., 2007; Millard and Rejeb, 2008). Table 7.1 summarizes the mechanical and hydraulic properties of the argillite.

Table 7.1. Mechanical and hydraulic properties of the argillite used in the HM analysis.

Parameter	Value	Unit
<i>Mechanics</i>		
Young's Modulus		
In bedding planes	21	GPa
Perpendicular to bedding planes	12.5	GPa
Poisson's ratio		
ν_{12}	0.115	
ν_{23} (normal to tunnel)	0.15	
Shear modulus		
G_{12}	4.5	GPa
Mohr-Coulomb failure criterion		
Cohesion, c	4.4	MPa
Angle of friction, ϕ	31	degree
Density of the rock		
ρ	2550	kg/m ³
<i>Hydraulics</i>		
Hydraulic conductivity, k	1e-14	m/s
Porosity		
ω	0.09	
Specific storage coefficient	5e-7	
Density of the fluid		
ρ_f	1000	kg/m ³
Biot coefficient		
α_B	0.85	

7.4.4. Finite element model

The length of the gallery is about 40 meters. In view of the extent of the gallery, a 2D analysis is performed under the assumption of plane strains. The analysis domain consists of a 30 m in width and 40 m in height of the rock mass as shown in Fig. 7.2. The gallery has a semicircular cross section of about 5 meters diameter. The boundary conditions of the model are as follows. The base of the model is fixed. Only vertical movement of the material is allowed at the vertical boundaries. The top boundary is subjected to a uniform vertical effective stress equivalent to the in-situ vertical effective stress (3.7 MPa). The gallery is maintained at atmospheric pressure, while its boundaries are maintained at pressures equal to the initial porewater pressures prior to excavation. It is assumed that the excavation will take place in 10 days.

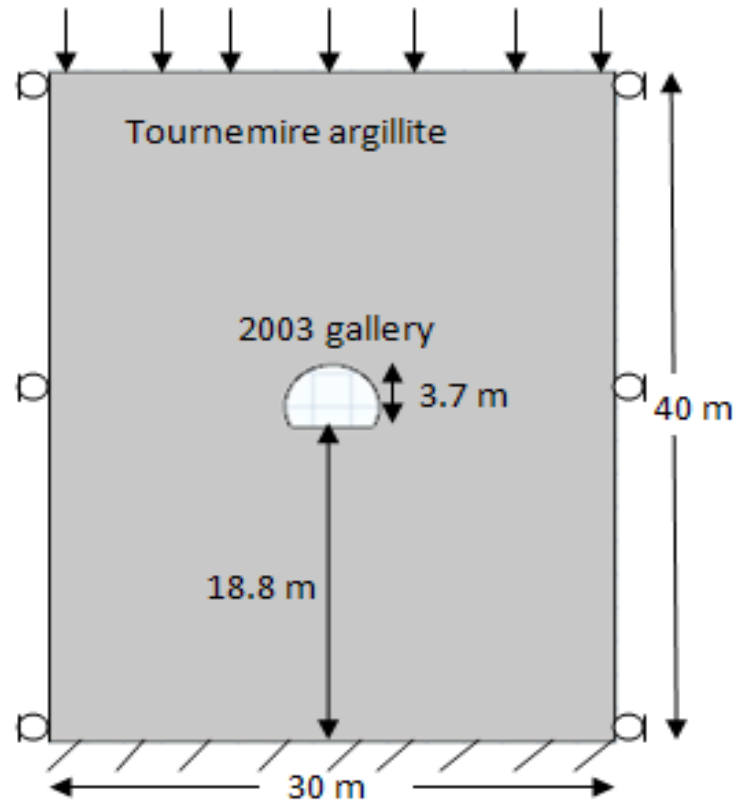


Figure 7.2. The finite element model and boundary conditions.

7.4.5. Interpretation of the numerical results and comparison to field measurements

7.4.5.1. Stress distributions

The state of stress around the gallery before the excavation takes place is shown in Fig. 7.3. The distributions of horizontal compressive stresses and vertical compressive stresses are shown in Figs 7.3.a and 7.3.b, respectively.

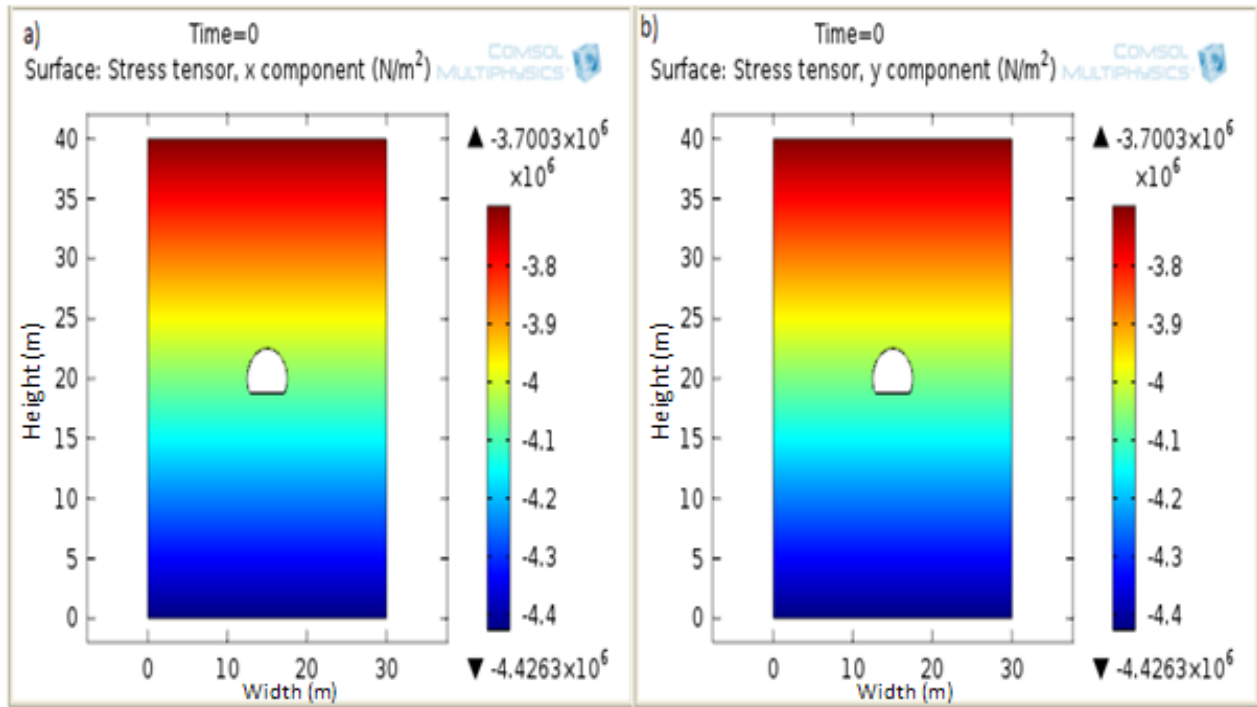


Figure 7.3. Distributions of stresses before the excavation: a) σ_H and b) σ_V .

At the end of the excavation of the gallery, radial effective stresses at the gallery wall become negligible and tangential effective stresses rise, in some locations, up to two times of the initial effective stresses as shown in Fig. 7.4. The corners of the gallery, the left and right sides of the gallery, and small part of the roof of the gallery are subjected to high compressive effective stresses. Compared to the effective stress state before the excavation, a decrease in effective stresses is calculated at the base of the galley after the excavation. The calculated stresses are in agreement with field measurements reported in NFPRO Project (Rejeb et al., 2007).

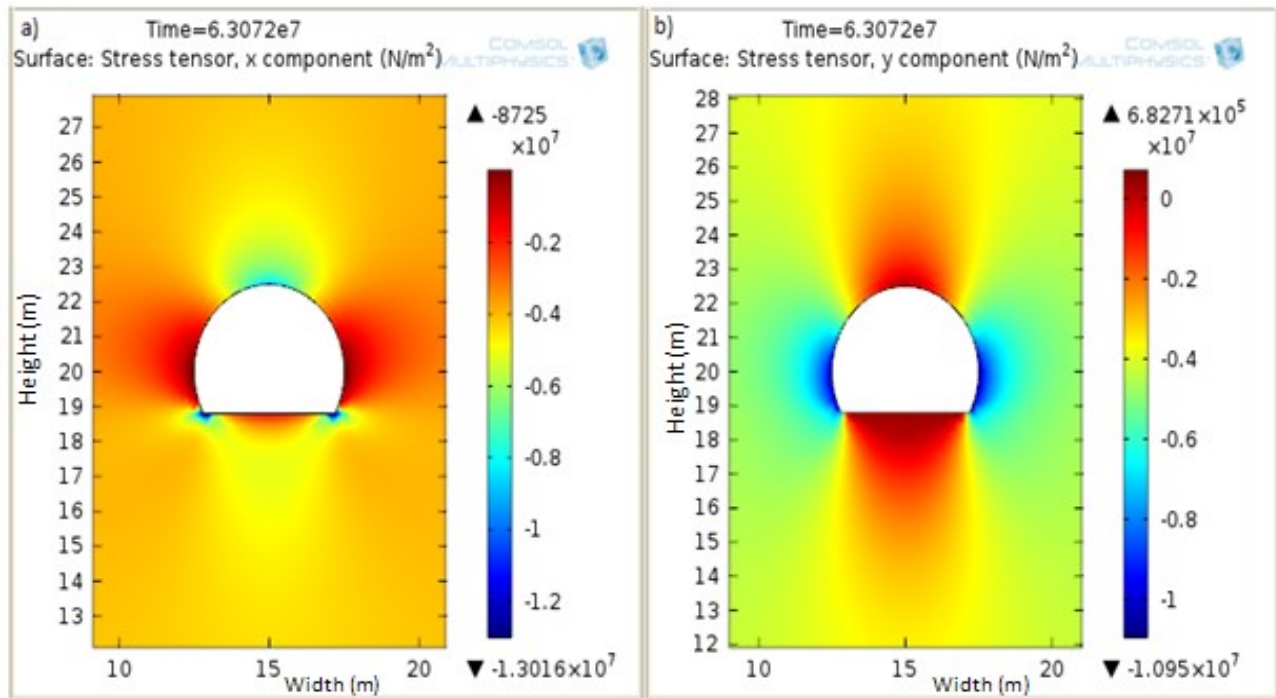


Figure 7.4. Distribution of compressive effective stresses after the excavation; a) σ_H and b) σ_V .

7.4.5.2. Volumetric strains

The excavation of the gallery results in the redistribution of in-situ stresses. As a result, compression and extension areas develop around the gallery as shown in Fig. 7.5. The amount and distribution of the volumetric strain depends mainly on the state of stress, geometry of the gallery, and hydraulic and mechanical properties of the rock mass. Compressive volumetric strains are developed at the corners of the gallery and on the left and right sides of the gallery. Expansive volumetric strains are developed at the base and at the roof of the gallery.

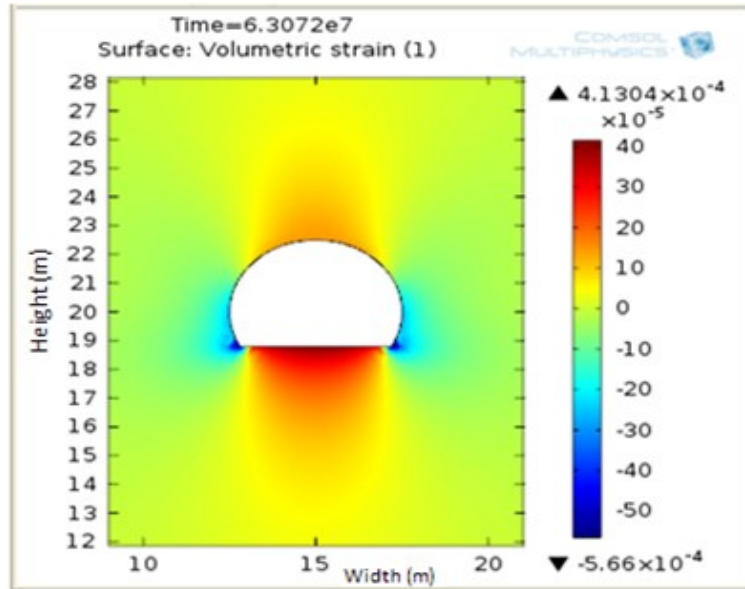


Figure 7.5. Volumetric strain distribution after the excavation.

7.4.5.3. *Effective plastic strains*

At the end of the excavation of the gallery, the failure criterion produces only negligible effective plastic strains, in the order of 10^{-5} , at the corners of the gallery as shown in Fig. 7.6. Thus, the mechanical behaviour of the argillite at the end of the excavation of the gallery is mostly elastic. The results are in agreement with field measurements (Millard and Rejeb, 2008).

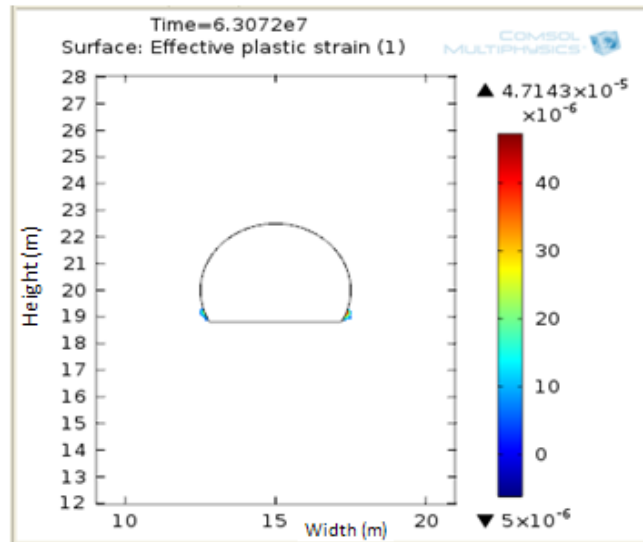


Figure 7.6. Effective plastic strain distribution after the excavation.

7.4.5.4. Displacements

The amount and distribution of displacements around the gallery, as shown in Fig. 7.7, is influenced mainly by the geometry of the gallery, state of stress, and mechanical and hydraulic properties of the rock. A maximum displacement of 1.6 mm is calculated at the crown of the gallery. Displacements are developed at the base and at the roof of the gallery. Negligible displacements are developed on the left and right sides of the gallery.

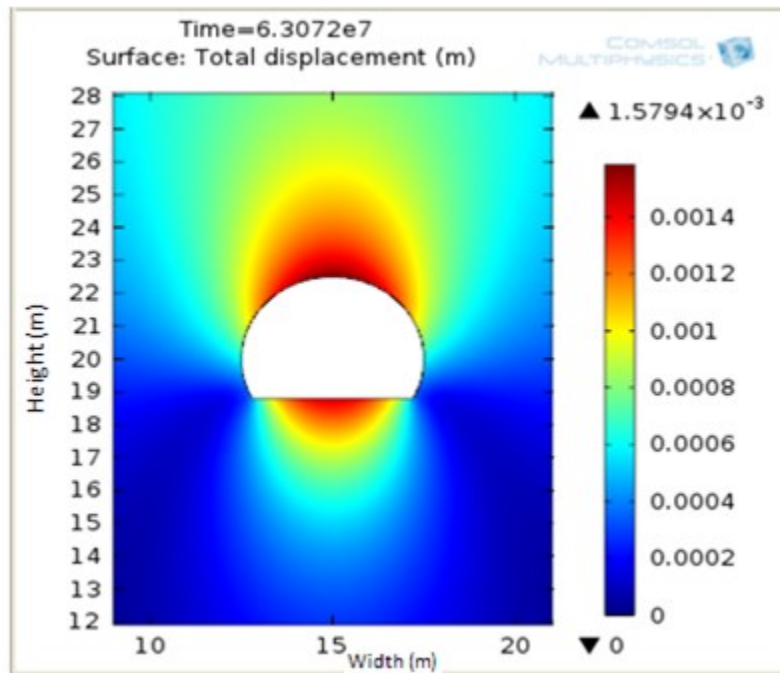


Figure 7.7. Distribution of displacements after the excavation.

In order to compare the calculated and measured displacements, a series of Points are selected from the domain as shown in Fig. 7.8. These Points are of distance of 1 m, 2 m, 3.5 m, 4.5 m, 5 m, and 10 m from the gallery wall.

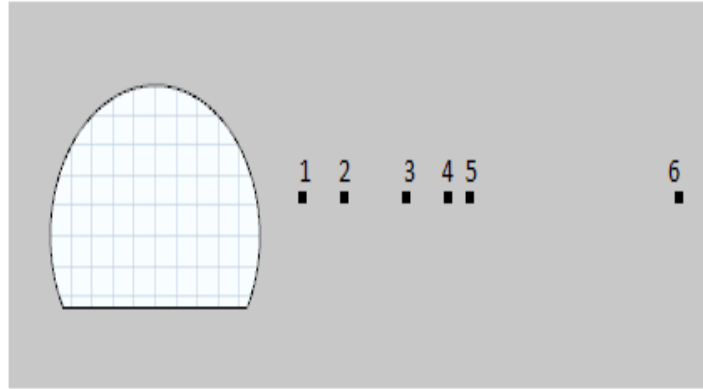


Figure 7.8. Selected Points from the domain.

Figure 7.9 shows the calculated and measured displacements in mm at Points 1, 2, 3, 5, and 6 from the domain. The data are plotted for a period of two years. Until the end of the excavation, a good agreement is obtained between numerical results and field data. Except at point 1, the field measurements indicate that the increase in displacements after the end of the excavation is insignificant. In order to predict better results, time-dependent deformation behaviour of the argillite (which is not the subject of this research work) should be included into the formulations.

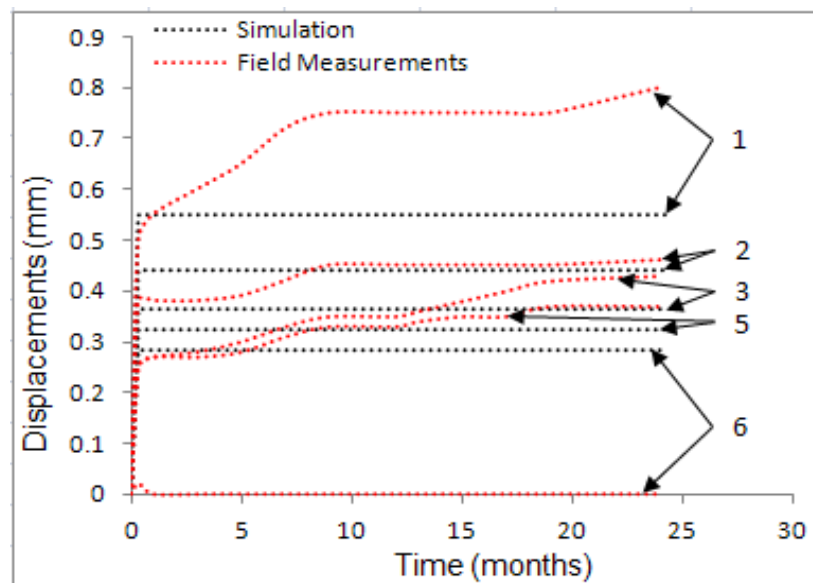


Figure 7.9. Displacements at five selected points from the domain for a period of two years.

7.4.5.5. Porewater pressure distributions

Two cases of permeability are considered:

Case 1: The permeability of the argillite is assumed to be constant and not affected by the damage ($\psi = 0$).

Case 2: The permeability of the argillite is affected by the damage (Equation [7.4]). The effect of the damage on the permeability is expressed as a function of the equivalent deviatoric strain (Equation [7.5]). Figure 7.10 shows the distribution of the equivalent deviatoric strain two years after the excavation.

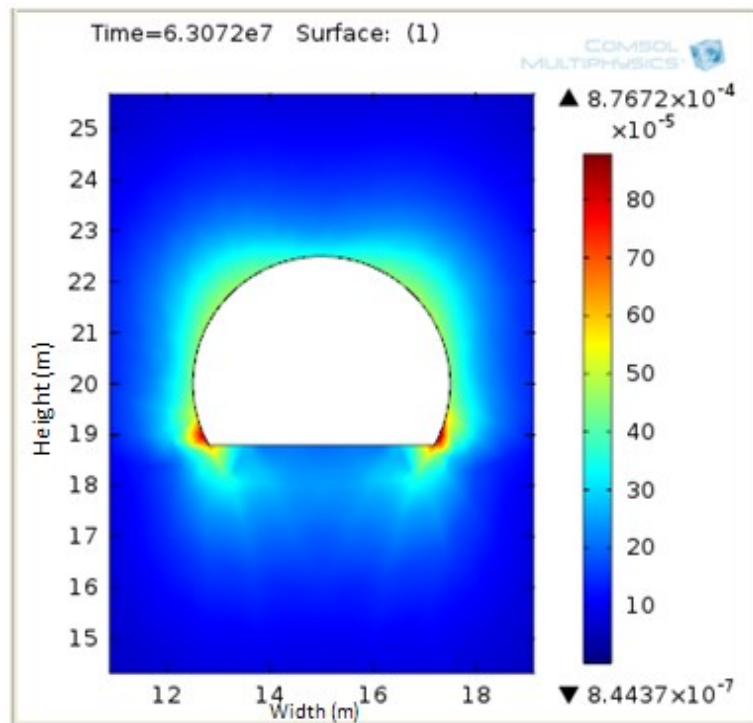


Figure 7.10. Equivalent deviatoric strain two years after the excavation.

Figure 7.11 shows porewater pressure distribution two years after the excavation for four different values of $\psi = 0, 2000, 7000$, and 20000 . In all these cases, the dissipation of porewater pressure can be divided into three subzones: full porewater pressure dissipation subzone, partial porewater pressure dissipation subzone, and no porewater pressure dissipation subzone. Experimental/field data are needed to calibrate the parameter ψ .

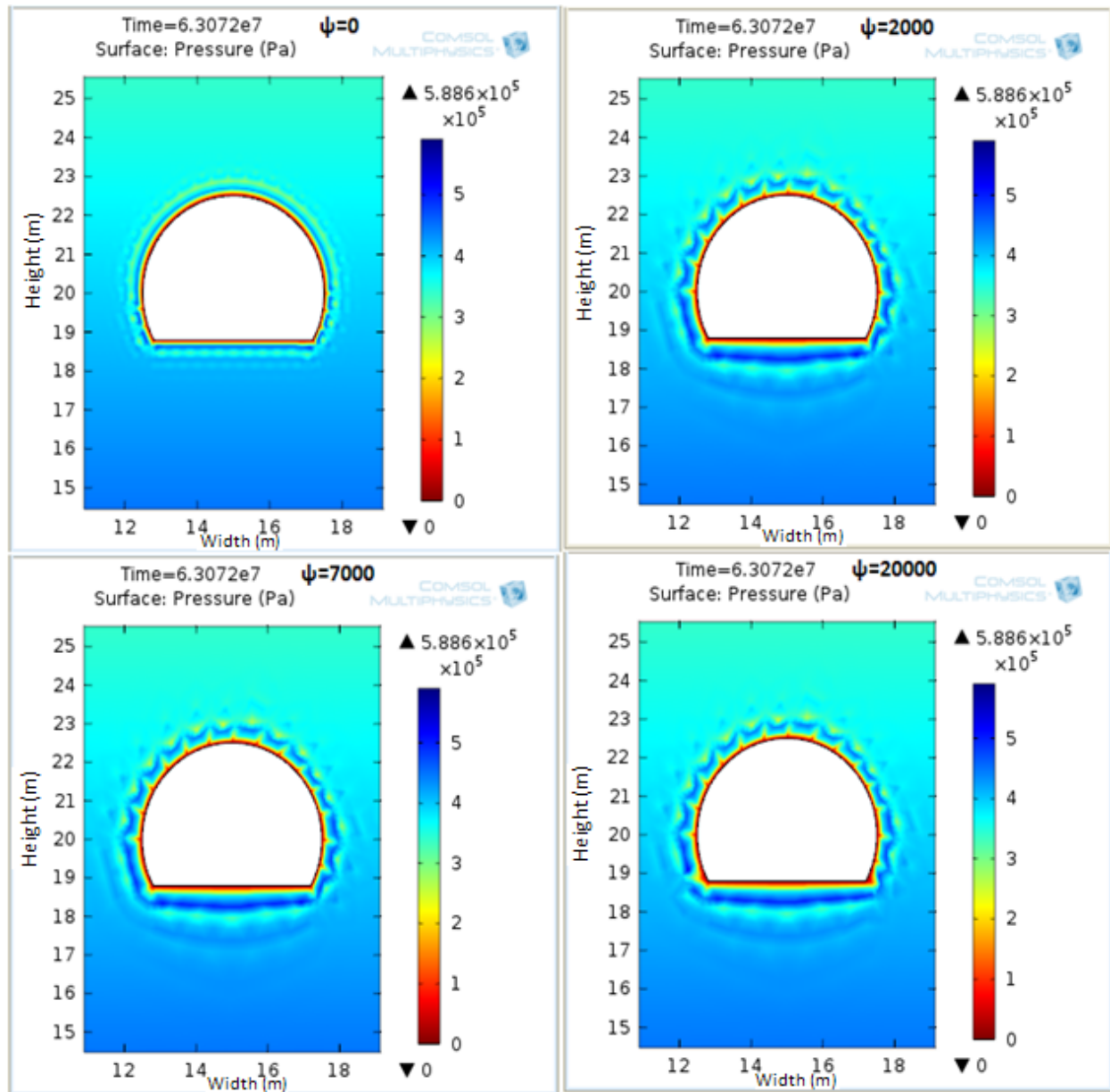


Figure 7.11. Distribution of porewater pressure two years after the excavation for four different values of ψ : 0, 2000, 7000, and 20000.

As mentioned earlier, strong hydraulic disturbances have been developed around the Main Tunnel, such as the development of unsaturated areas, due to seasonal effects and existing fractures. The field measurements were taken close to the Main Tunnel and at three points within the inclined borehole PH2 (Fig. 7.1) to the gallery. No increase in porewater pressure is calculated at the three points. However, field data showed an increase in porewater pressure at the three points. In the field test, the development of compressive areas around the gallery causes

the migration of porewater pressure from these areas towards areas with low porewater pressure (e.g.; unsaturated areas). This process results in an increase in porewater pressure at the three points where the field measurements were taken. In the numerical analysis, the medium is assumed to be fully saturated and the porewater pressure is uniformly distributed with 0.4 MPa at the level of the gallery (Assumption 7.1). It was expected that the predictions of porewater pressure would be different from field measurements and the predicted values of porewater pressure are not compared to field measurements.

7.5. Two and three-dimensional numerical simulations of the mechanical behaviour of the Cobourg limestone around an emplacement room at the Bruce site, Canada

Two and three-dimensional numerical analyses are carried out to simulate the mechanical behaviour of the Cobourg limestone around an emplacement room at the Bruce site. The mechanical properties of the Cobourg limestone are isotropic (OPG Report-2, 2008). The repository is located at a depth 683 m. The in-situ stresses at the level of the repository are: $\sigma_H = 2\sigma_V$, $\sigma_h = 1.2\sigma_V$, and $\sigma_V = 18$ MPa (Gartner Lee Limited, 2008). In the analysis, it is conservatively assumed that σ_H is acting perpendicular to the axis of the emplacement room and the direction of σ_h is parallel to the axis of the emplacement room. The effect of damage on the mechanical properties of the limestone is considered as described above. The Mohr-Coulomb failure criterion is used in the elastoplastic part. For the 2D analysis, the excavation process is simulated by gradually reducing stresses at the boundary of the room from the initial state to zero. For the 3D analysis, the excavation process is simulated by dividing the room into 10 slices. In order to simulate the construction stages, the first slice is excavated (gradually reducing stresses at the boundary of the room from the initial state to zero) starting at the room face. This process is repeated sequentially for all slices until the tenth slice. The objective of this section is to describe the numerical results and compare the results of the two-dimensional analysis with the results of the three-dimensional analysis.

7.5.1. DGR at the Bruce site

The DGR consists of many emplacement rooms, access tunnels, and shafts. The preliminary design of the DGR and bedrock stratigraphy can be found in (Jensen et al., 2009). The design of the emplacement rooms is shown in Fig. 7.12. Also, in Fig. 7.12, three rock layers are shown. The emplacement rooms are excavated in the Cobourg limestone. The horizontal distance between two consecutive emplacement rooms is 16 m. The length of the emplacement rooms is 10 m.

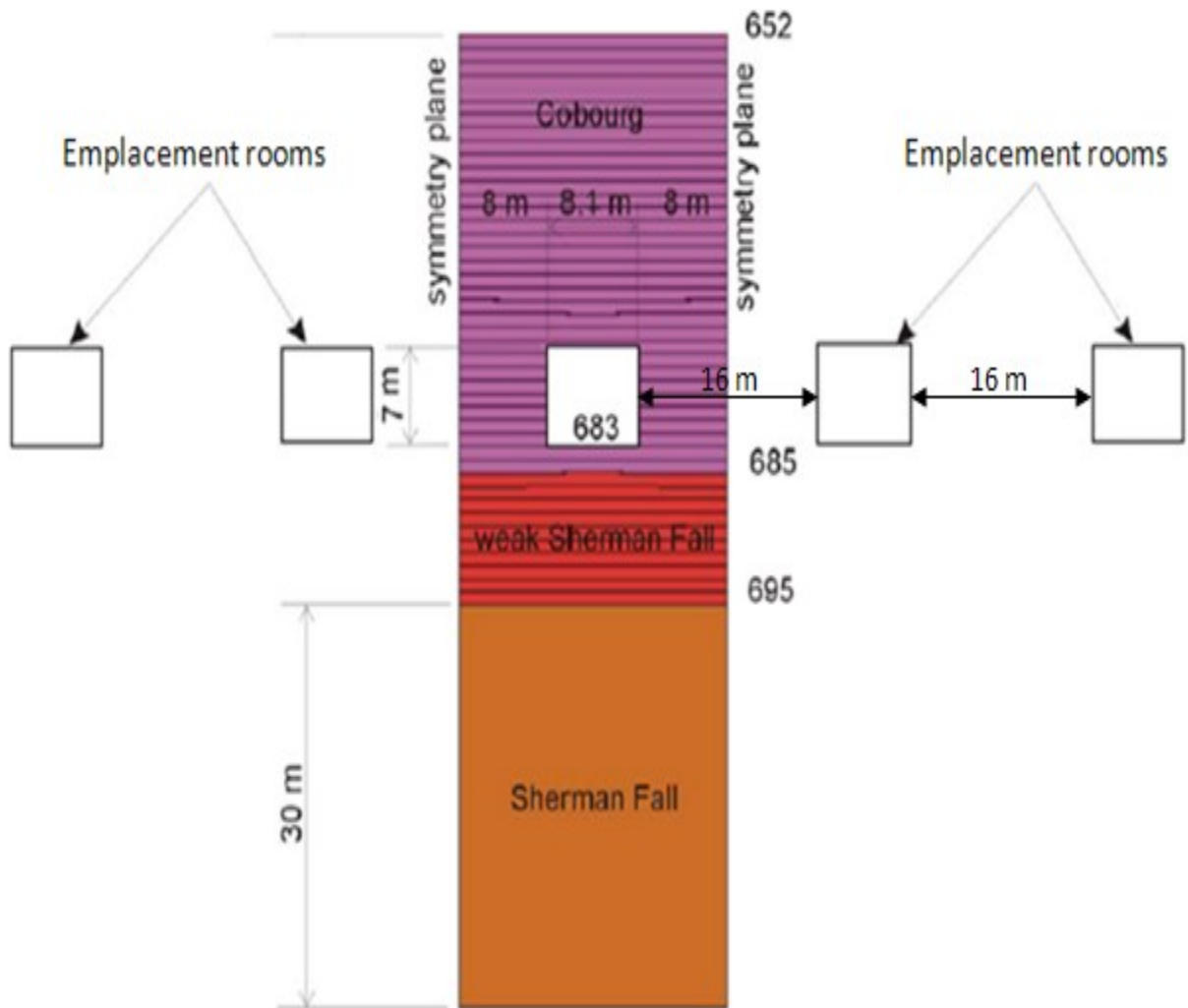


Figure 7.12. Design of the emplacement rooms at the Bruce site (OPG Report-2, 2008).

7.5.2. Finite element model

The finite element model consists of one emplacement room and the three rock layers that surround the room. The symmetry conditions are considered on the left and right side of the room. The room is of rectangular shape: 7 m height x 8 m width. Figure 7.13 shows the model geometry, rock layers, and boundary conditions. In 3D analysis, the dimensions of the analysis domain are 24 m along the x-axis (horizontal), 50 m along the z-axis (vertical), and 30 m in the y-axis. In 2D analysis, the dimensions of the analysis domain are 24 m along the x-axis (horizontal) and 50 m along the y-axis (vertical).

The boundary conditions of the model are as follows. The base of the model is fixed. Symmetry conditions are imposed on the left and right sides of the room. The top boundary is subjected to a uniform vertical stress equivalent to the in situ vertical stress (17.32 MPa). In 3D analysis, the remaining vertical boundaries are rollers.

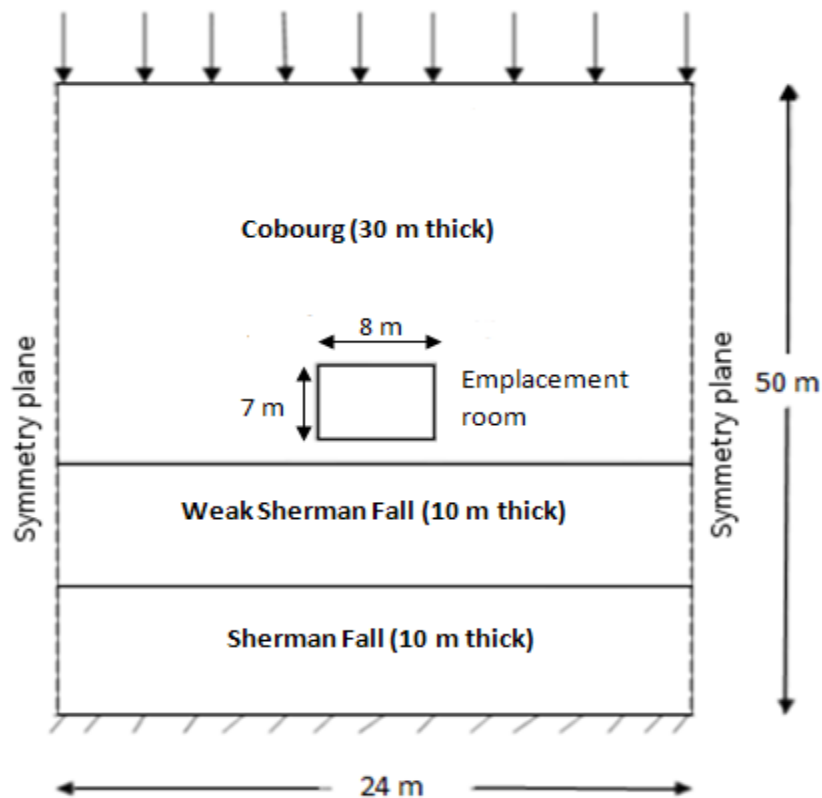


Figure 7.13. Finite element model and boundary conditions.

7.5.3. Mechanical properties of the rock layers

The mechanical properties required for numerical simulations for the three rock layers are described in Table 7.2.

Table 7.2. Mechanical properties for the rock units (OPG Report-2, 2008).

Type of rock	E (GPa)	ν	c (MPa)	ϕ , deg	ρ (kg/m ³)
Sherman Fall	26.45	0.23	12.4	45	2700
Weak Sherman Fall	10.74	0.08	6.23	45	2700
Cobourg	36.04	0.19	19.09	45	2700

7.5.4. Interpretation of the numerical results

The focus of this section is to describe and interpret the numerical results and compare the results obtained from 2D analysis to the results obtained from 3D analysis.

7.5.4.1. Stress distribution

Figure 7.14 shows the state of stress around the emplacement room before the excavation takes place. The distributions of horizontal compressive stresses and vertical compressive stresses are shown in Figs 7.14.a and 7.14.b, respectively.

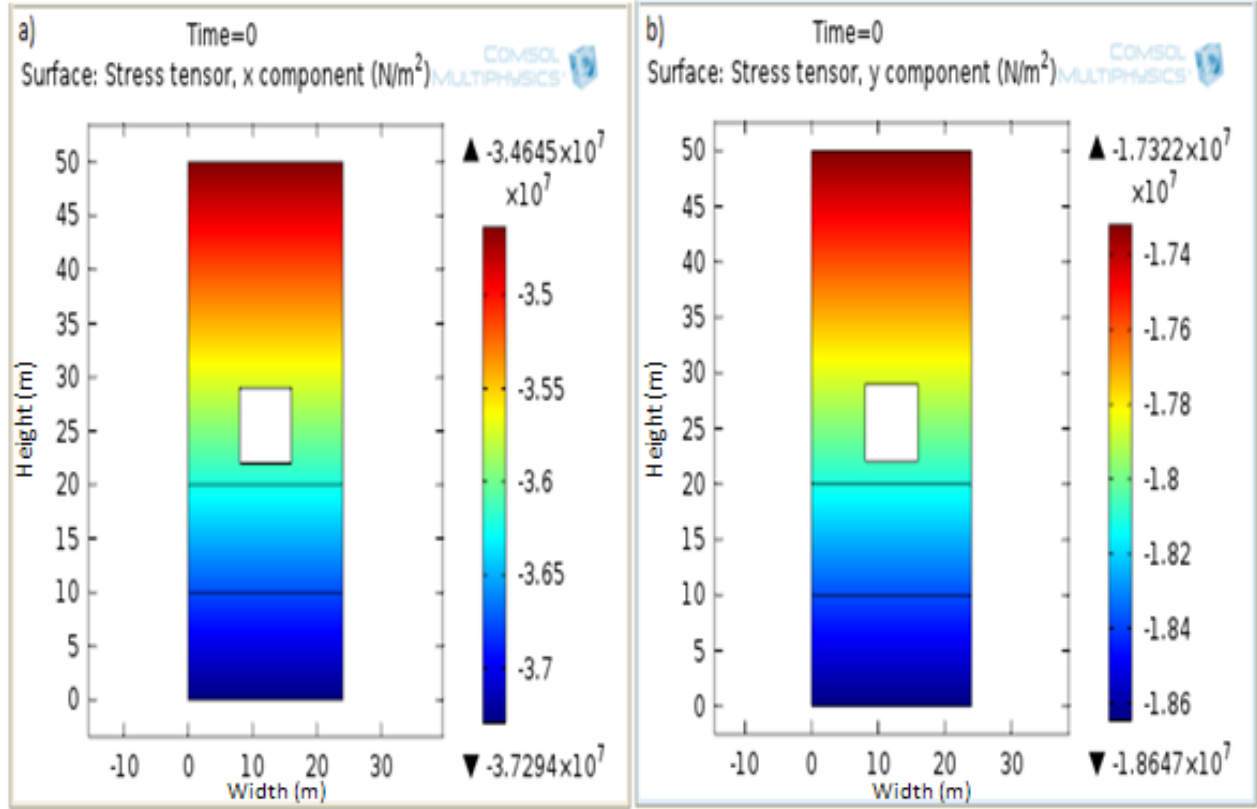


Figure 7.14. Distributions of stresses before the excavation: a) σ_H and b) σ_V .

Figure 7.15 shows the distribution of horizontal compressive stresses at the end of the excavation. Before the excavation, the magnitude of the horizontal compressive stress acting at the level of room was about 36 MPa (Fig. 7.14.a). At the end of the excavation, the magnitude of the maximum horizontal compressive stress in the whole analysis domain is 92 MPa and 117 MPa for the 2D analysis (Fig. 7.15.a) and 3D analysis (Fig. 7.15.b), respectively, and it occurs at the corners of the emplacement room.

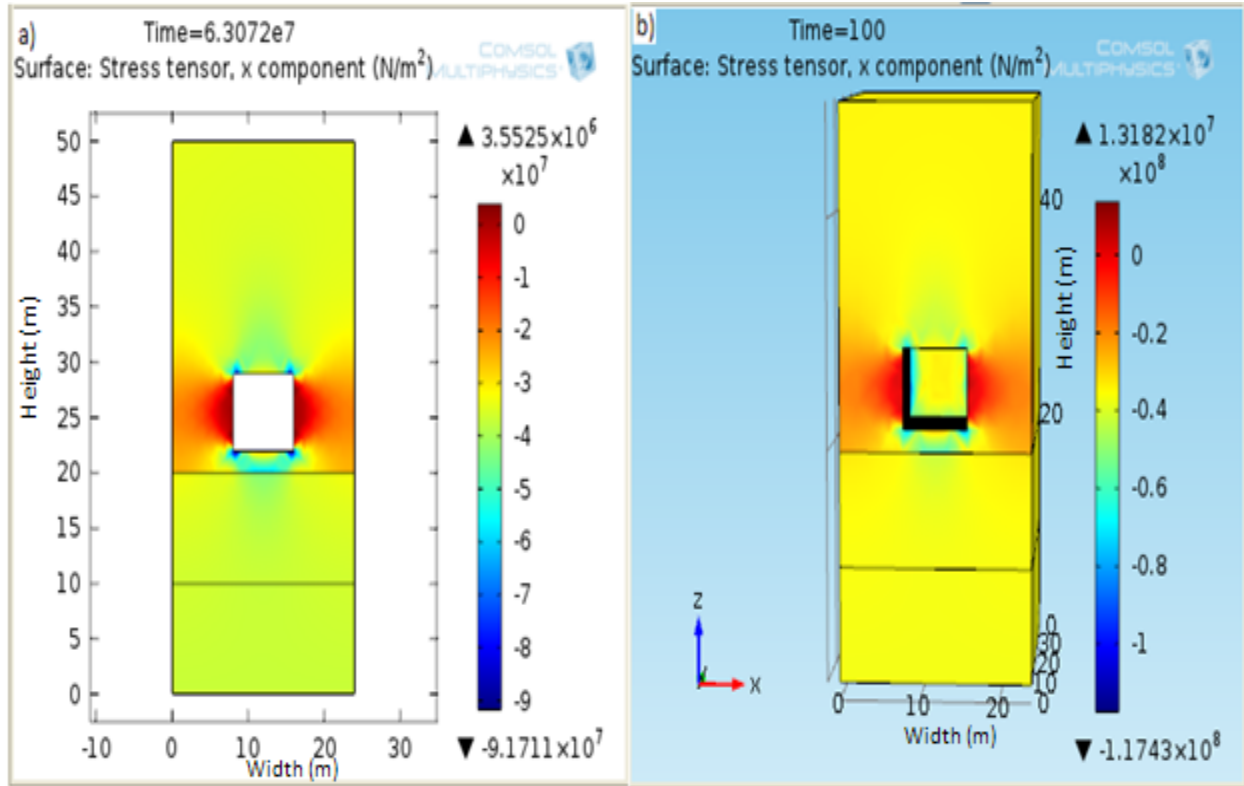


Figure 7.15. Distribution of σ_H at the end of the excavation: a) 2D and b) 3D.

Figure 7.16 shows the distribution of vertical compressive stresses at the end of the excavation. Before the excavation, the magnitude of the vertical compressive stress acting at the level of excavation was about 18 MPa (Fig. 7.14.b). At the end of the excavation, the magnitude of the maximum vertical compressive stress in the whole analysis domain is 62 MPa and 90 MPa for the 2D analysis (Fig. 7.16.a) and 3D analysis (Fig. 7.16.b), respectively, and it occurs at the corners of the emplacement room.

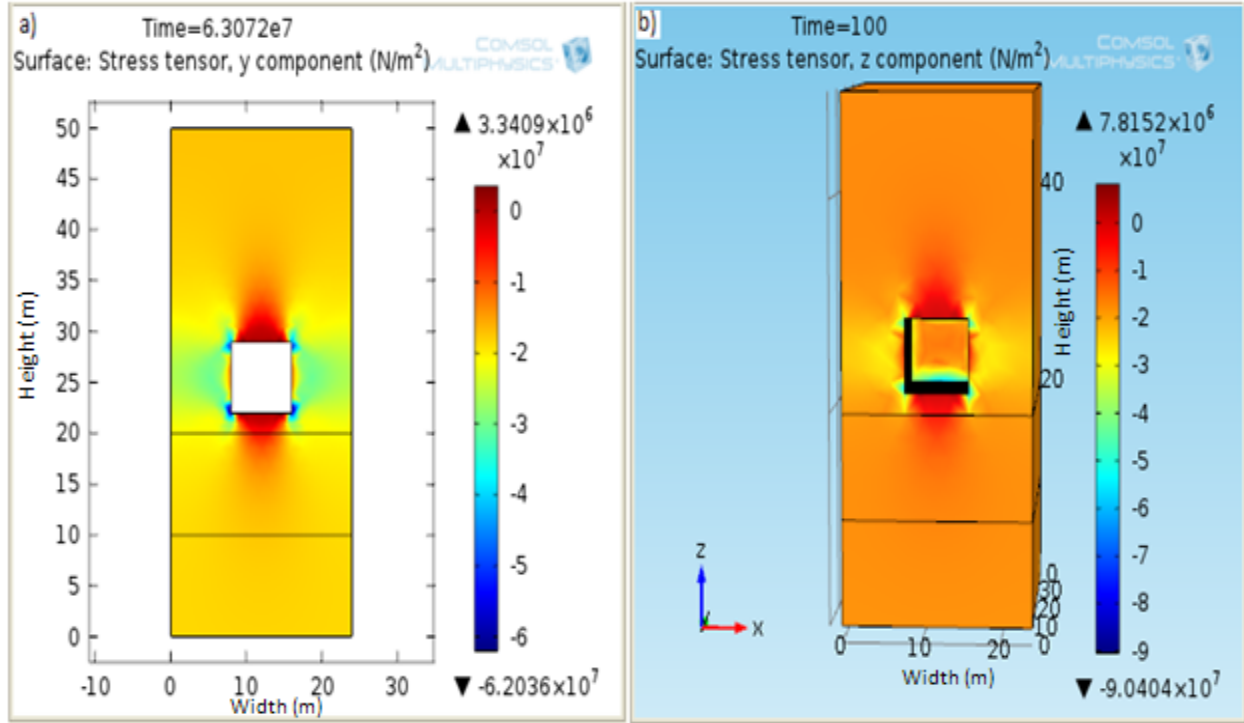


Figure 7.16. Distribution of σ_v at the end of the excavation: a) 2D and b) 3D.

The values of compressive stresses obtained from the 2D analysis are lower than the corresponding values obtained from the 3D analysis. However, the distribution of horizontal and vertical stresses is similar in both analyses.

7.5.4.2. Volumetric strains

Figure 7.17 shows the distribution of volumetric strains at the end of the excavation. Large expansive volumetric strains are developed on the right and left sides of the room. Expansive volumetric strains are also developed above and below the room and within the weak Sherman Fall layer located at 2 m beneath the room. Compressive volumetric strains are developed at the corners of the room. The values of volumetric strains obtained from the 2D analysis are lower than the corresponding values obtained from the 3D analysis. However, the distribution of volumetric strains is similar in both analyses.

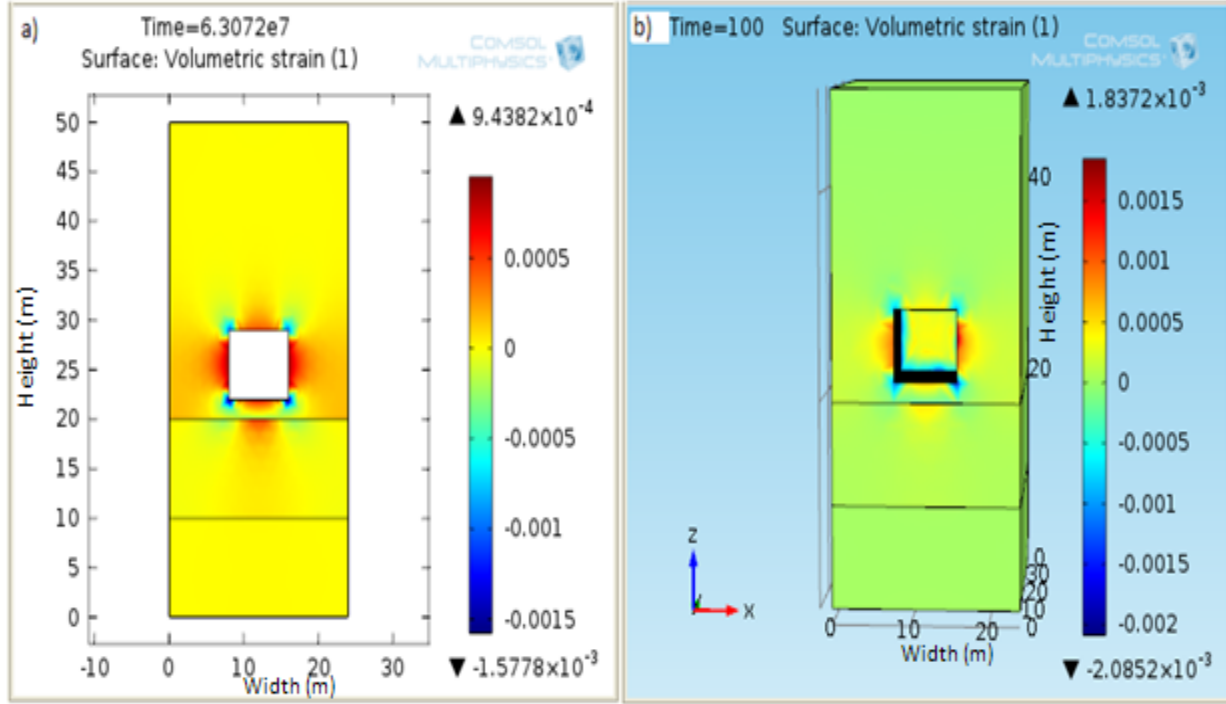


Figure 7.17. Distribution of volumetric strains at the end of the excavation: a) 2D and b) 3D.

7.5.4.3. *Effective plastic strains*

Figure 7.18 shows the distribution of effective plastic strains at the end of the excavation. For the 2D analysis, effective plastic strains are developed in limited places in the walls, roof, and base of the room. In 3D analysis, effective plastic strains are also developed in limited places in the walls, roof, base, and along the length of the room. Also, in 3D analysis the size of the regions of plastic strains depends on the excavation stages. The maximum values of the effective plastic strains are around 0.00011 and 0.0012 for the 2D and 3D, respectively.

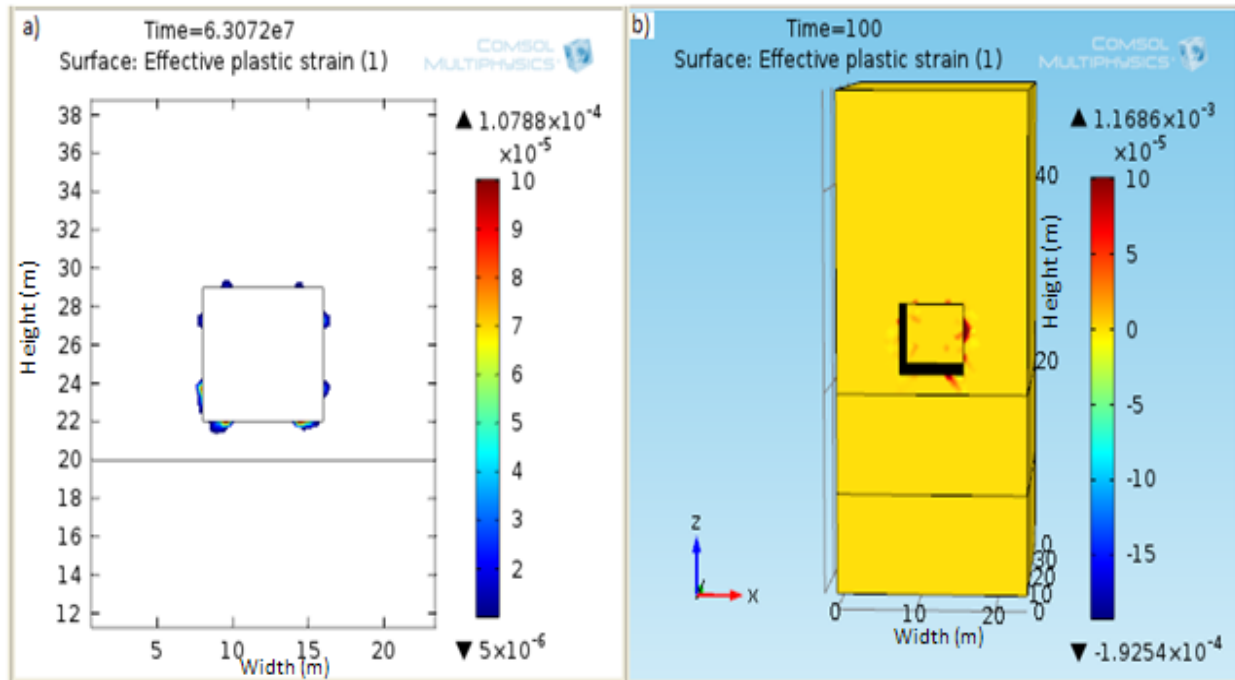


Figure 7.18. Distribution of effective plastic strains at the end of the excavation: a) 2D and b) 3D.

7.5.4.4. Displacements

Figure 7.19 shows the distribution of displacements at the end of the excavation. Maximum values of 7 mm and 6 mm are calculated in the domain for 2D and 3D, respectively. 2D analysis indicates that the roof of the room is experiencing the largest displacements. In the contrary, the 3D analysis indicates that the base and the vertical walls of the room are experiencing the largest displacements. In terms of the magnitude, the difference between the results of 2D analysis and the results of 3D analysis is insignificant. However, in terms of distribution of displacements around the emplacement room the results are quite different. For 3D analysis, the inward displacement at the crown along the length of the room is shown in Fig. 7.20. Most of the deformation produced by excavation at any point in the emplacement room takes place by the time the excavation moves 4 meters away from this point.

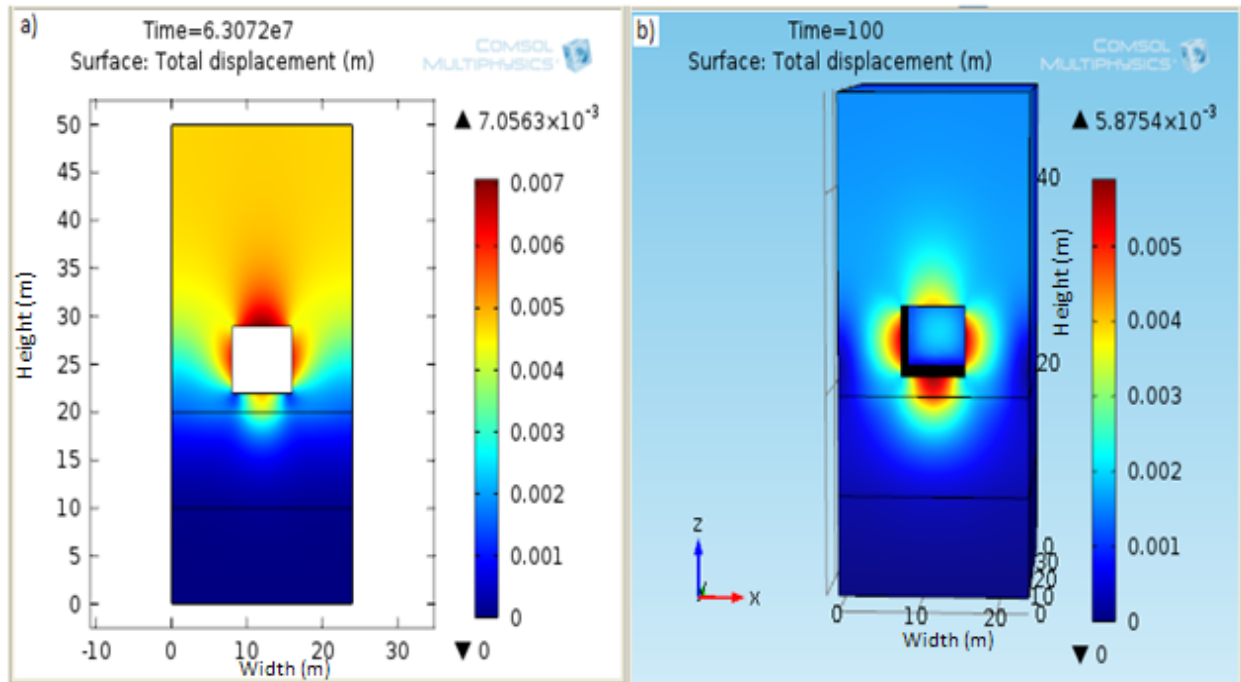


Figure 7.19. Distribution of displacements at the end of the excavation: a) 2D and b) 3D.

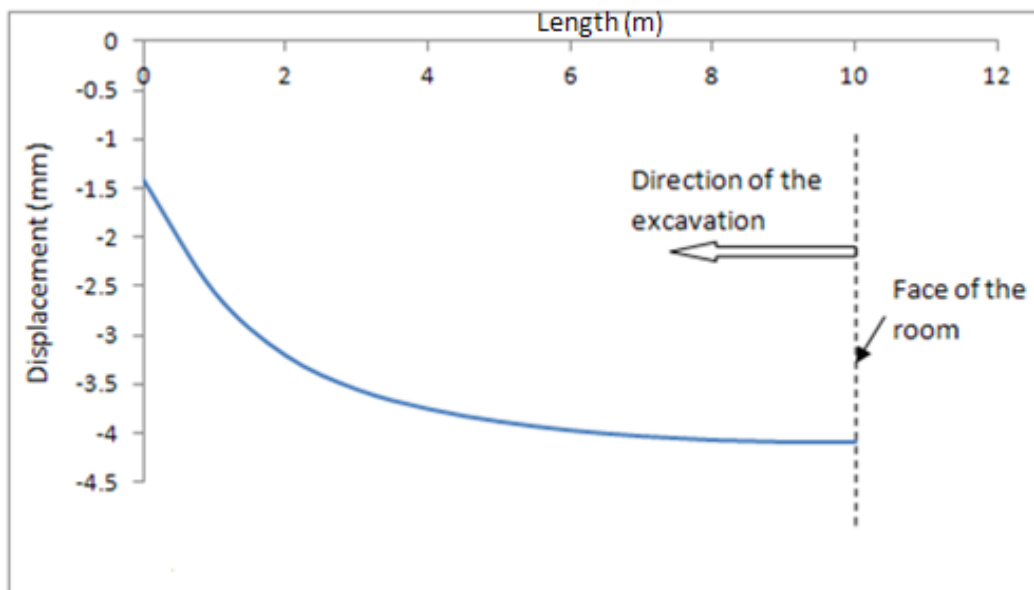


Figure 7.20. Inward displacement at the crown along the length of the room (3D analysis).

7.6. Two-dimensional numerical simulation of the hydromechanical behaviour of the limestone around an emplacement room at the Bruce site, Canada

Two-dimensional coupled hydromechanical analysis is performed for an emplacement room at the Bruce site. The parameters required for carrying such analysis are taken from the open literature (Avis et al., 2009, Gartner Lee Limited 2008, and OPG Report-2 2008). The hydrostatic pressure of the rock formation is uniformly distributed (Avis et al., 2009). The porewater pressure at the level of the repository is 5.9 MPa. At the location of the repository, the hydraulic conductivity in the horizontal direction is ten times higher than the hydraulic conductivity in the vertical direction. The effective in situ stresses are: $\sigma'_H = 2 \sigma'_V$ and $\sigma'_h = 1.2 \sigma'_V$. The moisture content of the rock mass is around 0.8%. The finite element model is described above (Fig. 7.13). The top boundary of the finite element model is subjected to a uniform vertical effective stress equivalent to the in situ vertical effective stress (13 MPa). Two failure criteria, Mohr-Coulomb (MC) and Hoek-Brown (HB), are used in the elastoplastic part and their results are compared. The effect of damage on the mechanical and hydraulic properties of the limestone is considered as described above. The excavation process is simulated by gradually reducing stresses and porewater pressures at the boundary of the room from the initial state to zero.

7.6.1. Mechanical and hydraulic properties of the rock layers

For the first analysis in which the MC failure criterion is used, in addition to the mechanical properties of the rock mass described in the Table 7.2, the hydraulic properties described in the Table 7.3 are also used. For the second analysis in which the HB failure criterion is used, in addition to the mechanical and hydraulic properties described in the Tables 7.2 and 7.3, the Hoek-Brown parameters described in the Table 7.4 are also used. The estimation of the Hoek-Brown parameters is based on laboratory testing of the rock, physical and visual inspection of the structure of the rock, and charts.

Table 7.3. Hydraulic properties of the rock units.

Type of rock	ω	k_x (m/s)	k_y (m/s)	α_B	Specific storage	ρ_f (kg/m ³)
Sherman Fall	0.02	9e-12	9e-13	0.8	1.3e-6	1100
Weak Sherman Fall	0.02	9e-12	9e-13	0.8	1.3e-6	1100
Cobourg	0.02	9.6e-12	9.6e-13	0.8	1.3e-6	1100

Table 7.4. Hoek-Brown parameters (OPG Report - 2, 2008). Some of the parameters are assumed.

Type of rock	Uniaxial compressive strength (MPa)	Geological strength index	Disturbance factor	Intact rock parameter
Sherman Fall	85	70	0	8
Weak Sherman Fall	45	65	0	8
Cobourg	130	85	0	8

7.6.2. Interpretation of the numerical results

7.6.2.1. Stress distribution

Figure 7.21 shows the distribution of horizontal effective stresses two years after the excavation. Before the excavation, the magnitude of the horizontal compressive effective stress acting at the level of excavation was about 31 MPa. The results shown in Fig. 7.21.a are obtained using MC failure criterion, and the magnitude of the maximum horizontal compressive effective stress in the whole analysis domain is 92 MPa. The results shown in Fig. 7.21.b are obtained using HB failure criterion, and the magnitude of the maximum horizontal compressive effective stress in the whole analysis domain is 90 MPa. In both cases, the maximum horizontal compressive effective stresses occur at the corners of the room. Comparing both cases, the distribution of horizontal effective stresses is similar.

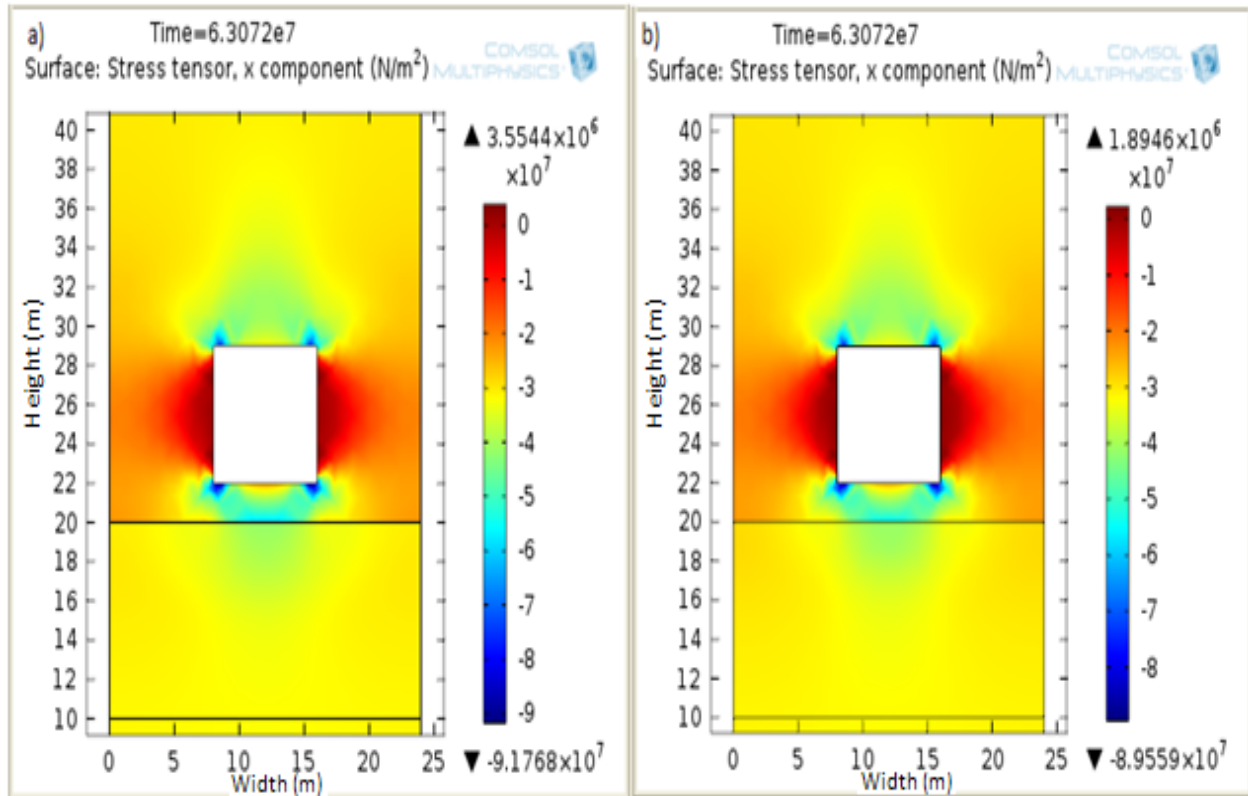


Figure 7.21. Distribution of horizontal effective stresses: a) using MC and b) using HB.

Figure 7.22 shows the distribution of vertical effective stresses two years after the excavation. Before the excavation, the magnitude of the vertical compressive effective stress acting at the level of the excavation was about 13 MPa. The results in Fig. 7.22.a are obtained using MC failure criterion, and the magnitude of the maximum horizontal compressive effective stress in the whole analysis domain is 62 MPa. The results in Fig. 7.22.b are obtained using HB failure criterion, and the magnitude of the maximum horizontal compressive effective stress in the whole analysis domain is 60 MPa. In both cases, the maximum vertical compressive effective stresses occur at the corners of the room. Comparing both cases, the distribution of vertical effective stresses is similar.

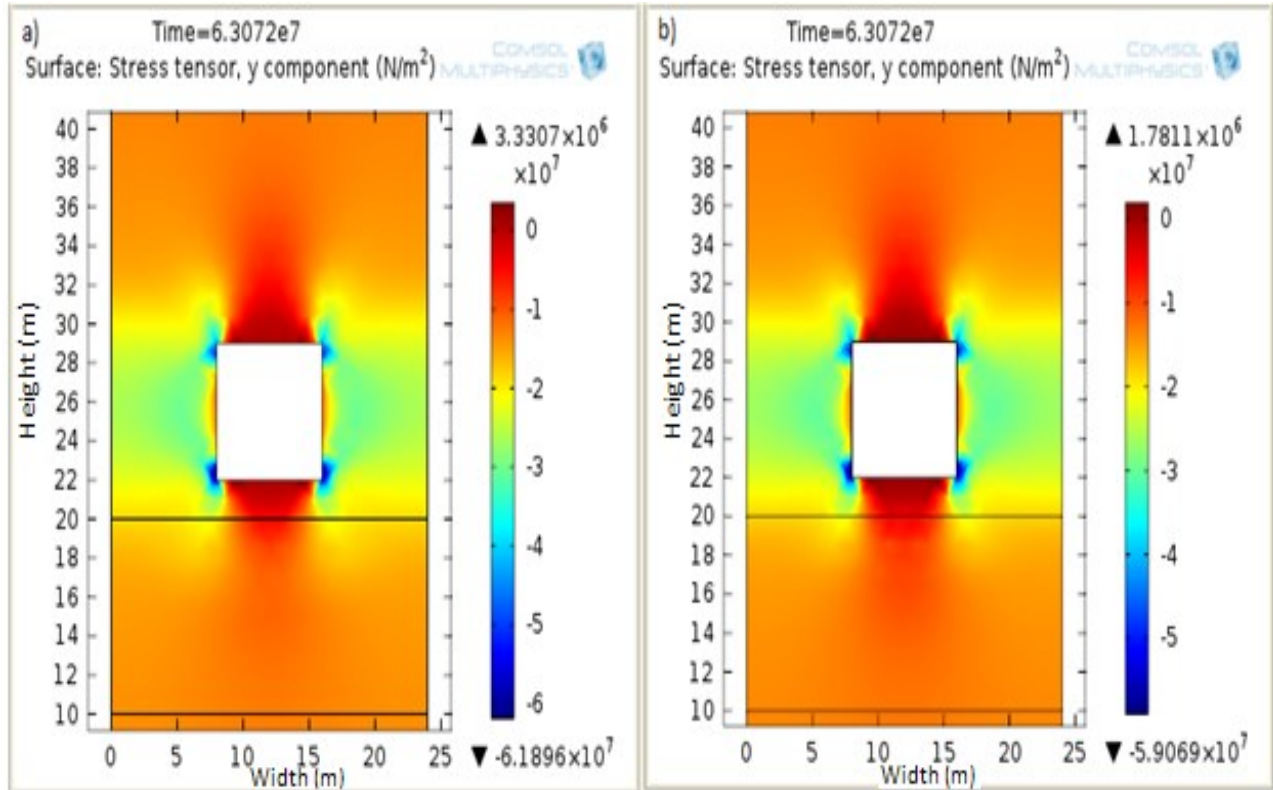


Figure 7.22. Distribution of vertical effective stresses: a) using MC and b) using HB.

7.6.2.2. Volumetric strains

Figure 7.23 shows the distribution of volumetric strains two years after the excavation. The results shown in Fig. 7.23.a and Fig. 7.23.b are obtained using MC failure criterion and HB failure criterion, respectively. Both failure criteria produced similar distribution of volumetric strain and almost the same results. The corners of the room are subjected to compression while the walls of the room are subjected to extension. Extension areas also occurred within the Weak Sherman Fall layer located 2 m below the room.

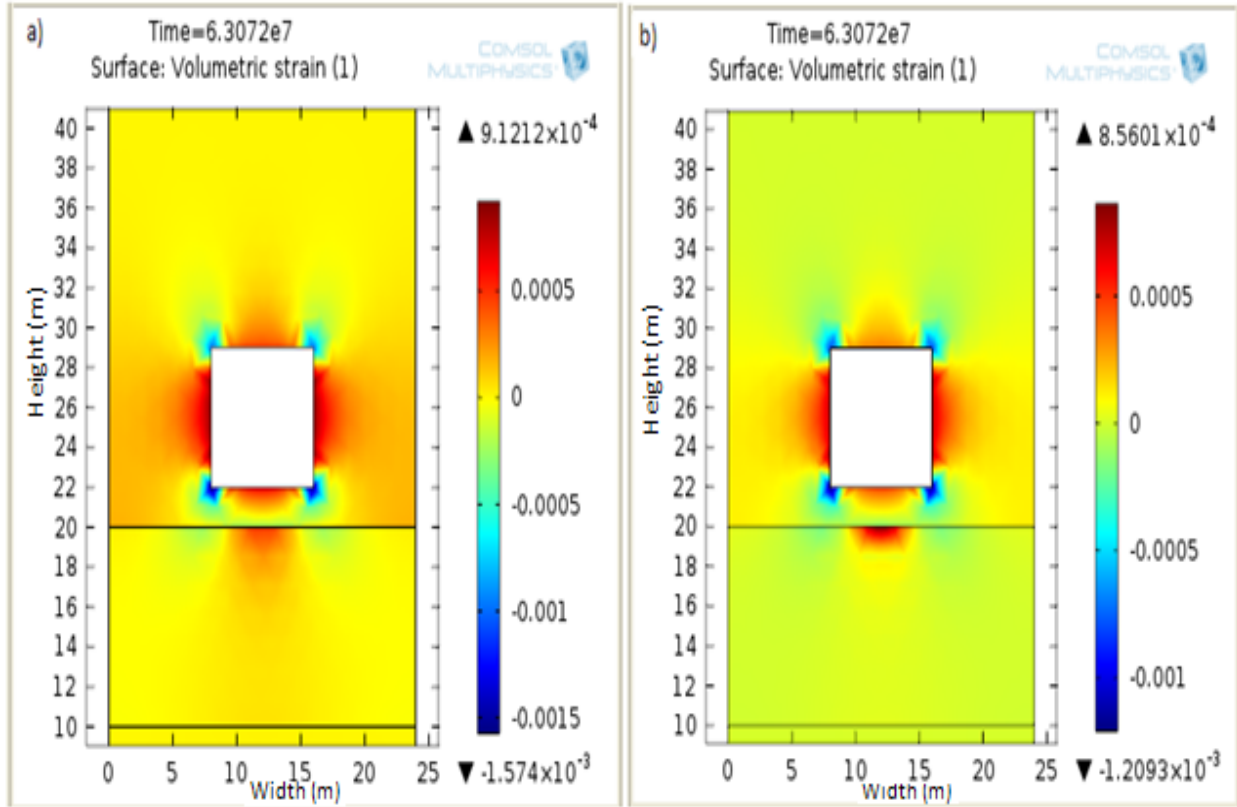


Figure 7.23. Distribution of volumetric strains: a) using MC and b) using HB.

7.6.2.3. *Effective plastic strains*

Figure 7.24 shows the distribution of effective plastic strains two years after the excavation. The results shown in Fig. 7.24.a and Fig. 7.24.b are obtained using MC failure criterion and HB failure criterion, respectively. HB failure criterion produced more effective plastic strain than the MC failure criterion. In both cases, the effective plastic strains occurred close to the corners of the room and within the Weak Sherman Fall layer located 2 m below the room.

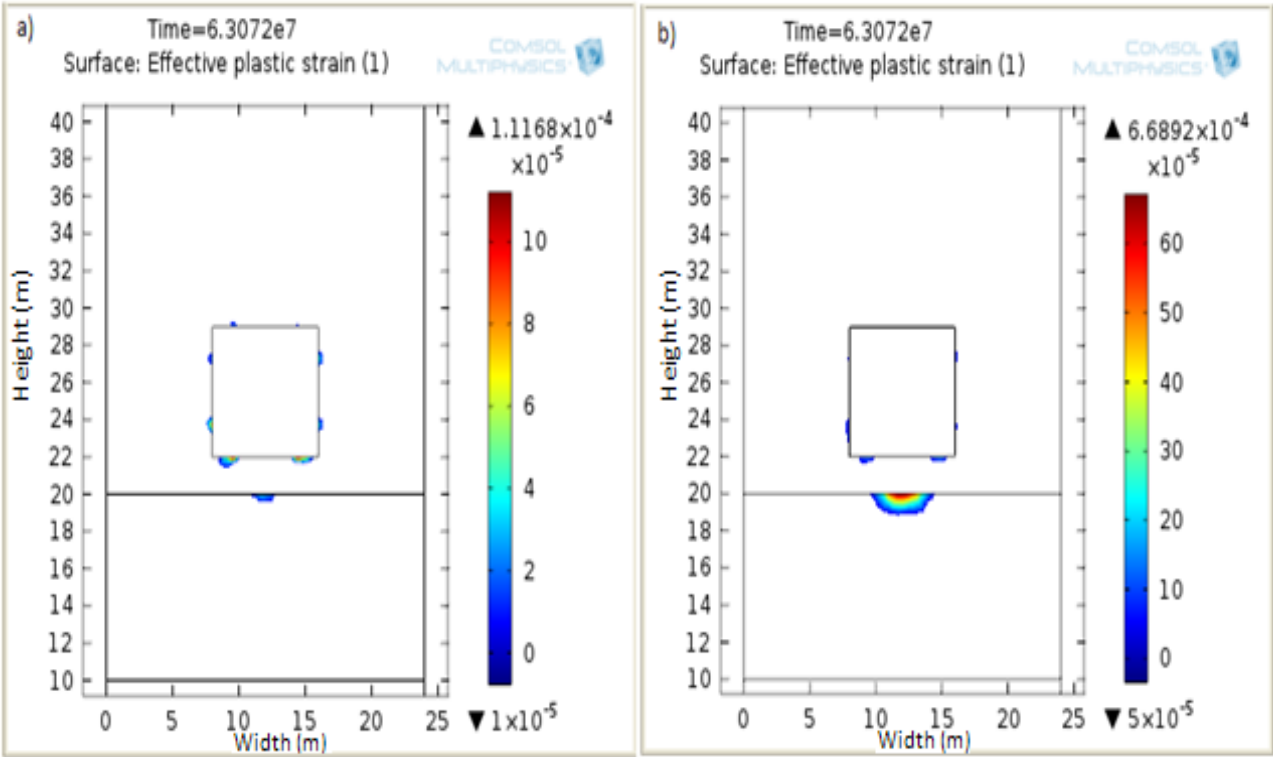


Figure 7.24. Effective plastic strains: a) using MC and b) using HB.

7.6.2.4. Displacements

Figure 7.25 shows the distribution of displacements two years after the excavation. The results shown in Fig. 7.25.a and Fig. 7.25.b are obtained using MC failure criterion and HB failure criterion, respectively. Maximum displacements of about 7 mm (Fig. 7.25.a) and 6.4 mm (Fig. 7.25.b) are calculated at the crown of the room. A displacement of 4 mm is calculated at the invert of the room. Comparing both cases, the distribution of displacements around the emplacement room is similar.

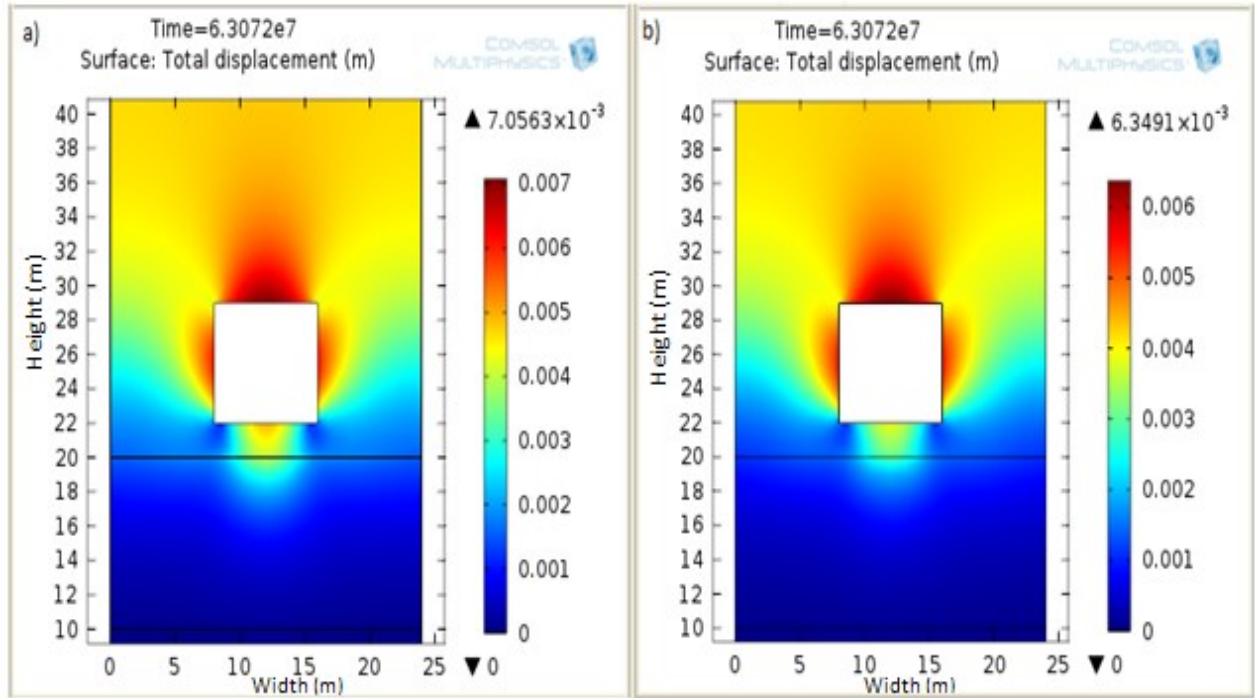


Figure 7.25. Distribution of displacements: a) using MC and b) using HB.

7.6.2.5. Porewater pressure distribution

Two cases of permeability are considered:

Case 1: The permeability of the argillite is assumed to be constant and not affected by the damage ($\psi = 0$).

Case 2: The permeability of the argillite is affected by the damage. The effect of the damage on the permeability is expressed as a function of the equivalent deviatoric strain (Equation [7.4]). Three different values of ψ (1500, 4000, and 7000) are considered in the analysis.

Figure 7.26 shows the distribution of the equivalent deviatoric strain two years after the excavation.

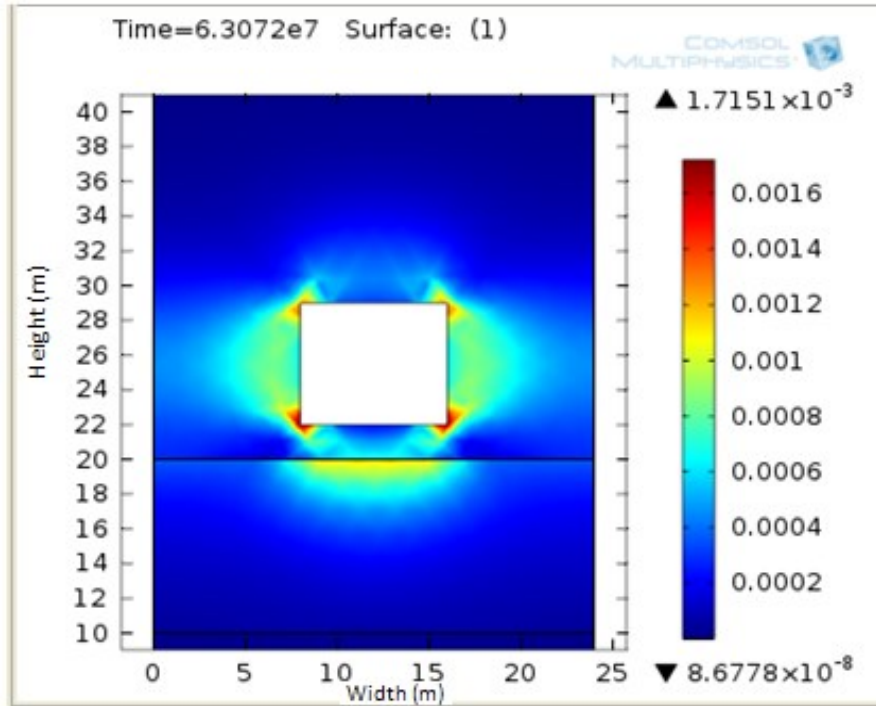


Figure 7.26. Equivalent deviatoric strain two years after the excavation.

Figure 7.27 shows porewater pressure distribution for four different values of $\psi = 0, 1500, 4000,$ and 7000 .

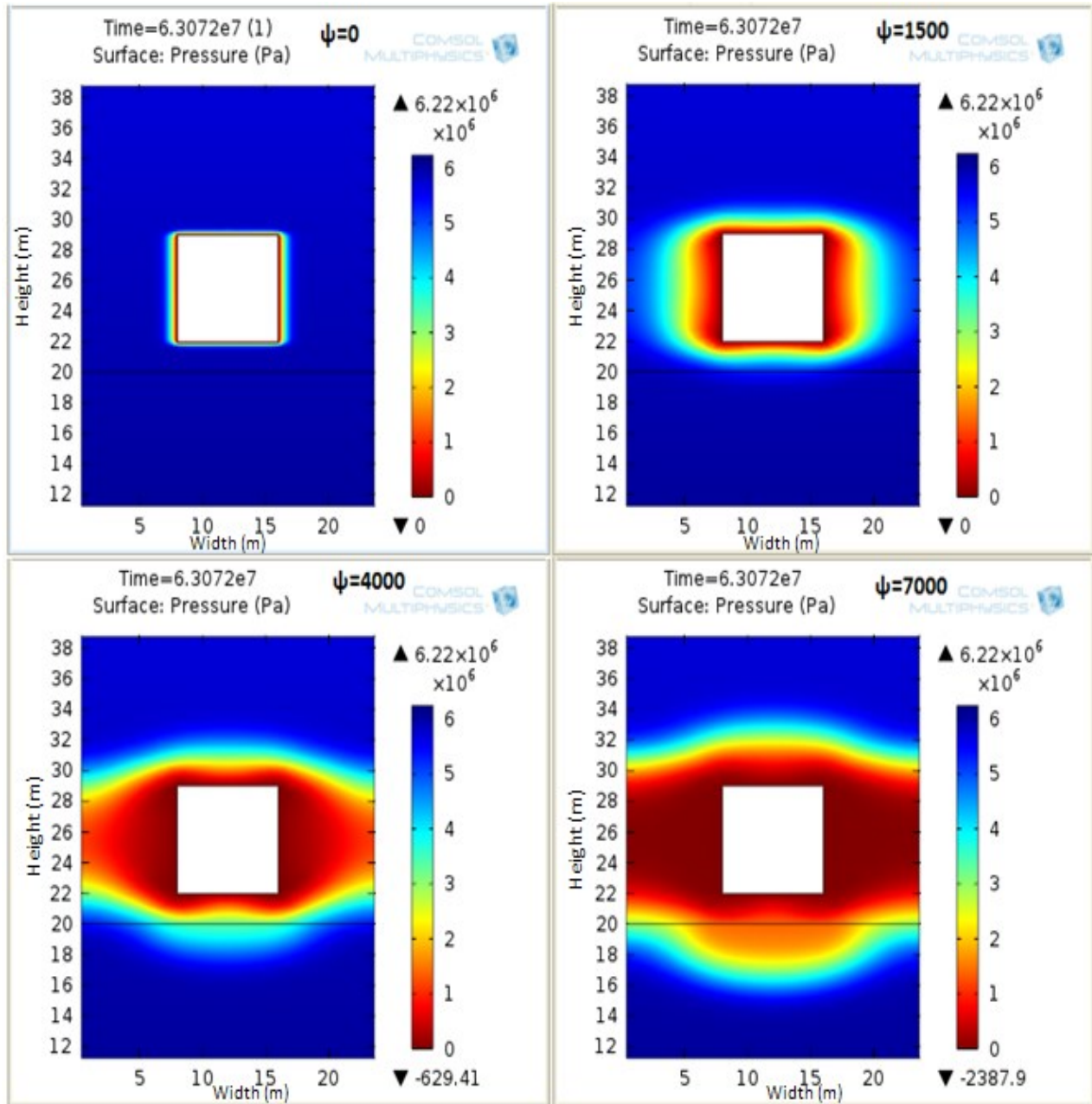


Figure 7.27. Distribution of porewater pressure two years after the excavation for four different values of ψ : 0, 1500, 4000, and 7000.

7.7. Characterization of the EDZ around the gallery and the emplacement room

In this section, the sizes of the EDZ around the Tournemire gallery and the Cobourg emplacement room are estimated. In each case, the EDZ is established based on the calculated displacements, volumetric strains, plastic strains, and porewater pressure dissipation.

Estimation of the size of the EDZ around the Tournemire gallery:

The Tournemire gallery is of semicircular shape, located at a depth 250 m below the ground level, and excavated within the Tournemire argillite. At this depth: the hydromechanical behaviour of the argillite can be considered as stress controlled; the in-situ stress field is around 4 MPa; and the porewater pressure is around 0.4 MPa. The excavation of the gallery results in redistribution of in-situ stresses and development of extension and compression areas around the gallery. The increase in stress in some locations is up to two times the stress prior to the excavation. The uniaxial compressive strength of the rock varies between 15 MPa ~ 30 MPa depends on the loading orientation angle, θ . The new stress state resulted in almost no yielding of the material. Only a negligible plastic strain, in the order of 10^{-5} , is calculated at the corners of the gallery and it could be the result of the movement of the material at the sharp corners of the gallery. A maximum displacement of 1.6 mm is calculated at the crown of the gallery.

Based on the numerical results shown in Figs 7.4, 7.5, 7.6, 7.7, 7.10, and 7.11, the HDZ, edz, and Edz are estimated for the gallery as shown in Fig. 7.28. Displacements, microcracks, and yielding of the rock are developed within the HDZ. Also, complete porewater pressure dissipation is developed within the HDZ. Elastic displacements and full/partial porewater pressure dissipation is developed within the edz. Possible hydromechanical modifications can develop within the Edz.

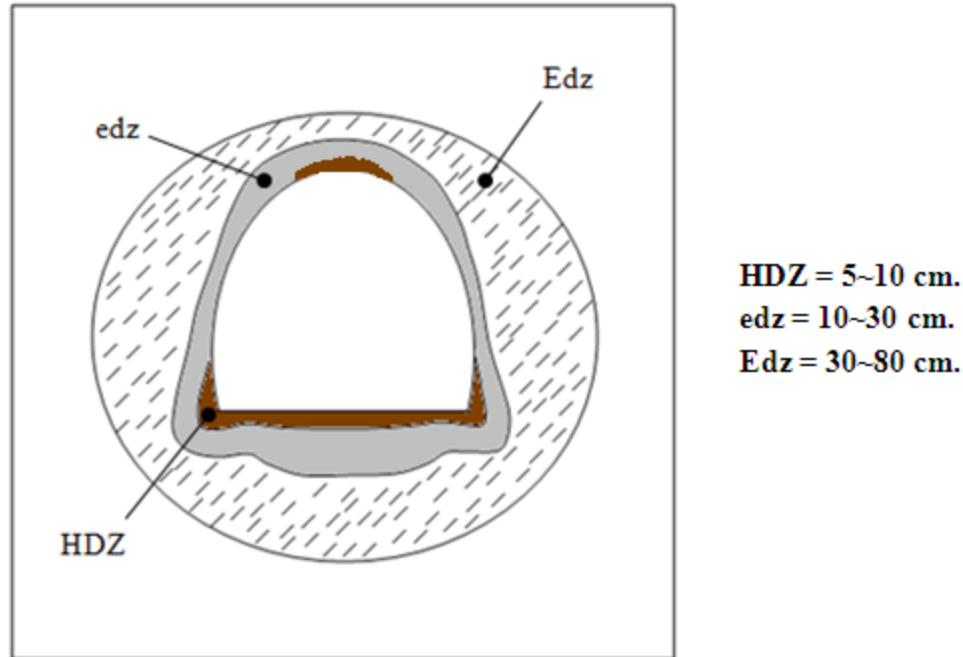


Figure 7.28. The HDZ, edz, and Edz for the Tournemire gallery.

Estimation of the size of the EDZ around the Cobourg emplacement room:

The emplacement room is of rectangular shape (8 m x 7 m) and located at a depth 683 m. At this depth: the hydromechanical behaviour of the limestone can be considered as stress controlled and the in-situ stress is anisotropic: $\sigma_v = 18$ MPa, $\sigma_H = 36$ MPa, and $\sigma_h = 21.6$ MPa. The excavation of the room results in redistribution of stresses and development of extension and compression areas around the emplacement room. The increase in stress in some locations around the emplacement room is up to four times the stress prior to the excavation. At these locations, the stresses approaches/surpasses the peak strength of the rock layers: Cobourg limestone (92 MPa), Weak Sherman Fall (30 MPa), and Sherman Fall (60 MPa). As a result, damage and yielding of the material occurred within the vicinity of the emplacement room and within the Weak Sherman Fall layer located 2 m below the room.

Based on the numerical results shown in Figs 7.21, 7.22, 7.23, 7.24, 7.25, 7.26, and 7.27, the HDZ, edz, and Edz are estimated for the room as shown in Fig. 7.29. Large displacements,

microcracks, yielding of the limestone, and complete porewater pressure dissipation are developed within the HDZ around the emplacement room. Yielding of the rock is developed within the HDZ in the Weak Sherman Fall layer. Elastic displacements and full/partial porewater pressure dissipation are developed within the edz around the emplacement room. Elastic displacements are developed within the edz in the Weak Sherman Fall layer. The Edz can experience hydromechanical perturbations. The area below the room could suffer from large displacements and microcracking (unstable).

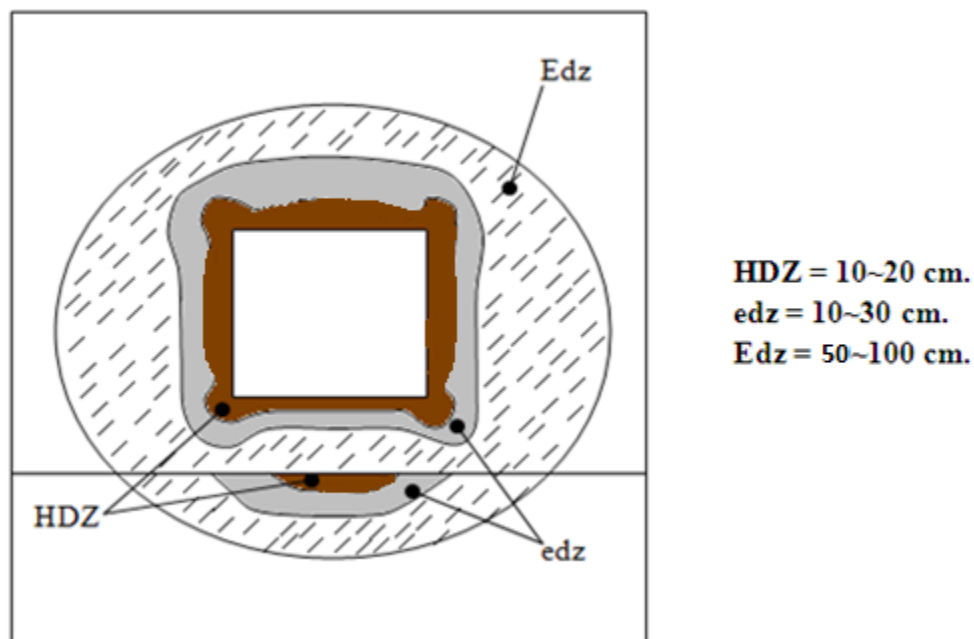


Figure 7.29. The HDZ, edz, and Edz for the Cobourg emplacement room.

Sedimentary rocks exhibit creep and the damage affect their mechanical and hydraulic properties (Chapter 4 and Chapter 5). Strength degradation, microcracking, and hydraulic loads result in the softening of the rock around the openings and can cause an increase in the EDZ. The long-term evolution of the EDZ with time depends also on geochemical changes, thermal effects, development of wet/dry cycles, and gas-pressure build-up.

7.8. Summary of the numerical results

- Underground excavations result in the development of compression/extension areas around the openings due to redistribution of in-situ stresses, porewater pressure dissipation/generation around the openings, and possible perturbation into the chemistry of the rock mass. The EDZ consist of both compression and extension areas.
- The initial size of the EDZ depend mainly on the state of stress, geometry and orientation of the opening, hydraulic and mechanical properties of the rock, excavation method, mineralogical composition of the rock, and geological structure of the rock mass.
- Damage within the EDZ consists of yielding, microcracking, fracturing, and rock fall. The damage can affect the mechanical and hydraulic properties of the rock.
- The EDZ can be divided into three subzones: Highly damaged subzone (HDZ), Excavation damaged subzone (edz), and Excavation disturbed subzone (EdZ).
- For clay-rich rocks such as sedimentary rocks, the EDZ evolves over time depending on the new stress state, strength degradation of the rock, hydraulic loads, thermal stresses, geochemical changes, development of wet/dry cycles, and gas-pressure build-up.
- The long-term dissipation of porewater pressure around the opening depends mainly on the hydraulic conductivity of the rock, state of stress, and the amount of damage. It can be divided into three subzones: full porewater pressure dissipation subzone, partial porewater pressure dissipation subzone, and no porewater pressure dissipation subzone.
- For the mine-by-test experiment, the elastoplastic behaviour of the rock is dominated by the elastic part and only a very small EDZ is predicted following excavation.
- For the Cobourg emplacement room, large EDZ is predicted following excavation.
- Two different failure criteria, Mohr-Coulomb and Hoek-Brown, produce similar distributions of stresses, strains, and displacements.
- 2D analysis predicted slightly lower values of stresses and strains than 3D analysis.

CHAPTER 8

CONCLUSIONS AND RECOMMENDATIONS

8.1. Conclusions

This work concerns laboratory investigation of the mechanical behaviour of the anisotropic Tournemire argillite, modeling of the mechanical behaviour of the Tournemire argillite, and numerical simulations of the mechanical and hydromechanical behaviour of two host sedimentary rocks, namely the Tournemire argillite and the Cobourg limestone, for deep geological repository for nuclear wastes.

An experimental database of the mechanical properties of the Tournemire argillite has been generated. It was found that the argillite is anisotropic in both strength and deformation, very sensitive to the change in moisture content, and it behaves as a brittle material. The mechanical behaviour of the argillite is highly nonlinear and the material shows hardening behaviour and then softening behaviour. Within the Tournemire URL, the strength and deformation properties of the argillite could be different at different locations.

The stress-strain relationship for the Tournemire argillite can be divided into four zones: microcrack/bedding closure zone, elastoplastic hardening zone, an elastic zone which exists only during unloading-reloading cycles, and post-peak zone. The compressibility and shear behaviour of bedding planes play a significant role in the deformation behaviour of the argillite. Key aspects of the observed behaviour are described: elastic deformation, plastic deformation, cyclic behaviour, creep behaviour, transition from contraction to dilation of volumetric deformation, damage due to plastic deformation and microcracking, microcracking behaviour, and failure behaviour.

The inelastic behaviour of the Tournemire argillite starts right at the beginning of the stress-strain relationship. The elastic parameters of the argillite are estimated from the material

behaviour during the unloading-reloading cycles of the experiments. The elastic parameters are influenced by the loading orientation angle and the number of cycles.

The compressive strength is highly sensitive to the shear strength characteristics of bedding planes. The peak strength and strength measured in post peak region of the Tournemire argillite depend on the loading orientation angle and confining pressure. For the range of confining pressure considered, the maximum peak strength is measured in tests performed at $\theta=0^\circ$. The minimum peak strength is obtained in tests performed at $30^\circ \leq \theta \leq 45^\circ$. The tensile strength depends on the loading orientation angle. The peak strength and tensile strength of dry specimens are three times higher than the corresponding strength obtained from specimens at their natural water content. The microstructure approach can be used to estimate the peak strength of the rock for any loading orientation angle and at any confining pressure.

Cyclic behaviour of the Tournemire argillite depends on whether the cyclic loading-unloading is performed before or after the peak strength. Before the peak strength, plastic strains are measured at each load cycle. The elastic modulus degrades and sample contraction increase with additional load cycles. The cyclic behaviour of the argillite after the peak strength is controlled by the developed fractures. Small hysteresis was observed in cyclic tests, and the hysteresis phenomena are the result of viscous properties of the argillite. Compared to uniaxial tests, cyclic loading had no significant effect on the strength of the argillite and the failure mode of the argillite.

The development of axial and volumetric strains in the Tournemire argillite depends on the loading orientation angle and confining pressure. Reducing the deviatoric stress in cyclic tests (unloading) provides the necessary data to calculate the elastic and plastic strains. Plastic strains were between two-thirds ($2/3$) and one-half ($1/2$) of the total strain. The volumetric strain change is compressive up to the peak strength, after which dilation rapidly develops.

For the Tournemire argillite, considering only the two principal material axes, $\theta=0^\circ$ and $\theta=90^\circ$, the strength anisotropy parameter is small and close to unity. Considering all the orientations, the strength anisotropy parameter is close to two and it becomes smaller for higher values of confining pressure. The measurements of the travel time of P-waves result in a coefficient of anisotropy of about 2.16.

The failure of the Tournemire argillite occurs in three principal modes: extension (splitting), shearing, and a combination of the extension and shearing. The failure mode is a function of the loading orientation angle. The range of confining pressures considered in this investigation has little effect on the failure mode.

The stress level at which the new microcracks start to develop within a rock mass under mechanical loads depends on the mineralogical composition of the rock. The Tournemire argillite has high clay content, 55%, which makes the argillite behave in a less brittle manner than hard rocks. According to the recorded acoustic emission data, the new microcracks start to develop within the argillite at a stress level of about $\sigma_{ci} = 70\% \sim 75\%$ of the peak strength and the crack damage stress level is reached approximately at a stress level of $\sigma_{cd} = 85\% \sim 90\%$ of the peak strength. For hard rock such as granite: $\sigma_{ci} = 40\%$ of the peak strength and $\sigma_{cd} = 80\%$ of the peak strength (Martin, 1993).

Creep tests indicate that the Tournemire argillite exhibits creep. If the stress level is less than the crack damage stress level (σ_{cd}), the effect of the creep is minimal. However, when the stresses exceed the σ_{cd} , the creep can cause the failure of the rock. This behaviour could be due to unstable crack growth under constant stress. Strength degradation, microcracking, and hydraulic loads can accelerate the creep rate.

Based on the acoustic emission recorded data, a concept describing the mobilization of strength parameters during loading the Tournemire argillite has been proposed and validated (concept of mobilized strength parameters). The mobilized cohesion of the argillite controls the location of the yield surface at low stress levels. At high stress levels, the mobilized friction of the argillite has more effect on the location of the yield surface.

An elastoplastic-damage model has been developed for describing the mechanical behaviour of the anisotropic Tournemire argillite. The model consists of four components: elastic properties of the argillite, a damage model, the concept of mobilized strength parameters, and classical theory of elastoplasticity. The capabilities of the model have been evaluated by simulating laboratory experiments.

Underground excavations result in the development of compression and extension areas around tunnels due to stress-redistribution, porewater pressure dissipation/generation around tunnels,

and perturbation into the chemistry of the rock mass. The size and distribution of the compression and extension areas around tunnels depend mainly on the state of stress, geometry and orientation of the tunnel, excavation method, hydraulic and mechanical properties of the rock, mineralogical composition of the rock, and geological structure of the rock mass. The EDZ consists of both compression and extension areas. The evolution of the EDZ with time depends mainly on the new stress state, strength degradation of the rock, hydraulic loads, thermal stresses, geochemical changes, development of wet/dry cycles, and gas-pressure build-up.

The EDZ can be divided into three subzones: Highly damaged subzone (HDZ), Excavation damaged subzone (edz), and Excavation disturbed subzone (EdZ). The initiation and evolution of the EDZ can be structured into three phases: the initial phase (development of the DGR), the transient phase (operational phase of the DGR), and the long-term phase (closure of the DGR).

The long-term dissipation of porewater pressure around tunnels depends mainly on the hydraulic conductivity of the rock, state of stress, and the amount of damage. It can be divided into three subzones: full porewater pressure dissipation subzone, partial porewater pressure dissipation subzone, and no porewater pressure dissipation subzone. Porewater pressure equilibration in the vicinity of the DGR may take many years after the excavation.

The numerical simulation of a mine-by-test experiment indicates that numerical approach can reproduce field measurements. 2D analysis predicted slightly lower values of stresses and strains than 3D analysis. Different failure criteria produce similar distributions of stresses, strains, and displacements after the excavations.

The damage affects the mechanical properties of argillaceous rocks and the effective plastic strain can be used as a measure of damage. The damage also affects the permeability of argillaceous rocks and the equivalent deviatoric strain can be used as a measure of damage.

The EDZ is established for two DGRs: the Tournemire gallery and the Cobourg emplacement room. Two forms of damage can occur within the EDZ. The compression areas may suffer from deformations, yielding, microcracking, and fracturing. The extension areas may suffer from large deformations, microcracking, fracturing, and rock falls. The external loads such as earthquakes and glaciations will cause more damage to the rock.

The experimental and numerical results obtained in this research program suggest that sedimentary rocks characterize with acceptable strength and deformation properties and they are more resistant to cracking than hard rock. The results of this research can help in designing a DGR in Canada and elsewhere. They also can help in estimating the EDZ in anisotropic sedimentary rocks. The elastoplastic-damage model can help in understanding the mechanical behaviour of anisotropic sedimentary rocks.

8.2. Recommendations for future studies

The experimental and numerical results obtained in this research program and the data from open literature indicate that sedimentary rocks could be good candidates for hosting deep geological repositories for the storage of nuclear waste. The initial size of the EDZ depends mainly on the state of stress, geometry and orientation of the tunnel, excavation method, hydraulic and mechanical properties of the rock mass, mineralogical composition of the rock, and geological structure of the rock mass. The evolution of the EDZ over time depends mainly on the new stress state, strength degradation of the rock, development of wet/dry cycles, hydraulic loads, thermal stresses, chemical reactions, gas-pressure build-up, and geological structure of the rock mass. The prediction of the long-term performance of DGRs in sedimentary rocks requires the consideration of all parameters that can affect their performances. Thus; more field and laboratory tests, more modeling development, numerical simulations of all coupled THMC processes, and consideration of external loads are required.

The following further studies are suggested:

1. Field and laboratory testing:

- Strength degradation of the rock plays a big role in the evolution of the EDZ over time. Field and laboratory tests should be carried out to investigate the strength degradation of the rock.

- Field and laboratory tests should be carried out to investigate the shrinkage of the rock during drying, the effect of repeated wetting/drying cycles, swelling, desiccation, thermal effect, and self-healing of microcracks and fractures.
- Field and laboratory tests should be carried out to investigate the effect of the initiation and propagation of microcracks and fractures on the permeability of the rock.
- The measurements of pore pressure around excavations are important for carrying unsaturated hydro-mechanical analyses.

2. Model development:

- The analysis of the mechanical behaviour of anisotropic rocks is complex and requires the use of an anisotropic failure criterion and an anisotropic flow rule. A couple of attempts have already been made in this direction; however, so far there is no valid failure criterion that can be used. The research should be encouraged in order to develop a well tested and valid anisotropic failure criterion for sedimentary rocks.
- The elastoplastic-damage model described in Chapter 6 does not capture all the details. It can be improved by adding to it a damage model to account for the effect of cycling.

3. Numerical simulations:

- After the closure of the DGR, a heat source should be added and a Thermal-Hydraulic-Mechanical (THM) analysis should be carried out for a DGR.
- Chemical reactions that can result during different phases of the DGR (development phase, operational phase, and closure phase) should be analyzed and a complete Thermal-Hydraulic-Mechanical-Chemical (THMC) analysis should be carried out for a DGR.

4. External loads:

- Earthquakes are unpredictable and they can occur in any place. The effect of earthquakes on the performance of DGRs should be considered in the analysis.
- The most long-lived radioactive wastes must be contained and isolated from humans and the environment for millions of years. The glaciations events can occur in the future and the effect of external loads that can result from such event on the performance of DGRs should be considered.

CHAPTER 9

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APPENDICES

APPENDIX A: MTS Rock Mechanics Testing System (Model-815)

A-1. MTS Rock Mechanics Testing System, Model 815:

CANMET-MMSL owns and operates a computer controlled, servo-hydraulic MTS Rock Mechanics Testing System, Model 815. The system comprises of four independent modules for axial loading, triaxial confinement, permeability and elevated temperature testing, as well as a control and data acquisition module, to control the hydraulic and pore pressures and proceed to data acquisition. A picture of the testing system is shown in Fig. A-1. The specifications of the test system are summarized in the Table A-1.



Figure A-1. MTS triaxial apparatus (Model 815).

Table A-1. Specification of CANMET-MMSL Test System.

System Designation : MTS Rock Mechanics Testing System, Model 815		
Description	Capacity	Tolerance
Uniaxial Unit : Model 315 Load Frame Assembly		
Rated capacity	4600 kN	-
Stiffness	11000 MN/m	-
Maximum Stroke	50 mm	1.0%
Testing scale	500, 2600, 4600 kN	1.0%
Triaxial Unit : Model 286 Confining Pressure Intensifier		
Rated Capacity	140 MPa	-
Testing scale	140 MPa	1.0%
Permeability Unit : Model 286 Pore Pressure Intensifier		
Rated Capacity	140 MPa	-
Testing scale	140 MPa	1.0%
Temperature Unit : High-temperature Control Package		
Range (for 24 hours)	Ambient to 200°C	1.0%
Other measuring sensors		
Linear transducers	10 mm	1.0%
Chain extensometer	4, 8, 15 mm	1.0%
Pressure transducers	51.7 MPa	1.0%
Proportional valve	2.0 MPa	5.0%
Control Module : MTS FlexTest GT Controller / Signal Conditioner		
Resolution	24 bit	-
Sampling rate	49.15 kHz	-
Data Acquisition Module : MTS Multi-Purpose Test (MPT) Software		
Resolution	16 bit	-
Sampling rate	Up to 6 kHz	-

A-2. Uniaxial tests, triaxial tests, unconfined and confined cyclic tests, and creep tests

The tests were carried out using the MTS. Three linear variable differential transducers (LVDTs) were displayed at 120 degree intervals around the test specimen to measure axial displacements. An MTS chain-extensometer was used to measure the circumferential displacement of the

specimen during compression tests. Displacements were converted into strains. The specimens were covered by a thin Teflon heat shrink jacket prior the beginning of the test, to isolate the specimen and prevent the intrusion of hydraulic oil in the case of triaxial tests, and to protect the instrumentation and avoid the projection of fragments in the case of uniaxial tests. Test specimens were loaded at constant axial displacement rate (0.03 mm/min). For confined tests, specimens were confined at a constant stress rate of 0.1 MPa/sec, up to the confinement level specified. All data, e.g. time, axial load, confinement level, deviatoric and axial stresses, and axial and circumferential displacements, were sampled and recorded every 0.25 second upon testing, and converted into engineering units afterwards, e.g. conversion of loads and displacements into stresses and strains, respectively. Photographs of broken test specimens were taken at the end of testing. The failure modes were inspected.

A-3. Brazilian Tensile Strength Tests

Specimens with the same diameter (as in triaxial tests) and a nominal length of 40 mm were prepared to carry out splitting tensile (Brazilian) tests. MTS machine is used to perform the tests. Test specimens were loaded diametrically with a loading rate of 3.4 kN/min. Tests were carried out in conformity with the requirements of the ASTM D3967 standard test procedure for indirect (Brazilian) tensile tests (ASTM, 2012d). Wood inserts, e.g. 3-mm thick Okoume plywood bands, were used systematically with specimens to avoid any crushing at the contact of the test specimen with loading platens. This procedure is also consistent with the requirements of the ASTM C496 standard test procedure for concrete materials. Broken specimens were inspected at the end of the test program. All test data, i.e. time, diametrical load, tensile stress and vertical displacement (Ram LVDT), were recorded every 0.25 second over the whole test duration. Failure modes observed on failed specimens were identified. Specimens were photographed.

APPENDIX B: COMSOL Multiphysics

COMSOL Multiphysics is a general-purpose software platform, based on advanced numerical methods, for modeling and simulating physics-based problems. It is a modeling package for the simulation of any physical process that can be described with partial differential equations (PDEs). In addition to conventional physics-based user interfaces, COMSOL Multiphysics also allows for entering coupled systems of PDEs. Several add-on products (COMSOL Modules) are available. Application-specific modules bring terminology, material libraries, solvers and elements, as well as visualization tools appropriately specialized to the application area. Here, the 2D and 3D mechanical behaviour of the Cobourg limestone is analyzed using the Structural Mechanics Module. The hydromechanical behaviour of the Tournemire argillite and Cobourg limestone are analyzed using the Geomechanics modulus.

B-1. Structural Mechanics Module:

Structural Mechanics Module is dedicated to the analysis of mechanical structures that are subject to static or dynamic loads. It can be used for a wide range of analysis types, including stationary, transient, modal, parametric, quasi-static, frequency-response, buckling, and prestressed. The Structural Mechanics Module comes with standard nonlinear material models that describe plasticity: Mohr-Coulomb, Hoek-Brown, Cam-Clay, Otoe, etc. Sedimentary rocks can be treated as porous materials and their deformation behaviour can be analyzed using the Structural Mechanics Module.

Governing equations

Conservation of Momentum:

$$\nabla \cdot \sigma + F = 0 \quad [B.1]$$

where σ = stress tensor, F = Body force vector, and ∇ is the Laplace operator.

B-2. Geomechanics Modulus:

As an add-on to the Structural Mechanics Module, the Geomechanics Module allows the analyses of geotechnical applications, such as tunnels, excavations, slope stability, and retaining structures. Here, it is used to analyze the hydromechanical behaviour of the Cobourg limestone and Tournemire argillite. Both theories, poroelasticity theory and plasticity theory, are combined and used to analyze the coupled rock deformation and fluid flow.

Governing equations

Conservation of Mass:

$$\left[C_s(1 - \omega) + C_f \omega \right] + \frac{\partial p}{\partial t} + \nabla \cdot \left[- \left(\frac{K_s}{\mu} \right) (\nabla p + \rho_f g \nabla D) \right] = -\alpha_b \frac{\partial \varepsilon_{vol}}{\partial t} \quad [\text{B.2}]$$

Conservation of Momentum:

$$\nabla \cdot \sigma + F = 0 \quad [\text{B.3}]$$

where C_s & C_f = compressibility of solid and fluid; ω = porosity; K_s = permeability; μ = viscosity; ρ_f = density of the fluid; g = acceleration of gravity; ∇D = unit vector in the direction of g ; p = fluid pressure; σ = stress tensor; F = Body force vector; α_b = Biot-Willis coefficient (it is assumed to be 0.8 for the limestone); ε_{vol} = volumetric strain.

APPENDIX C: P-waves technique

P-waves are a type of elastic wave that can travel through a medium (Fig. C-1). The medium can be made up of gases, liquids, or solids. P-wave has the highest velocity and is the first to be recorded. P-wave is a body wave and its velocity in the direction of the propagation can be estimated as follows.

$$V_p = \sqrt{\frac{b + \frac{4}{3}s}{\rho}} \quad [C.1]$$

where b is the bulk modulus of the material, s is the shear modulus of the material, and ρ is the density of the material.

By examining the measurements of the time arrival of the wave in different planes, the microstructure of the rock and the anisotropy of the rock can be established. The degree of anisotropy is an indication of the fabric of the rock in different directions. The anisotropy can be the result of sedimentary fabric, cracks, pores, crystallographic preferred orientation, and/or lattice preferred orientation.

The anisotropic characteristics of P-wave velocities of anisotropic rocks have been investigated. Relationships have been established between the P-wave velocity and the properties of anisotropic rocks (porosity, cementation, effective stress, fluid saturation).

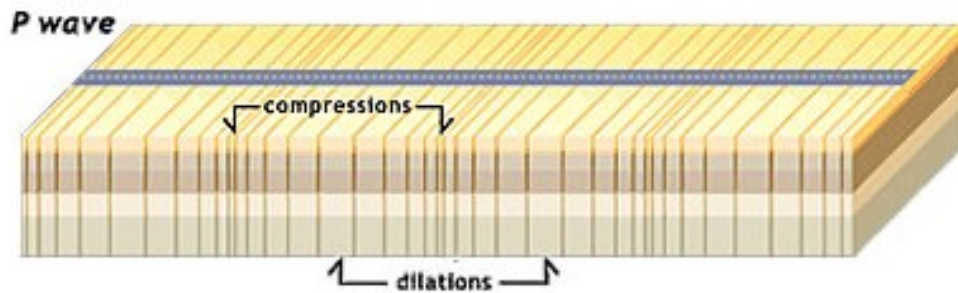


Figure C-1. P-wave propagation throughout the medium (<http://www.sms-tsunami-warning.com/pages/seismic-waves>).

APPENDIX D: Acoustic Emission

Acoustic emission is an elastic stress wave generated by the rapid release of energy within a material. Most materials emit energy in the form of mechanical vibrations (acoustic emission) as a result of sudden change such as plastic deformation, crack initiation, and crack growth. These acoustic emissions propagate from the source, throughout the material. The technique of electronically “listening” to these acoustic emissions is a world-recognized technique and used for detecting and locating defects as they occur, across the entire monitored area (Fig. D-1). This technology can be used to examine the behaviour of sedimentary rocks deforming under stress. Acoustic Emission is incorporated into many ASTM standards.

Here, acoustic emissions were recorded during uniaxial compression tests. The system used was a Pocket AE-2™ system manufactured by Physical Acoustics Corporation. This system is a dual channel 18-bit system connected to Pico™ sensors with a frequency range of 200-750 kHz and a sampling rate of 20Ms/sec. Two sensors were fixed on opposite sides of uniaxial test specimens, at 180 degrees from each other, one sensor being fixed at 2-cm above specimen mid-section and the second sensor, 2-cm under the mid-section, so these were not interfering with the movement of the MTS chain-extensometer. Acoustic emission data were recorded systematically. Main features recorded were the number of hits and the waveform signal.

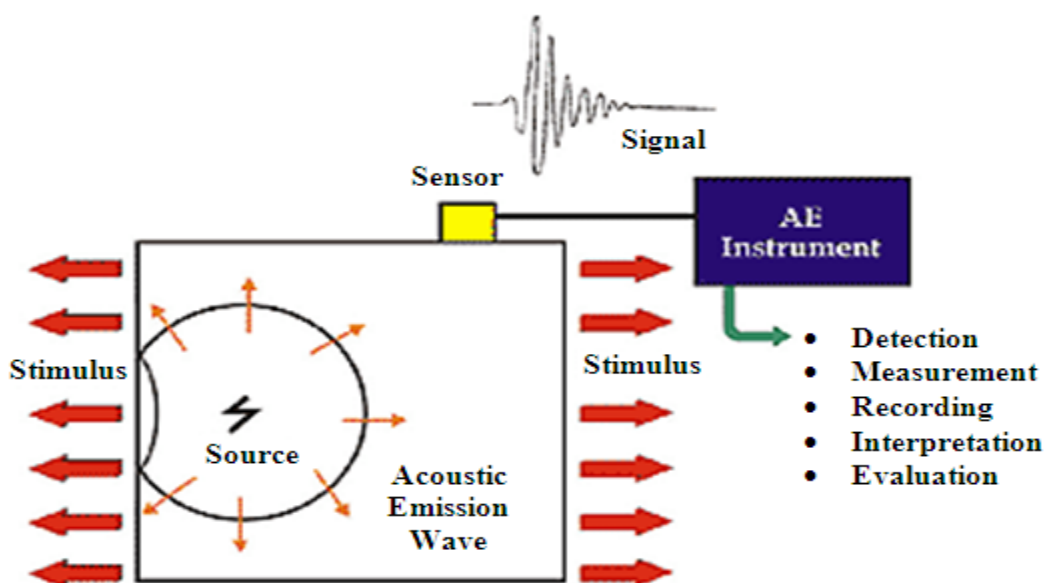


Figure D-1. Acoustic Emission concept.