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Fractures and Self-healing within the Excavation Disturbed Zone in Clays (SELFRAC)

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Foreword

Management of spent fuel and long-lived radioactive waste is an important environmental issue today for all nuclear power generating countries. Deep geologic disposal of these wastes in clay is found to be a suitable management option. An essential feature that needs to be considered for the long-term safety of a high-level waste underground repository is the Excavation Damaged Zone (EDZ) near the repository and its hydro-mechanical and geochemical evolution with time.

The creation of an excavation disturbed or damaged zone is expected for all geologic formations. Macro- and micro-fracturing, and in general a rearrangement of rock structures, will occur in this zone resulting in significant increases of permeability to flow. Implications of the higher permeability of the damaged zone and its time evolution under various repository scenarios need to be evaluated as part of waste repository safety assessment. Various issues, such as processes creating fractures in the excavation damaged zone, the degree of permeability increases, the potential for sealing or healing (with permeability reduction) in the zone, all need to be part of the evaluation.

In recent years, efforts along these lines have been made. However, there was no established consensus about the terminology and the impact of the EDZ on the performance of radioactive waste geological repositories. The challenge of the SELFRAC project was to clarify the significance of the EDZ in the safety case for disposal in clay host rock building the bridge between geomechanics and performance assessment.

The SELFRAC project was a key issue to build the sound scientific background in this field, to carry out a safety assessment, and to increase public confidence in long-term predictions of the behaviour of a deep nuclear waste repository in clay host rocks.

The SELFRAC project was co-ordinated by EIG EURIDICE (European Underground Research Infrastructure for Disposal of Nuclear Waste in Clay Environment), Belgium. It was a joint project with NAGRA, Switzerland, Laboratoire 3S, France, LMS-G3S, France, Katolieke Universiteit Leuven, Belgium, École polytechnique fédérale de Lausanne, Switzerland, and Solexperts, Switzerland.

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Executive summary

Radioactive waste must be managed and disposed off in ways that ensure the protection of people and the environment. In this sense, the long-term prediction of disturbances in the surrounding rock mass induced by the construction and the operation of a waste repository is essential to ensure a maximal protection of the environment and the people for both current and future generations against soil contamination and radionuclide release. The Damaged Zone (DZ) is first initiated during the repository construction. Its behaviour is a dynamic problem, dependent on changing conditions that vary from open-drift period, to initial closure period, to the entire heating-cooling cycle of the decaying waste. Other factors concern the even longer-term issues of chemical reactions and biological activities. The SELFRAC project focuses on the DZ created by the excavation (EDZ) in clay host rocks. Two different potential geological formations for deep radioactive waste repositories were investigated: the indurated opalinus clay of Mont Terri (Switzerland) and the plastic Boom Clay of HADES (Belgium).

For a high-level nuclear waste repository, the damaged zone caused by excavation is of concern for the performance assessment if it affects the radionuclide migration. Damage in Performance Assessment (PA) is therefore different from damage in geomechanics. Consequently, a clear distinction has been proposed within the SELFRAC project between the Excavation Damaged Zone (EDZ) and Excavation disturbed Zone (EdZ). The definitions were established in the context of underground radioactive waste disposal, establishing a bridge between geomechanics and PA:

- The Excavation disturbed Zone (EdZ) is defined as a zone with hydro-mechanical and geochemical modifications, without major changes in flow and transport properties. Within the EdZ there are no negative effects on the long-term safety.
- The Excavation Damaged Zone (EDZ) is defined as a zone with hydro-mechanical and geochemical modifications inducing significant changes in flow and transport properties. These changes can, for example, include one or more orders of magnitude increase in flow permeability.

These definitions were internationally discussed and accepted at the conference "Impact of the Excavation disturbed or Damaged Zone on the performance of radioactive waste geological repositories", held in Luxembourg on 3-5 November 2003. However, the definitions should be deepened for each type of rock and each site. In particular, the terms "significant" and "major" should be quantified and the time effect should be determined for each particular site.

Within the SELFRAC project, laboratory tests and models were developed to better understand and quantify the fracturing, sealing and healing processes.

Substantial progress was realised about our understanding of the fracturing processes. Fractures may appear around any underground excavation in clay host rocks, provided the stress concentration is higher than the material strength. Most of the observed processes can be reliably described and understood even if sometimes a complete answer on the origin and development of the fracturing is still missing. From a PA point of view, understanding the origin of fracturing is not of a critical importance. It is the state of the fractures and their evolution with time that is important, to assess the impact on the transport properties. The most important parameters for PA are therefore the porosity and the overall effective permeability. Within the SELFRAC project, the influence of the porosity and the discontinuities on the global permeability has been established. Induced fractures along tunnels has been characterised in details from various in-situ experiments both at the Mont Terri Rock Laboratory and at HADES underground research laboratory. We can therefore conclude that fractures and porosity changes due to excavation are now well characterised and quantified as well as their influence on the permeability.

Microfocus X-ray computer tomography reveals to be a very valuable technique to visualise the sealing process and its evolution with time. The acoustic emission (AE) technique can be used to investigate damage and damage evolution at micro-scale, and hence also sealing and healing. However, the quantification of the processes needs further research.

All experimental teams observe both in Opalinus Clay and Boom Clay a clear tendency for sealing processes while healing processes are quite limited. An influence of the interstitial water chemistry on the sealing process was clearly observed. For Boom Clay the sealing process occurs very quickly after saturation of the system. Opalinus Clay requires much more time for the sealing processes. Nevertheless, observations indicate reductions of transmissivity of several orders of magnitude within a few years. The process is fast enough from a PA point of view since they are considering very long periods of time. Even if the processes leading to sealing and healing behaviour are not fully understood yet, the governing processes (swelling, creep, disintegration and consolidation) have been identified within the SELFRAC project. Although detailed understanding of the sealing process on the micro-scale is not critical for PA, a sound scientific understanding is necessary to support the arguments presented within a safety case. The tests realised in SELFRAC have allowed characterising and quantifying the sealing processes in Boom Clay as well as in Opalinus Clay. The influence of the sealing processes on the evolution of the permeability was also determined.

The in-situ experiments performed in Opalinus Clay show that the effective hydraulic conductivity of the EDZ is expected to be relatively quick, within several years, lower than 10^{-10}m/s as soon as the bentonite backfill of the emplacement drifts becomes fully saturated and the expected swelling pressure will be built up.

The in-situ experiments in Boom Clay have allowed to follow the evolution with time of the hydro-mechanical behaviour of Boom Clay around a gallery excavated by industrial technique and to quantify the effect of the sealing processes on the hydraulic conductivity evolution. The radial extent of the fracture zone around the gallery is about 1 m. However, a slight increase of the hydraulic conductivity was measured up to 6-8 m into the host-rock. The results confirm that Boom Clay has important sealing properties. It was shown that two years after the excavation, the interconnected fractures zone was reduced from 1 m to less than 60 cm around the gallery. The hydraulic conductivity in the sealed zone and beyond in the host rock remains lower than $2.5\ 10^{-11}$ m/s.

It can be expected, for both Opalinus Clay and Boom Clay, that in the long term, the hydraulic conductivity of the EDZ will still decrease. It can therefore be assessed that the maximum hydraulic conductivity of the EDZ will be approximately one order of magnitude higher than that value of intact clays, which is in the case of Mont Terri in the overall range of $2\ 10^{-12}$ to $2\ 10^{-14}$ m/s (Heitzmann, 2004) and which is in the case of Boom Clay in the overall range of $2\ 10^{-12}$ to $4\ 10^{-12}$ m/s. Performance assessments models for different repository designs in different clay formations have demonstrated that even for significantly higher hydraulic conductivity of the EDZ than the expected ones, the overall performance of the repository system would not be adversely affected and that radiation doses remain well below regulatory guidelines.

Great efforts were placed in the modelling work during the SELFRAC project. On the one hand, new constitutive and numerical developments were performed to model the fracturing and sealing/healing processes; on the other hand, numerical simulations have been used to help the interpretation and the better understanding of the laboratory tests.

An important progress has been made in fracture process modelling. The strain localisation analysis, which consists in a numerical approach to predict the onset of the shear banding, crack and fracturation in geo-materials, studied for more than 20 years for monophasic materials, was extended to a coupled hydromechanical analysis. The CLoE model implemented in the finite element code LAGAMINE has been calibrated for the Boom Clay and the Opalinus Clay on the basis of the laboratory test results. The numerical simulations showed that such a theoretical framework is capable of fully describing the HM behaviour of the material, including the occurrence of localised deformation and related local drainage phenomena in Finite Element Modelling (FEM). Moreover, the local second gradient theory has been developed and extended to hydromechanical-coupled porous media for the post-bifurcation analysis. While this study is still at a preliminary stage and some further investigations are necessary in order to draw more clear conclusions on the influence of permeability, in the near future, more realistic constitutive models (non-associativity, yield function depending on mean stress, etc.) have to be used in coupled applications. It is necessary to stress that these developments are needed to properly model those problems involving localised zones. Especially a complete study in order to predict the development of the localised zone with the influence of these zones on the seepage by a full coupled problem is of great interest for the future. And this has to be able to predict the evolution of the EDZ caused by the excavation of openings.

Concerning the development for sealing/healing process modelling, based on the literature review and on the experimental results, a framework for the modelling of a time and stress dependent sealing process is proposed and discussed for the two studied clay rocks. In the present project, such a model could be applied but is not indispensable for the Boom Clay in which sealing is almost instantaneous when the state of stress is compressive. In the case of the Boom Clay, a classical poro-plastic or poro-viscoplastic contractant-dilatant model together with a porosity dependent permeability model is sufficient to account for the hydromechanically induced permeability variation in the compressive stress domain. Conversely, the proposed framework could be reliable for modelling the sealing process in Opalinus Clay provided that a single discontinuity or a well-identified set of discontinuities are considered for the calibration of the model, but there is a lack of experimental data to achieve that goal. The proposed constitutive model is promising to deal with fracture initiation and sealing. But further efforts are still needed for an application to a real-case modelling.

The developed constitutive and numerical tools have been applied to the in-situ test modelling. Efforts were concentrated on the connecting gallery excavation at Mol site with special attention on the strain localisation and parametric sensibility analysis on the hydraulic disturbance zone. The modelling of the excavation process and the strain localisation prediction along the process has been performed using the finite strain Finite Element Code Lagamine, with the constitutive law CLoE. A special modelling procedure was designed, taking into account the progressive excavation as a process in time, the placement of a lining behind the excavation front, the existence of an over-excavation with respect to the lining dimensions, and the hydromechanical coupling. The results are consistent with the site observations, especially the localisation predicted. Further refinements would be necessary to reduce some discrepancies observed with respect to site data. Sensitivity analysis using poro-viscoplasticity analysis revealed some important influencing factors with respect to the hydromechanical responses of Boom Clay to the excavation.

Results from the SELFRAC experimental programme as well as the developments in numerical modelling have strengthened the sound scientific background for the arguments presented within safety cases.

1 Introduction

In all nuclear power generating countries, spent nuclear fuel and long-lived radioactive waste management is an important environmental issue today. Disposal in deep clay geological formations is one of the promising options to dispose of this waste. An important item for the long-term safety of underground disposal is the evaluation of the EDZ in the clay host rock. The EDZ is first initiated during the repository construction. Its behaviour is a dynamic problem, dependent on changing conditions that vary from open-drift period, to initial closure period, to the entire heating-cooling cycle of the decaying waste. Other factors concern the even longer-term issues of chemical reactions and biological activities.

The SELFRAC project aimed to characterise the EDZ and its evolution with time in plastic and indurated clays. The perturbation of the excavation could lead to a significant increase of the permeability, related to diffuse and/or localised crack proliferation in the material. However, an opposite process, which involves the sealing and healing properties of clays, can in turn reduce the permeability in time. The main objective of the project was to understand and quantify these processes and to assess their impact on the performance of radioactive waste geological repositories.

This report describes the achievements reached within the SELFRAC project. In the following sections first the scientific and socio-economic objectives of the project are given with emphasis on strategic aspects. This is followed by a summary of the background information on the existing theoretical and experimental studies. A new terminology for the terms 'EDZ', 'sealing' and 'healing' is proposed. Results of the laboratory and in-situ tests to characterise the fracturing and the sealing processes are discussed and compared with the numerical simulations. Finally the report concludes on the long-term performance of a nuclear waste repository in respect of the EDZ zone around excavations and recommendations are suggested for excavation techniques and for repository designs.

In the frame of the SELFRAC project, an international conference/workshop, “Impact of the Excavation disturbed or Damaged Zone (EDZ) on the performance of radioactive waste geological repositories”, was held in Luxembourg on 3-5 November 2003. It was jointly organised with the EC in the frame of its CLUSTER forum. This event was essential to stimulate exchanges between the organisations in charge of building the safety cases for geological repositories, the regulatory authorities, and the scientific community.

[1] CLub of Underground Storage, TEsting and Research facilities for radioactive waste disposal.

2 Objectives and strategic aspects

The SELFRAC project complies with the objectives mentioned in the nuclear fission programme of the European Commission in the frame of the safety of the fuel cycle research activities.

Radioactive waste must be managed and disposed off in ways that ensure the protection of people and the environment, now and in the future. In this sense, the long-term prediction of disturbances in the surrounding rock mass induced by the construction and the operation of a waste repository is essential to assure a maximal protection of the environment and the people for both current and future generations against soil contamination and radionuclide release. Indeed, the evaluation of the evolution of the EDZ with time is particularly important since its presence may result in changes of the transport characteristics of the rock mass adjacent to underground openings.

The SELFRAC project aimed to characterise the EDZ and its evolution with time and to assess its impact on the performance of radioactive waste geological repositories. The main objective was to understand and to quantify the fracturing and the sealing/healing processes. Two different potential geological formations for deep radioactive waste repositories were investigated: the indurated Opalinus Clay of Mont Terri (Switzerland) and the plastic Boom Clay (Belgium).

A systematic approach was used for the development of the constitutive models on basis of laboratory tests. The developed tools allowed to simulate and to quantify the development of fractures around excavations and the sealing processes for a better assessment of radionuclide transport in the host rock. In situ tests were also performed, in order to characterise the EDZ and its evolution with time and to compare the in situ results with numerical simulations. Recommendations for excavation techniques and design of repositories were proposed. Based on the progress made within the project it is believed that SELFRAC could genuinely be considered as a key reference in terms of scientific background to support safety assessment exercises and to increase public confidence in the ability to make long-term predictions on the performance of deep nuclear waste repositories in clay host rocks. By this way, the SELFRAC project supports the EU policy objective on the adoption of national programmes on geological disposal.

In summary, the main scientific objectives of the project were:

- Review and reporting of the state of the art on basis of existing theoretical studies and in situ observations realised in underground laboratories. As in the literature some terms led to confusion, the terminology needed to be clarified;
- Experimental characterisation of fracturing and sealing/healing processes in surface laboratories to give the necessary data to improve and calibrate constitutive models. More than 90 tests were performed and interpreted;
- Improvement and development of constitutive models to simulate fracturing and sealing/healing processes;
- Realisation and interpretation of four in-situ tests in Mont Terri and HADES underground laboratories;
- Numerical simulations and comparison with the experimental results;
- Drawing conclusions on the long-term performance of a nuclear waste repository in respect of the EDZ around galleries and shafts as well as making recommendations for excavation techniques and design of deep nuclear waste repositories in clay host rocks;
- Drawing lessons from other applications: civil works, petroleum industry, gas storage.

3 Scientific and technical description of the results

3.1 Background information

The general hydromechanical behaviour of undisturbed over-consolidated clays depends mainly on their stress history and their structure. However, the hydromechanical behaviour of over-consolidated clays is also influenced by the presence of “discontinuities". Discontinuities can represent, e.g. a potential sliding surface, a zone of localised strains or a path for a significant flow of water. An elastic stiffness reduction and variations of the permeability can be the consequence of micro-cracks generated. Moreover, the influence of discontinuities on the mechanical behaviour of clays depends on the state of stress, the stiffness, the strength of the intact clay and also on the orientation of the discontinuities.

In each rock, micro-cracks and defects are present. The more indurated the rock, the greater its susceptibility to fracturing. There are many different causes of fracturing. A change in the state of stress can induce fractures. This can be a natural stress variation due to e.g. weathering (unloading), desiccation (decrease of pore water pressure) or tectonic processes. Fracturing can also be a consequence of human activities during borehole drilling or tunnel excavation resulting in a stress redistribution and allowing swelling and decompression that could lead to the failure of the material. Chemical effects can also affect the clay structure: chemical reaction with incoming water (dissolution), oxidation, etc.

The onset of failure and the mechanisms of propagation of discontinuities can be studied using both fracture mechanics and the bifurcation approach. The existing investigation and characterisation techniques of the discontinuities in the surface laboratories may include measurements of, for example, acoustic emission, wave velocities, conductance (or resistivity), permeability and strain, microscopic observations (optical or electron microscope), X-ray scanner computed tomography and stereo-photogrammetry.

In Boom Clay, the fracturing scheme induced by the excavation of a circular gallery is now well studied. All observed fractures at the Mol site were excavation induced. Pre-existing fractures were not observed.

Natural fractures in the Opalinus Clay have been observed in the folded Jura while only a very limited number of fractures has been detected in the Tabular Jura. In general, these fractures show no significant increase in hydraulic parameters compared to the intact rock as long as the effective stress normal to the fracture planes exceeds a certain threshold value (no inflow or wet spots are observed in tunnels in Opalinus Clay with more than 200 m of overburden – Gautschi, 2001). The structure of the EDZ is controlled by the stress field and the anisotropy (transversely isotropic material) of the Opalinus Clay. For tunnels constructed parallel to bedding, extensional features due to unloading dominate the EDZ in the side walls while the deformation in the roof and floor is controlled by the strength of the bedding planes. A combination of shear and extensional failure characterises this area.

The review on the sealing and healing processes showed that there was not yet an established theory to describe them. The processes depend highly on the properties of the rock and its state. Stress state for contractancy, dilatancy, swelling and sealing with newly formed minerals are the main mechanisms mentioned in the literature.

3.2 Terminology

For a high-level nuclear waste repository, the damaged zone caused by excavation is of concern for the performance assessment if it affects, directly or indirectly, the radionuclide migration. Damage in PA is therefore different from damage in geomechanics. Consequently, a clear distinction has been proposed within the SELFRAC project, between the Excavation Damaged Zone (EDZ) and the Excavation disturbed Zone (EdZ). The following definitions try to establish a bridge between PA and geomechanics:

The **Excavation disturbed Zone** (**EdZ**) is defined as a zone with hydro-mechanical and geochemical modifications, without major changes in flow and transport properties. Within the EdZ there are no negative effects on the long-term safety.

The **Excavation Damaged Zone** (**EDZ**) is defined as a zone with hydro-mechanical and geochemical modifications inducing significant changes in flow and transport properties. These changes can, for example, include one or more orders of magnitude increase in flow permeability.

These definitions were validated at the conference/workshop "Impact of the Excavation disturbed or Damaged Zone (EDZ) on the performance of radioactive waste geological repositories", held in Luxembourg, 3-5 November 2003. However, the definitions should be further detailed for each type of rock and each site. In particular, the terms "significant" and "major" should be quantified and the time effect should be discussed for each particular site.

The definitions of 'sealing' and 'healing' are as follows:

Sealing is the reduction of fracture permeability by any hydromechanical, hydrochemical, or hydro-biochemical processes. During the excavation and/or storage phases discontinuities can be created or reactivated, increasing among others the hydraulic transmissivity and decreasing the strength of the overall material. Sealing is considered to be the closure of the newly formed discontinuities or re-activated discontinuities, so that a stress transfer across the discontinuities is possible. Sealing could occur by e.g. an increase of the stress state, by a change in water content and/or swelling of clay minerals. As no structural change occurs between both sides of the discontinuities, the discontinuities are still present after the sealing process and there is no bond between both discontinuity sides.

Healing is sealing with loss of memory of the pre-healing state. Thus, for example, a healed fracture will not be a specially preferred site for new fracturing just because of its history. While micro cracking, crack growth or fracturing are considered to be processes by which the cohesion of a plane is lost, healing is considered to be the process by which the cohesion is restored. The process can be influenced by stress, temperature, pore pressure, chemical (e.g. oxidation and precipitation) and mineralogical changes (e.g. crystallisation), etc. This means that structural changes occur between both sides of the discontinuities. The difference between the result of the sealing and healing process, is that for the latter a mechanical strengthening occurs, which is absent for sealing. In case of partial healing, the hydraulic and the mechanical properties assume values that are intermediate between those of newly fractured and fully healed material.

Self: In the literature we are often speaking about Self-Sealing or Self-Healing. 'Self' means that the process of healing or sealing happens spontaneously in the rock mass without interference by intentional human actions. Intentional human actions mean that they are not linked to the normal excavation and disposal phases. The fact that the sealing or the healing happens spontaneously or not could be sometimes controversial. Moreover this information is not relevant for PA. Therefore, only the generic terms of 'sealing' and 'healing' are used in this report.

3.3 Laboratory tests

The perturbation caused by an excavation may lead to a significant increase of the permeability, related to diffuse and/or localised crack proliferation in the material (Process A). However, an opposite process, which involves the sealing and healing properties of clays can in turn reduce the permeability in time (Process B).

An extensive laboratory program including about 90 tests was realised to characterise the fracturing and sealing/healing processes around an excavation. Innovative experimental set-ups were used to reach this objective: triaxial tests with permeability evolution monitoring, visualisation of the sealing/healing process by micro-focus X-Ray computer tomography (μCT), characterisation by acoustic emission. Two types of clay were investigated: the indurated Opalinus Clay and the plastic Boom Clay.

For sake of conciseness only the most relevant tests are presented in this report. The complete description of the tests procedures and results can be found in Vervoort et al., 2006.

3.3.1 Uniaxial, biaxial and triaxial tests

Uniaxial, isotropic compression and triaxial compression tests, including permeability measurements, plus biaxial tests were performed by L3S to characterize the hydromechanical coupling in Boom Clay and Opalinus Clay. These data were used to calibrate the constitutive model.

3.3.1.1 Boom Clay

The stress-strain relation has been established under various states of stress ranging from the overconsolidated domain to the normally consolidated domain.

On the basis of the results obtained, we can say that Boom Clay exhibits two types of mechanical behaviour depending on the mean effective stress (shear bands develop for highly overconsolidated state whereas no localisation in shear bands occurs at the in situ mean effective stress). At the in situ state of stress and in drained condition, the behaviour is ductile and contractant, i.e., the q-ε_{axial} curve shows a plateau. The hydraulic conductivity was found to slightly decrease from $7.15\ 10^{-12}$ to 10^{-13} m/s due to the contractance of the pore space during the shearing phase. Conversely, at a low mean effective stress (0.4 MPa), the mechanical behaviour is characterized by the development of shear bands leading to the failure of the specimen. The q-ε_{axial} curve shows a peak and the specimen exhibits dilatancy (see Figure 1).

However local strain measurements and the stereophotogrammetry technique allow the characterization of the fracturation processes. Local measurements of axial and radial strains have allowed the detection of the localisation onset: shear band formation occurs just before the peak stress. Stereo-photogrammetry method was successful in studying the propagation of shear bands in the specimen for the biaxial tests (see Figure 2). In addition, an interesting result has been observed: strain localisation appears to be dependent on the strain rate, i.e., at high strain rate shear bands developed even at the in situ mean effective stress but at low strain rate no shear bands occur.

The hydromechanical (HM) behaviour of Boom Clay has been well characterised and quantified through this testing program. Experimental data on the HM behaviour under isotropic and deviatoric conditions (including triaxial extension tests) have been obtained.

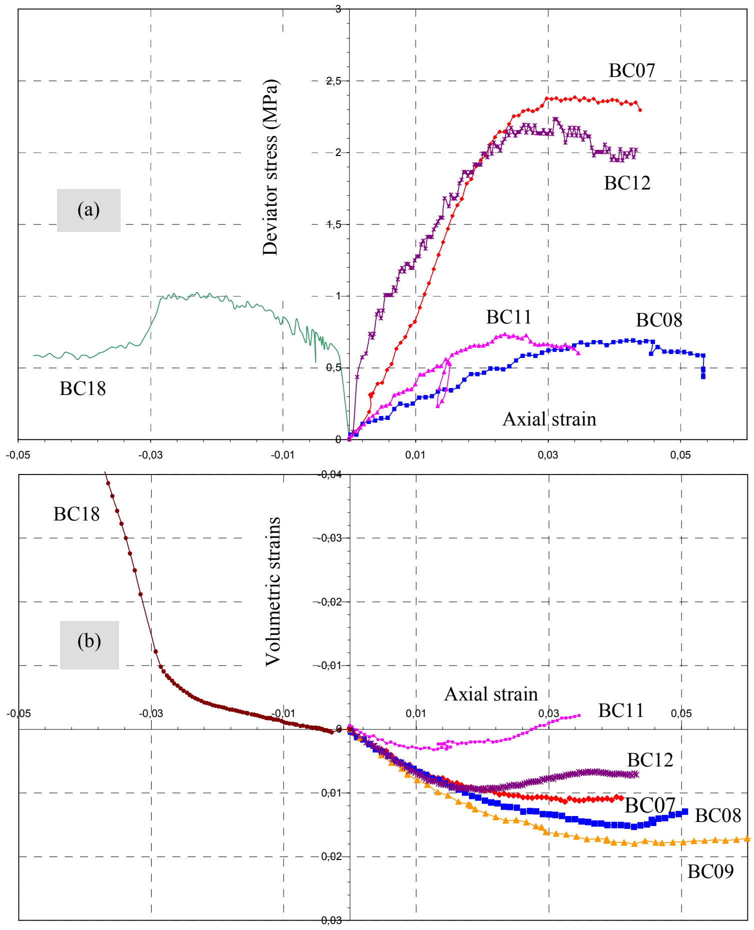


Figure 1: Shear tests on Boom Clay on samples consolidated at different effective isotropic stress: 0.4 MPa (BC11, BC08), 2 MPa (BC18), 2.3 MPa (BC07, BC12) (a) deviator stress, (b) volumetric strains versus axial strain

(a) (b) (c)

Figure 2: Boom Clay specimen after biaxial testing (a), detail of a shear surface (b) and a surface which opened up in tension (c)

3.3.1.2 Opalinus Clay

Uniaxial compression tests and undrained triaxial compression tests have been performed. However, the very low permeability associated to a very low porosity and low water content make the tests on Opalinus Clay more difficult to perform than on Boom Clay. As a result the consolidation phase was not performed but specimens were in contact with the pore fluid under pressure (2.2 MPa) during a few hours in order to improve the saturation of the system (porous disc, hydraulic lines).

During undrained triaxial compression tests, performed at various mean effective stress (0.4, 2.3 and 5 MPa), Opalinus Clay exhibited a brittle behaviour (the q-ε_{axial} curves showed a peak) with the development of numerous fissures more or less parallel to the direction of the major stress and we can not say if shear bands were pre-existent to these fissures. Fracturation is less important at high mean stress. Moreover a decrease in excess pore water pressure was observed at both ends of the specimen before the peak stress indicating dilatancy. The intact permeability of Opalinus Clay was not measured but, at low effective mean stress (0.4 MPa), a significant increase of the hydraulic conductivity (up to 10^{-9} m/s) was observed after the failure of the specimen. At high mean effective stress (for tests with 2.3 and 5 MPa), the specimens did not exhibit this evolution of permeability even after undergoing large axial strains. This indicates a probable influence of the confining pressure on the fracture apertures and so on the fluid flow in the fractures.

Concerning the fracture process, fissures were observed rather than shear bands. Stereo-photogrammetry was not sufficient to quantify them (using image correlation is not suitable to deal with large discontinuities).

3.3.2 Triaxial tests on hollow cylindrical samples

Tests were performed by LMR-EPFL in the LEGEP triaxial cells (see Figure 3) on hollow cylindrical samples subjected to pseudo-isotropic stress conditions. Radial convergent and divergent flows were generated in the sample by imposing a constant pressure difference between the outer and inner hydraulic circuits.

The challenge of the tests was to quantify the permeability changes induced by localised cracks and their potential sealing on the same sample and without removing it from the LEGEP cells. For this purpose, the testing procedure was foreseen as follows:

1. measurement of radial convergent and divergent fluid flows on the undisturbed sample;
2. tensile fracturing of the sample (without removing it from the cell) induced by an axial traction;
3. measurement of fluid flows just after the fracturing of the sample;
4. sample kept under a constant confining pressure and measurement of radial convergent and divergent fluid flows from time to time.

Step 3 allows measuring the increase of permeability related to localised crack proliferation (process A), and step 4 enables to check the potential reduction in permeability with time and thus the sealing capability of the tested clays (process B).

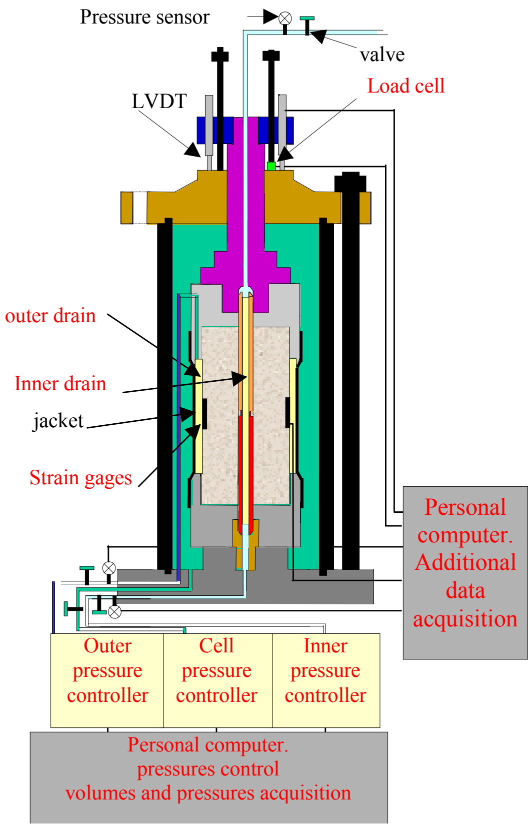


Figure 3: Modified LEGEP testing device

As several experimental difficulties arose during a trial test with the original LEGEP set-up available at LMR-EPFL and the results were found not sufficiently satisfactory and reliable, the testing device and procedure were completely reconsidered. Improvements were made to different parts of the test apparatus (inner drain, outer hydraulic circuit, pressure controllers, data acquisition, etc.) and new cells were built (see Figure 3) An improved testing procedure was as well defined, with due consideration on (i) the saturation and consolidation stages of the specimens after their preparation and set-up, (ii) the generation of a tensile fracture in the sample without removing it from the triaxial cell, and (iii) the measurement of the change in permeability during the tests.

Six tests were carried out on Boom Clay samples. The first test with the modified LEGEP testing device was not completely successful as no tensile fracture occurred in the sample. The hydraulic conductivity measurements (4.3 to 4.4 $\cdot$ 10^{-12} m/s) carried out during this test were however interesting because no change in water conductivity was observed, even under an axial and extensional deformation of 10 % and the very clear development of shear bands. This conclusion of no change in hydraulic conductivity related to shear bands is in accordance with observations made by L3S on Boom Clay samples subjected to triaxial compression tests.

After further adaptations of the testing procedure, the next experiments on Boom Clay were completely successful with the creation of a tensile fracture in the sample, and hydraulic conductivity measurements carried out before and after fracturing to follow the evolution of the hydraulic conductivity with time (Processes A and B). The following conclusions can be drawn from those tests:

- The laboratory measurements of the hydraulic conductivity on intact samples range between 2.3 and $4.1 \cdot 10^{-12}$ m/s and are in excellent agreement with values usually reported in literature (2 to $4 \cdot 10^{-12}$ m/s).

- As soon as the tensile fracture arises in the sample (after 0.5 to 1 % of deformation), a huge increase in flow (several orders of magnitude) is observed. The hydraulic conductivity when the induced fissure is open was however not measured because of the difficulty to keep a constant pressure gradient between the outer and inner drains (clear connection through the induced fissure) and because the flow depends on the fracture aperture.
- On the other hand, once the induced fracture is closed and subjected to a certain normal stress, the hydraulic conductivity measured a few days after fracturing (when a nearly steady-state flow is reached in the sample, i.e. after about 3 days) is found to be hardly different from that measured on the virgin sample.

It appears in conclusion that fractures developing in Boom Clay can undergo a very fast sealing when subjected to an almost hydrostatic state of stress close to the in situ one (4.5 MPa total stress). After the removal of the samples from the cell, the fracture was hardly discernible and a small traction was even required to reopen it, pointing out some healing effect (unfortunately not quantifiable).

The testing campaign on the Opalinus Clay specimens from Mont Terri (sandy facies) started early 2004. Owing to the extremely low hydraulic conductivity of virgin Opalinus Clay (in the range of 10^{-13} m/s), very long testing time was expected and it was decided to test three samples in parallel. As a small sealing process was noted 2 months after fracturing, the tests were carried on till the end of the SELFRAC project and lasted more than 10 months. The following conclusions can be drawn:

- The hydraulic conductivity values for virgin samples measured in the laboratory tests are in the same range as those derived from in situ experiments (water inflows in boreholes, hydraulic testing) carried out in the Mont Terri Underground Laboratory, i.e. in the order of $3 \cdot 10^{-13}$ m/s for flows parallel to the bedding planes within undisturbed Opalinus Clay of the sandy facies (e.g. Bossart & Thury, 1999).
- After the tensile fracturing of the sample, a huge increase in flow was observed (about 4 to 5 orders of magnitude higher than on the virgin sample) and a steady state flow was easily achieved. On the other hand, it was not possible to keep a 100 kPa pressure difference between the outer and inner drains because of their clear connection through the induced crack. The hydraulic conductivity derived from the flow measurements just after fracturing ranges between $2 \cdot 10^{-9}$ and 10^{-8} m/s. This range is rather consistent with hydraulic conductivity values in the excavation-disturbed zone at Mont Terri yielded from a number of experiments with differing concepts and instrumentation. Martin & Lanyon (2002) report conductivities of the order of 10^{-8} to 10^{-7} m/s in the sidewalls rocks (whereas a substantially lower value of about 10^{-10} m/s is observed in the excavation-disturbed rocks of the invert and roof).
- The last step of the experimental procedure consisted in keeping the samples under a constant confining pressure and in measuring radial convergent and divergent fluid flows from time to time to check the potential sealing of the fracture with time. A reduction in water conductivity is noted with time (see Figure 4): i.e. a decrease by one order of magnitude 5 months after fracturing. This rate is similar (a bit slower) to the decrease in transmissivity with time obtained from hydraulic in situ testing performed at the EH-1 site of Mont Terri before the in situ experiment n°1 carried out in the framework of this project (see section 3.4.1.1). The progressive sealing of the induced crack is confirmed by the longer time necessary to reach a steady state flow in the sample as well as by the increase in pressure difference imposed between the outer and inner drains (≡ increase in hydraulic gradient in the fissure).

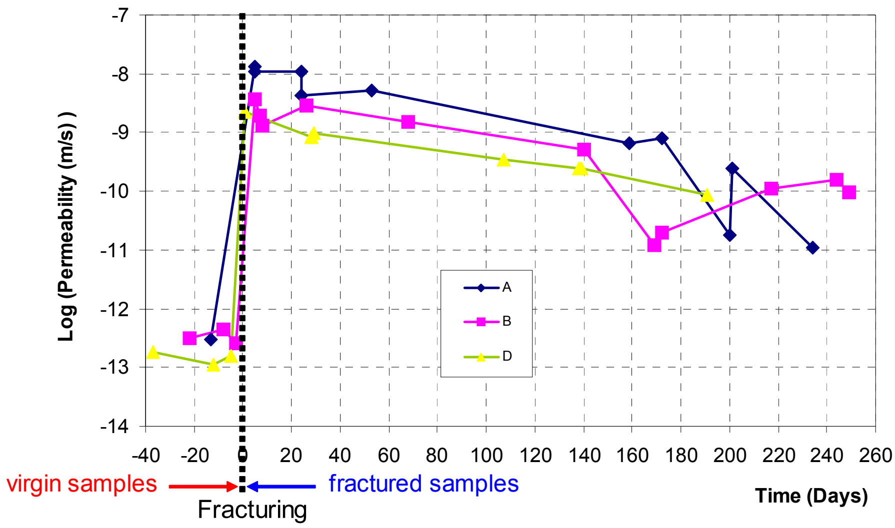


Figure 4: Evolution of the hydraulic conductivity with time in fractured Opalinus Clay samples

In conclusion, when a localised fracture is generated by traction in Opalinus Clay, a huge increase in hydraulic conductivity is noticed, even if the induced crack is kept closed under a normal stress close to the in situ one (4.5 MPa total stress). However, if the fracture is kept closed during a certain time under this 4.5 MPa normal stress, a slow sealing process is noted as time elapses (i.e. about one order of magnitude less permeable after 5 months). The reduction in hydraulic conductivity after fracturing is however much slower than the one measured on Boom Clay (extremely fast recovery of the virgin clay hydraulic conductivity). Moreover, contrary to Boom Clay, the fracture was clearly discernible after the removal of the samples from the cell and almost no traction was required to reopen it (no healing effect).

Permeability tests on hollow cylinders were also performed by LMS/G3S. Sealing processes have been observed for both Boom Clay and Opalinus Clay but no healing. For the Boom Clay, a significant – and reversible – mean stress-permeability dependency is observed: permeability increases by a factor 3 when the mean stress is decreased from 4.5MPa to 1MPa. This result is quite consistent with the observations of L3S.

3.3.3 Isostatic and permeameter tests with µCT visualisation

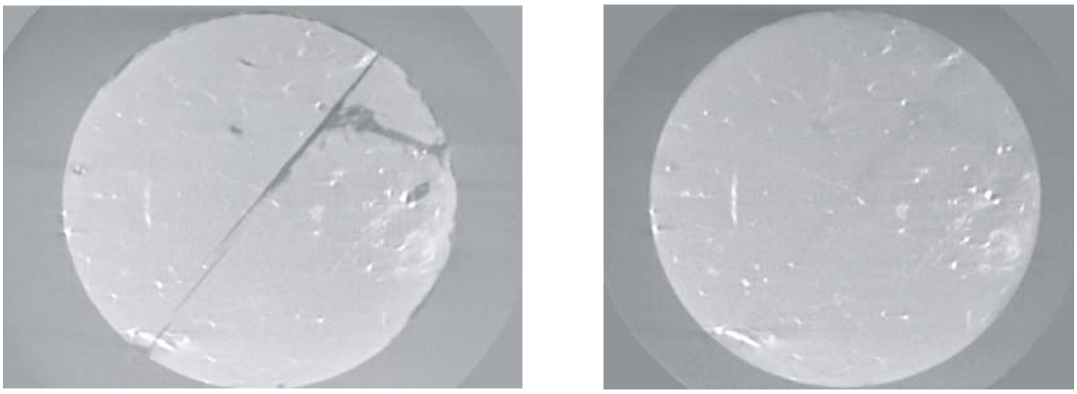
Isostatic and permeameter tests were performed by SCK.CEN. The specific objectives of these laboratory tests were:

- To assess the modifications of flow properties due to the presence of a main single discontinuity in Boom Clay and Opalinus Clay samples by monitoring the evolution of their hydraulic conductivities;
- To test the influence of the chemical composition of the solution injected through the clay samples on their sealing properties;
- To establish a correlation between the type of clay, the discontinuity geometry and dimensions, the state of stress and the flow properties;
- To visually observe the evolution of the sealing process with a microfocus X-ray computer tomography technique

To meet these objectives, two different types of laboratory tests were carried out: permeameter and isostatic tests. The permeameter tests consisted in testing the influence of the injected fluid chemistry on the sealing capacity of the clay. An artificial planar fracture has been produced through the axis of a cylindrical sample prior its installation in a permeameter cell. The evolution of the samples hydraulic conductivity has been monitored. We used micro focus X-ray computer tomography (µCT) as an imaging technique to visualise the sealing process of the fracture. The isostatic tests consisted in testing combined mechanical and chemical sealing. The principle was to drill a hole along the axis of a cylindrical clay sample and to submit it to an increasing confining pressure, while the evolution of the clay hydraulic conductivity was monitored by applying an axial hydraulic gradient. Mean pore water pressure and mean effective stress were kept balanced throughout the test.

A sealing phenomenon occurring both for Boom Clay and Opalinus Clay has been clearly observed. It was also found that changes in the interstitial water chemistry had a clear influence on the speed of the sealing process. In particular the use of an alkaline fluid was the most favourable.

For all the Boom Clay samples tested, the final measured hydraulic conductivity was comparable to undisturbed clay, and the initial planar fracture was no longer discernible (see Figure 5). However, the difference between the tested fluids mostly lied in rate of the sealing phenomenon, and in the recovered cohesion level. The use of an alkaline fluid was favourable for both parameters.



a b

Figure 5: Visualisation of the sealing process by µCT technique after saturation of the fracture

For the tested Opalinus Clay samples, small difference was observed between the tested fluids. The final hydraulic conductivity measurement varied between 2.10^{-12} m/s for the low-salinity Pearson water and around 6.10^{-12} m/s for the same solution saturated with Calcium hydroxide. A slower evolution towards the final value was observed for the high-salinity Pearson water.

In the isostatic tests carried out on the Boom Clay samples a fissure appeared in the centre of the samples, after the release of the confining pressure and the dismantling operation revealed the presence of a central zone with a lower density. The extent of this zone and of the fissure was clearly dependent on the original hole diameter (6 and 12 mm). The fissure was oriented in a parallel direction vs. the bedding plane. Figure 6 shows two images of the mean slice of the Boom Clay sample after the dismantling from the isostatic cell. The left image represents the mean slice, averaging the clay matrix while ignoring the fractures, with original artificial hole of 12 mm diameter (indicated in white). Around the original position of the hole, a low density zone (black colours) with an eye-shaped form is seen. The long axis of this form is parallel to the bedding plane orientation. The right image shows the same mean slice but with superimposition in white of the occurrence of the fracture throughout the sample. It is clear that the fracture is limited to the low density zone.

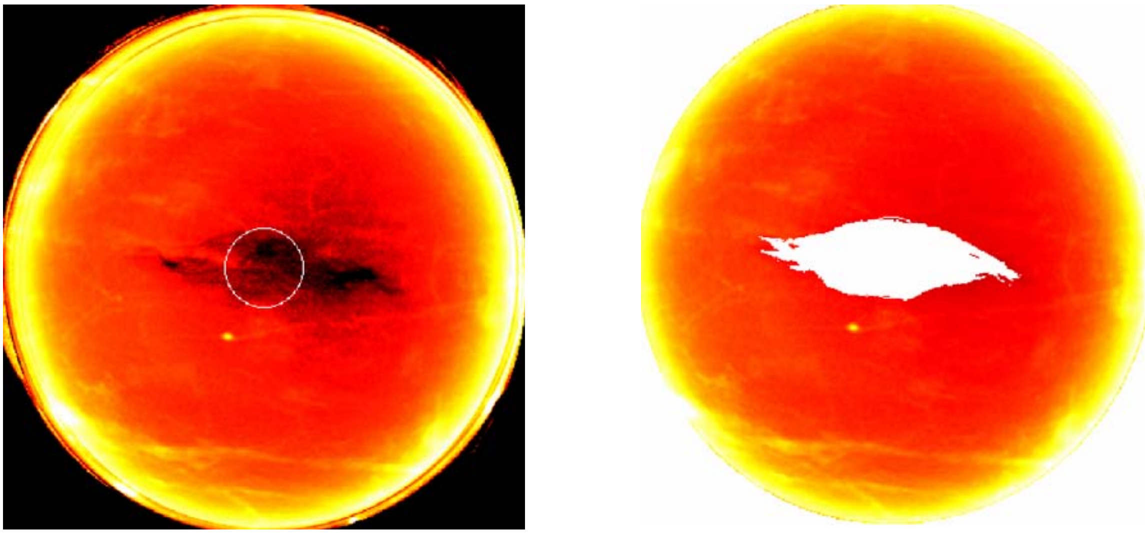


Figure 6: Mean slice of the Boom Clay sample dismantling from isostatic cell

For the tested Opalinus Clay samples, the sample with the lowest internal hole diameter (2 mm) showed evidence of a sealing process (see Figure 7). However, as for the permeameter tests, the initial undisturbed hydraulic conductivity value could never be reached, but it might be a matter of time. The sealing process in Opalinus Clay is indeed much slower than for plastic clay.

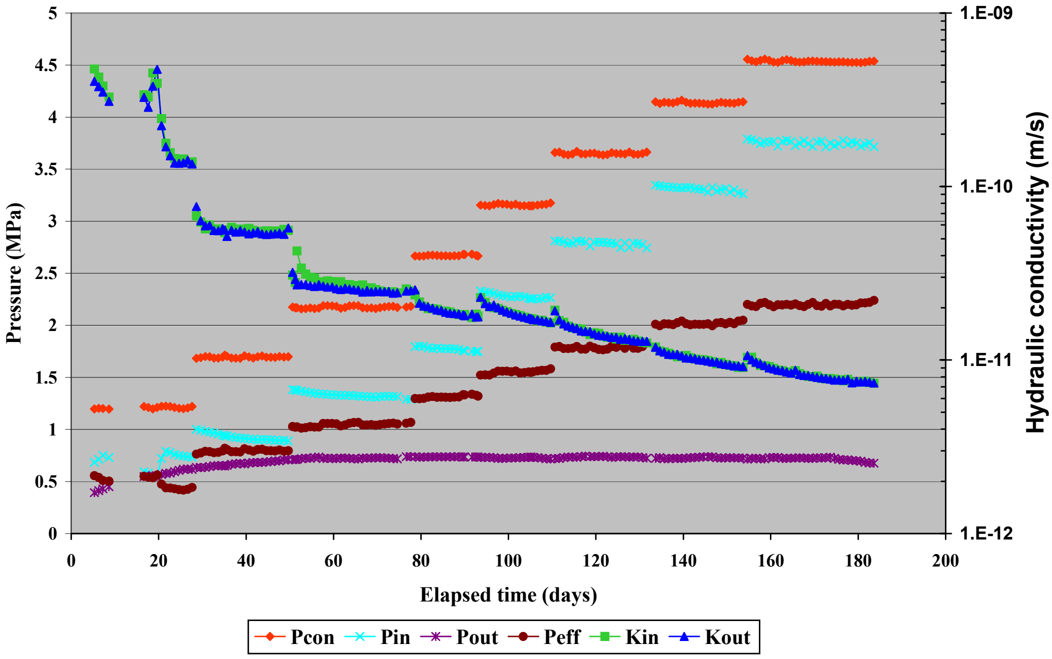


Figure 7: Evidence of sealing process in Opalinus Clay

In conclusion, both clay formations show a clear trend of a sealing process, when subjected to a confining pressure, while healing is quite limited. Boom Clay seals very fast, even in absence of loading. Opalinus Clay requires more time: a complete sealing could not be demonstrated within the time scale of the laboratory tests. However, a long-term in situ sealing experiment on Opalinus Clay seems to indicate that a complete hydraulic sealing could be achieved (Blümling et al., 2005).

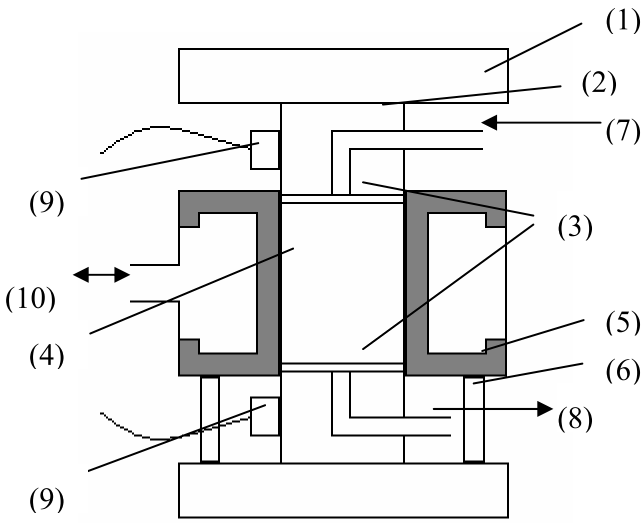
3.3.4 Characterisation by acoustic emission measurements

KUL realised acoustic measurements to characterise the sealing/healing process.

The objective of this technique is to obtain an indirect estimation and quantification of fracture sealing/healing intensity in rocks and soils can be made by using the Felicity ratio parameter linked to the phenomena of stress memory recovery. Stress memory means the ability of geomaterials to accumulate, to keep and to reproduce information about the peak stress level experienced in the past. This ability manifests itself in the form of certain anomalies (maximums, jumps, inflexions, etc) in physical fields, properties and state parameters, taking place when the current stress exceeds the maximum previously applied stress level. Experiments include two cycles of loading of clay samples, with acoustic emission and strain measurements in each cycle.

A total of 22 preparatory tests and 23 triaxial tests were performed. For the research presented here, acoustic emissions are measured under different stress and loading conditions. Sealing/healing is studied by subjecting the samples to a time delay, under conditions close to the in situ state, and observing the differences in AE behaviour before and after this time delay. Prior to the real experiments, different loading regimes and set-ups were tested, allowing the most optimal AE measurements.

Apart from some initial tests, the tests were conducted in a triaxial cell, whereby the axial load is controlled by a loading machine and the confining pressure by an external pump. Pore pressure was controlled by an additional pump. AE hits were measured with broadband sensors, connected to the AMS3 system.



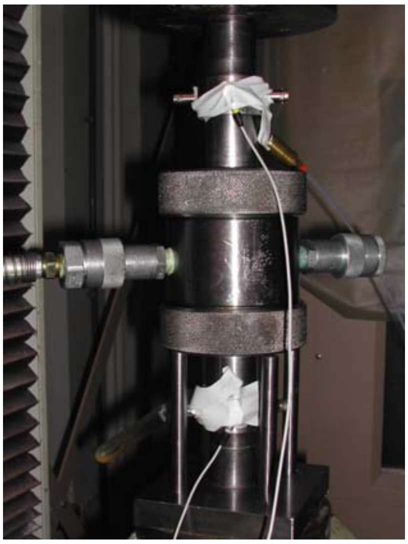


Figure 8: Schematic presentation (not at scale) of test set-up

with platens of loading machine (1), pistons (2), inserts (3), sample (4), rubber jacket (5), supporting rods (6), pore pressure inlet (7), pore pressure outlet (8), AE sensors (9) and confining pressure (10). Photo presents actual view of the triaxial cell, showing also sensors, pore fluid inlet and confining pressure inlet

A schematic presentation and photo of the test set-up are given in Figure 8. It shows how the sample is contained within the triaxial cell and how it is loaded. The axial load is exerted by the loading platens (1) onto the pistons (2), which transmit the load to the sample (4). A confining pressure ($\sigma_2 = \sigma_3$) is applied at the start of the test and its value is maintained by pumping oil in or out the cell (10). The rubber jacket (5) separates the sample from the oil and applies no extra pressure to the sample, if the radial deformations remain relatively small (< 5 mm). A pore pressure is applied through the top inlet (7) and is distributed evenly over the top of the sample through the porous inserts (3).

The pore fluid outlet (8) remains open, so no backpressure is applied. This means that the tests are performed with open drainage, and for high permeable porous soils or rocks (e.g. sandstone), the applied top pressure would only cause a water flow from top to bottom, and not a pore pressure. However, the hydraulic conductivity of Boom and Opalinus Clays is so small ($K \approx 3^{-12}$ m/s for Boom Clay, $K \approx 1^{-13}$m/s for Opalinus Clay (Coll et al., 2004)), that the water does not reach the bottom, and a pressure is in fact built up throughout the sample. But since steady state conditions are not reached in the given time, it is assumed that an inhomogeneous pore pressure distribution exists across the core sample. This low permeability has to be taken into account when calculating stresses. Even though the drainage line is left open, the tests can be regarded as partly undrained under the used displacement rate of 0.05 mm/min. However, the pore pressure within the sample is inhomogeneous, so effective stresses are also inhomogeneous. The pore pressure of 0.5 MPa is chosen low enough compared to total stresses (> 5 MPa) to be neglected, so the total stresses are approximately equal to the effective stresses. Other influences of the pore pressure are discussed in the detailed report by the K.U. Leuven (Vervoort et al., 2004).

The pore water fluid supplied by the pump consisted of distilled water in all but one test. It is thought useful to study sealing/healing without a chemical influence, so that sealing/healing is only determined by the different stress situations. The extra chemical influence was studied in a test which made use of the Synthetic Boom Clay Water. Furthermore, it is believed that the influence of pore water composition is negligible here, since time delays of the performed tests (21 days maximum) were too small to cause large chemical alterations (with the actual (low) concentrations).

Sensors were attached to the pistons, since no direct access to the sample is possible. This sensor set-up is also used in other reported studies (Khair, 1998). The dimensions of the triaxial cell required accurate and consistent dimensions for the tested specimens. The samples were cylindrical, with a diameter of 38 mm and a height of 76 mm, providing a height/diameter ratio of 2.

Different loading schemes during time delay were proposed, but the type I delay, with the same stresses as during the saturation period, is the most used in the real tests. An overview of the complete axial loading scheme for the type I test is presented in Figure 9.

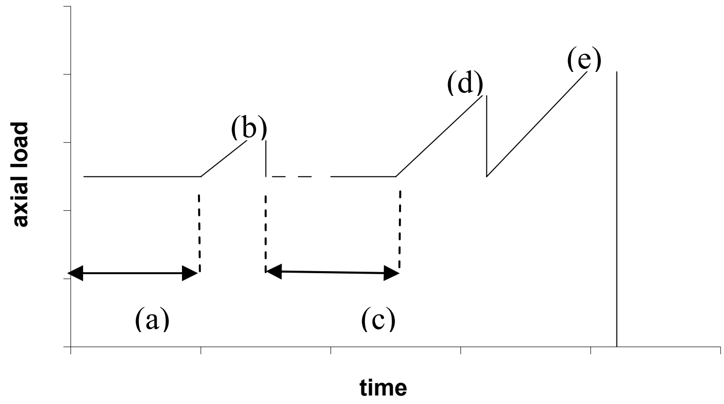


Figure 9: Schematic overview of axial loading scheme for type I test, with: isotropic loading (a), first loading cycle (b), time delay (c), second loading cycle (d) and third loading cycle (e)

Figure 9 only shows the axial load, since the confining pressure and pore pressure remain constant throughout the entire test. The different stages are also indicated in Figure 9: isotropic loading (a), first loading cycle whereby only the axial load is increased (b), time delay (c), second loading cycle (d) and third loading cycle (e). The third loading cycle is performed immediately after the second cycle and acts as reference; it should display a perfect Kaiser effect with an Fr (Felicity Ratio – represents an "index" of damage) of 1.

For both types of clay, a decrease in DSR (Deviatoric Stress Ratio – take into account the confining stress) is noticed. Furthermore, the Kaiser effect caused by frictional sliding is recognized in all cases, and yields a Fr of 1, as expected. The onset of frictional sliding corresponds in most cases to the onset of AE in the second cycle. This onset also decreases with increasing time delay, but its trend is less clear. It is believed that this onset is not significant for sealing/healing, and is caused by effects other than micro-fracturing in clay. However, modelling could give more insight into the processes at hand, and the consequences for sealing/healing.

Figure 10 compares the 'DSR clay' for both Opalinus and Boom Clay. It appears that both curves for Boom and Opalinus Clay lie close together. This would mean that the processes at hand (and thus possibly also sealing/healing) happen at the same rate for Boom as for Opalinus Clay. When looking at both curves, it seems that mainly from 2 to 9 days there is a decrease.

Results in literature point towards different conclusions concerning sealing/healing. In Shin and Kanagawa (1995), the Kaiser effect in granite was still clearly observed after 300 days, with a Fr of 1. However, other studies (Michihiro et al., 1992) point towards a fast decay in the Fr of granite, namely a Fr of 0.44 after 20 days time delay. This last study documented a monotonic decrease.

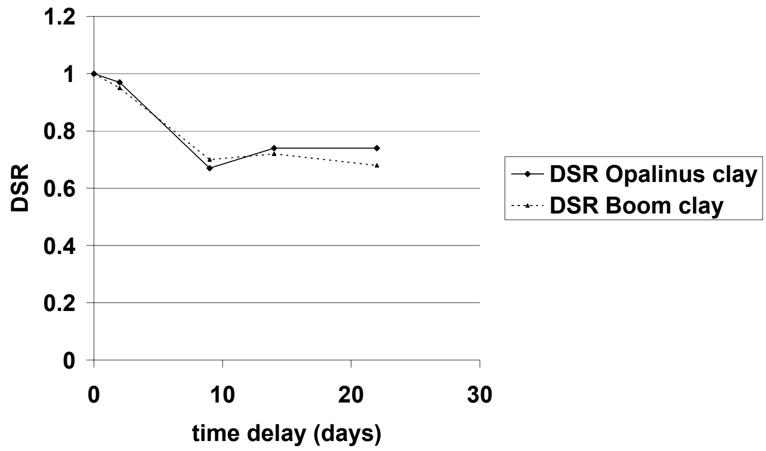


Figure 10: Comparison between DSR for Opalinus and Boom Clay

Since the AE method is a non-destructive technique, results rely on interpretations of secondary effects and not on direct observations. Instead of actually following micro-fracture formation, the generation of elastic waves, which is a result of this process, is monitored. This however causes uncertainties about the source of the signals. The future however may provide means to make more detailed distinctions between various AE sources. Also, the technique relies on human interpretation, which is always subject to error.

Furthermore, it is not known a priori if sealing/healing induces a decrease in the Fr. It is quite possible that other processes (also) induce this result. This problem is addressed in a separate modelling study (§ 3.5.3.2), whereby different processes are simulated, and the results projected towards the AE technique.

The acoustic emission technique can be used to investigate damage and damage evolution at micro-scale. However, further research is needed to be able to quantify these processes.

3.3.5 Conclusions

Microfocus X-ray computer tomography reveals to be a very valuable technique to visualise the sealing process and its evolution with time. The acoustic emission technique can be used to investigate damage and damage evolution at micro-scale. However further research is needed to be able to quantify these processes. In both Opalinus and Boom Clays clear tendencies to sealing processes have been observed while healing processes are quite limited. The composition of the interstitial water chemistry has a marked influence on the speed of the sealing process. For Boom Clay the sealing process occurs very quickly after saturation of the system. The results of the laboratory tests were used to calibrate and determine the parameters of the constitutive models.

Besides the scientific developments, the results of the laboratory programmes can be considered as key references in the confidence building of the performance of clay geological formations to host long lived radioactive waste and spent fuel.

3.4 In-situ tests

The development of the EDZ in the Opalinus Clay of the Mont Terri Rock Laboratory and in Boom Clay of the HADES underground laboratory has been the subject of numerous experiments (Bossart et al., 2002). The most important parameters for the performance of a repository concerning the EDZ are its properties in the long-term perspective and not the properties during the construction and operational phase. Therefore, investigations have been carried out to evaluate the behaviour of the EDZ after waste emplacement. Significant differences in the long-term perspective compared to the construction and operational phase are that the near field will re-saturate with time. In the repository design considered for Opalinus Clay, the use of swelling backfill in the tunnels will also contribute to fracture closure within the EDZ. In contrast to hard rock, the EDZ in clay or mudstone will be strongly affected by these time dependent processes.

3.4.1 Mont Terri

In the framework of SELFRAC, two in-situ experiments (In-situ Experiments 1 & 2) have been performed at the Mont Terri underground rock laboratory. The first experiment, the so-called long-term plate load experiment, used the former EH-experiment site (Figure 11), where the reaction of the EDZ due to re-saturation had been investigated without applying any back pressure on the tunnel wall (Meier et al., 2002). For the second in situ experiment, the long-term dilatometer test, a new borehole (BSE-3) was drilled.

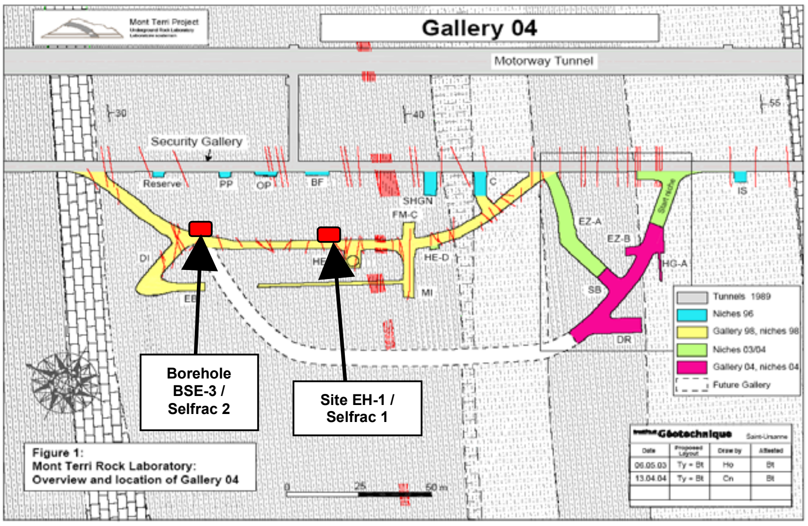


Figure 11: Experiment location at the Mont Terri underground rock laboratory

3.4.1.1 In-situ Experiment 1: Long-term plate load experiment

It was demonstrated that the increase of hydraulic conductivity of the EDZ in Opalinus Clay during the excavation of a tunnel is partly compensated during re-saturation by a sealing process (EH experiment reported by Meier et al. 2002). It was expected that loading would further reduce the hydraulic conductivity of the EDZ in a repository, therefore an additional mechanical load was applied on the tunnel wall by a plate loading apparatus to simulate the effect of the swelling of the buffer material (highly compacted bentonite) and/or creep processes of the host rock (Heitz et al., 2002).

The experiment had been equipped with 14 mechanical single packer systems and two single-point extensometers (BEH-54 and BEH-55). Figure 12 gives a schematic overview of the test area with the test boreholes equipped with mechanical packers and extensometers. The grey circle indicates the size and position of the mortar bed prepared for the installation of the load plate equipment.

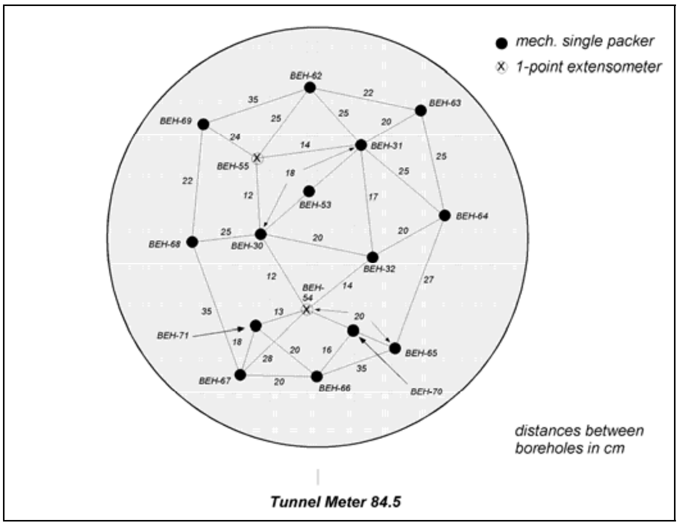


Figure 12: EH-1 test site with arrangement of the boreholes and the boundary of the mortar bed

The plate loading apparatus consisted of two end plates transferring the applied load to the tunnel wall, a central piece with three hydraulic jacks (133 tons each), a seal/pressure pad with a convex and a concave part, and a very massive steel beam used as an extension. Additionally, a load control device with two electrical pumps generating a maximum pressure of 60 MPa. The displacement of the two end plates was measured during the first load step as a relative displacement of the two end plates with respect to each other. Then it was decided to record the absolute displacement of each single plate with respect to the steel frames mounted to the tunnel walls outside the radius of influence. At the same time, two 4-point extensometers (BSE 5 and BSE 6) were installed in two new boreholes above the EH-1 site and a single-point extensometer (BSE 4) was installed at the side of the opposite plate. Pump pressure, displacement of the two plates, temperature at the site, and the shortening of the extensometers as a result of the applied load were continuously recorded. A picture of the experimental layout is shown in Figure 13.

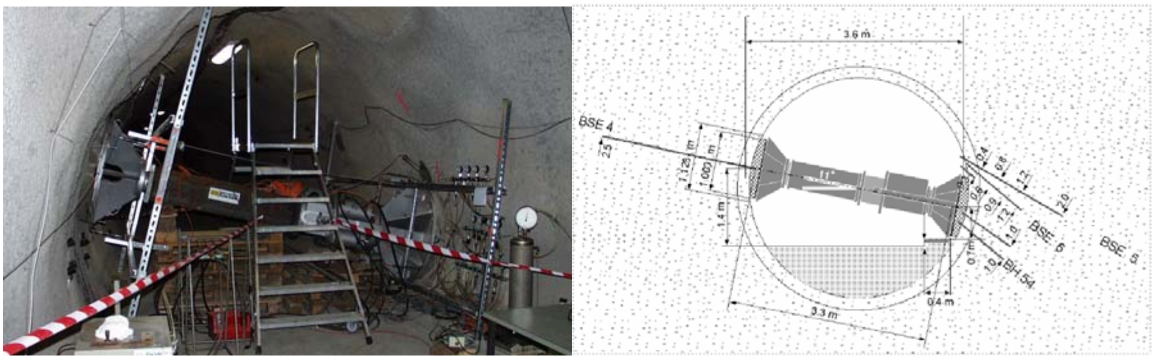


Figure 13: Experimental layout of loading equipment and the extensometers

The applied load has been increased to the tunnel wall in steps: 1, 2, 3 and 4.8MPa (see Figure 14). Several hydraulic tests were performed during the loading phases. The injection borehole was selected to be BEH-31 except for RI-test Nos. 14 and 15. RI-test No. 14 was performed in BEH-30 and RI-test No. 15 in BEH-32.

The mechanical response of the tunnel wall measured with different extensometers is represented in Figure 14. The total displacement of the plate at the test site measured with the longest extensometer is of the order of 0.8 mm. The displacement of the plate was not completely reversible. This is in agreement with the reduced hydraulic conductivity which was measured after load release (RI-test 19). The displacement measured with the two multiple-point extensometers BSE 5 and BSE6 show a clear correlation of displacement as a function of anchor depth. Steady-state conditions of the displacement could not be reached during the loading steps. In fact, this was not to be expected within the planned testing period based on long-term dilatometer experiments (Bühler, 2000a, b, 2003).

The hydraulic tests were analysed using the straight line approach (Cooper & Jacob, 1946). The obtained transmissivities correspond to a straight-line fit performed on the infinite radial acting flow period (IARF). The results of the tests of the EH-experiment (test No. 10) are based on similar analysis. Therefore, the results of both experiments can be directly compared. Recovery flow periods (RIS) were analysed using the Agarwal equivalent time (Agarwal, 1980). The test results of the hydraulic tests performed in borehole BEH-31 are presented in Figure 15 showing the derived transmissivity and hydraulic conductivity values vs. time in comparison with the applied load. (Note: Hydraulic conductivities values (k) are derived from the transmissivity values (T) assuming a homogeneous radial flow model. The conversion is made by k = T / L, where L = interval length.)

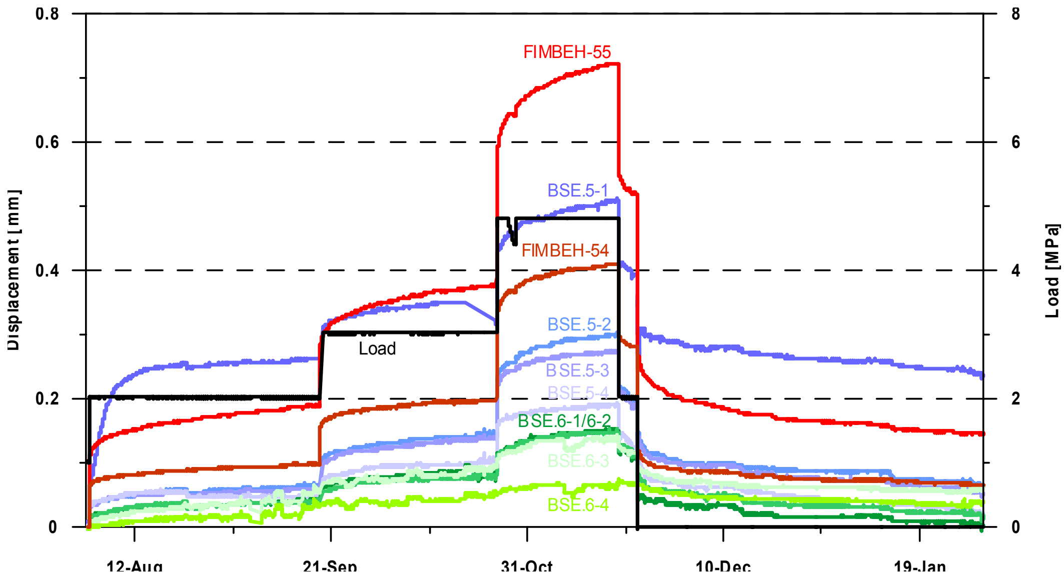


Figure 14: Displacement of the loaded surface measured with two single-point extensometers (FIMBEH-54/55) and two multiple-point extensometers BSE.5-1/4 and BSE.6-1/4)

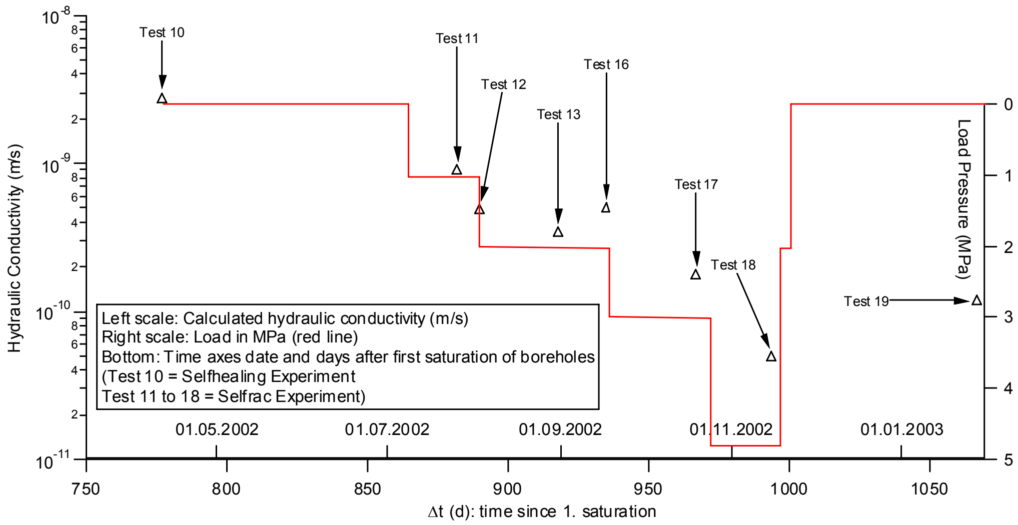


Figure 15: Hydraulic conductivity vs. time

The periodic hydraulic tests shows a significant reduction of the hydraulic conductivity (nearly two orders of magnitude) after the start of the loading test up to a maximum load of 4.8 MPa: 2.1 * 10^{-9} m/s (Test 10) $\Rightarrow$ 4.2 to 1.3 * 10^{-11} m/s (Test 18; the lower value corresponds to a late time fit of the test data). The reduction in hydraulic conductivity with time increases significantly in relationship to the previous EH-experiment and is also related to the applied load. The assumed sealing effect/process combined with a significant reduction in hydraulic conductivity has been proven with the performed pressure load test. This shows that a load applied to the tunnel wall leads to a partial closure of the fracture network in the EDZ, thus reducing the hydraulic conductivity in the near field. After the release of the load the hydraulic test No. 19 showed an increase of roughly about one order of magnitude in hydraulic conductivity under unloading conditions to a value of 1.0 - 1.5 10^{-10} m/s (Test 19).

3.4.1.2 In-situ Experiment 2: Long-term dilatometer experiment

For the second in situ experiment, it was planned to study the EDZ along a borehole. Instead of investigating the radial transmissivity of the EDZ, the second in situ experiment was focused at testing the axial transmissivity of the EDZ (Bühler, 2005). It was aimed to study the hydraulic-mechanic coupled effect of the EDZ along a borehole. A zone with higher transmissivity axial along the borehole, similar to the EDZ in the direct vicinity of the tunnel, is expected to be formed after the drilling of the borehole as a result of stress release and stress re-distribution. A different test setup and another test approach were chosen to achieve this goal.

The general concept of this experiment is based on experience from long-term dilatometer tests e.g. carried out in the IS-B experiment 1996 - 1997 (Bühler, 2000a) and in the DM experiment 1998-1999 (Bühler, 2000b) as well as from numerous hydraulic tests with multi-packer systems. In this experiment, the two techniques were combined for the first time. A dilatometer probe (packer length 1 m) is combined with two inflatable packers in a single test string (Figure 16) and then installed in the newly drilled borehole BSE-3 (12.10 m = 11.9 m Opalinus Clay and 0.2 m concrete) at the Mont Terri rock laboratory.

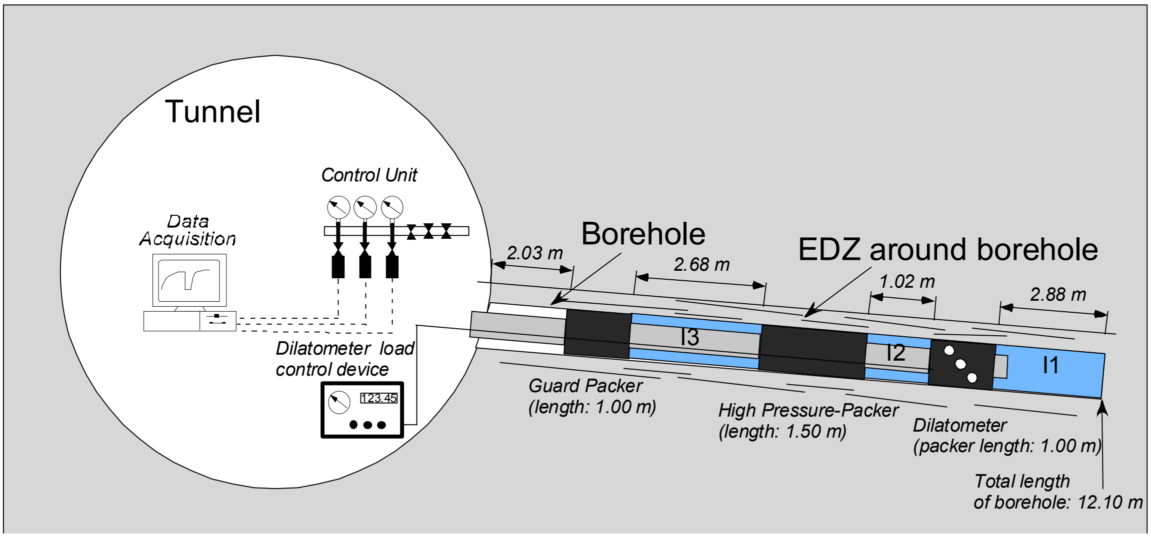


Figure 16: Experiment layout of long-term dilatometer experiment

The dilatometer probe was expanded with silicon oil. The load control device maintains a constant preset pressure over an extended period of time. It consists of a hydraulic pump, a control circuit and an oil reservoir. Pressures up to 60 MPa can be regulated by setting an upper and a lower pressure threshold value. A decrease of the pressure in the system below the selected threshold value causes the pump to operate, whereas an over-pressure is released by an electrical valve. The pressure sensor for pressure measurement in the dilatometer was placed directly at the coupling of the pressure hose with the load control device. It was connected to a multiplexer unit that interfaced the displacement transducers of the dilatometer probe with the data acquisition system. Both, the central packer and the top packer were expanded with water.

The pressure in the dilatometer probe was increased stepwise to about 3 MPa over a period of two months, and several pulse withdrawal tests with fluid production from interval 1 were carried out. At this time interval the pressure in interval 2 was almost the same as in interval 1, indicating that the dilatometer could not separate the two intervals hydraulically. An instantaneous pressure reaction in interval 2 on pressures changes in interval 1 proofed the existence of this connection in the first four tests. They were performed in the period of November 2003 to January 2004. The dilatometer pressure was increased from 1.6 to 2.7 MPa in this period. The instantaneous pressure reaction did not allow analyzing the transmissivity of these first tests.

Only when a dilatometer pressure of 3 MPa was reached, the pressures of both intervals started to separate with a higher value recorded in interval 1 (see Figure 17). It became even more evident after raising the dilatometer pressure to 3.5 MPa. With a dilatometer pressure of 4 MPa or more, the pressure in interval 1 had the tendency to increase, whereas the pressure in interval 2 gradually decreased. This indicates a hydraulic separation of the two intervals above and below the dilatometer.

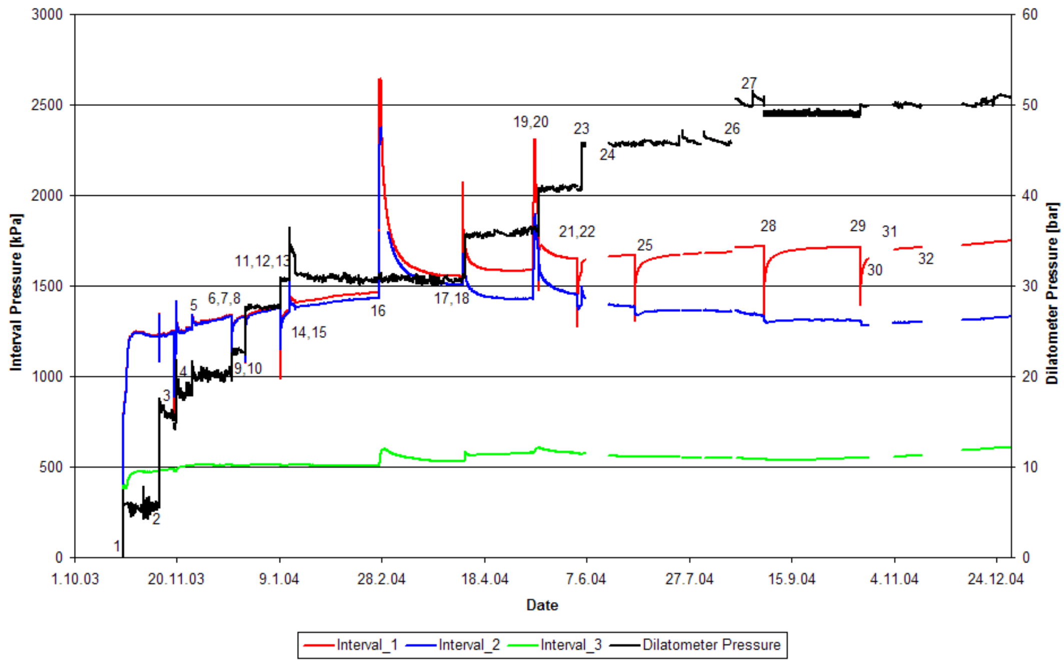


Figure 17: Overview of the project progress (note that he numbers correspond to the test events listed in Bühler 2005)

Fluctuations of dilatometer pressure in the first seven weeks of the experiment could be correlated with daily temperature changes of the tunnel atmosphere. The pressure changes were clearly smaller after thermal insulation of the dilatometer high pressure hose.

The pressure in interval 3 remained fairly constant. A distinct pressure increase was observed as a reaction to the first constant rate/head injection test. Smaller pressure responses were observed from the pulse injection test and the second constant rate injection test. No pressure reaction in interval 3 was observed from pulse withdrawal tests performed in interval 1.

Periodically hydraulic tests were performed under different dilatometer pressures. The pulse tests, notably the early ones, showed clearly a fast pressure reaction in the neighbouring interval between the dilatometer and the central packer. This led to the conclusion, that at least some water flows parallel to the borehole along the dilatometer and the basic assumption of the SELFRAC experiment of an excavation disturbed zone along the borehole appears to be valid.

After the injections test at the beginning of the experiment the following hydraulic tests were all performed as pulse withdrawal tests with fluid production from interval 1, to avoid hydraulic jacking of existing fractures. The results are compiled in Tab. 1.

Tab. 1 Results of hydraulic tests performed in interval 1

Test	Type of test	Transmissivity [m²/s]	Hydraulic conductivity [m/s]	Nominal dilatometer pressure [MPa]
RI/HI-Test-week09-04	Rate Injection/Head Injection Test	1.57E-11	5.46E-12	3
PI-Test-week15-04	Pulse Injection Test	1.80E-12	6.23E-13	3
RI-Test-week20-04	Rate Injection Test	2.86E-12	9.92E-13	3.5
PW-Test-week23-04	Pulse Withdrawal Test	4.62E-13	1.60E-13	4
PW-Test-week27-04	Pulse Withdrawal Test	2.02E-13	7.03E-14	4
PW-Test-week36-04	Pulse Withdrawal Test	1.59E-13	5.52E-14	5
PW-Test-week43-04	Pulse Withdrawal Test	4.70E-13	1.63E-13	5

A comparison of the transmissivity values reveals that the transmissivity measured in interval 1 decrease with the progress of the experiment. The decrease is related to the increasing load of dilatometer acting onto the borehole wall.

It is shown that an EDZ with a distinct higher transmissivity than surrounding the rock mass develops in short time around a borehole and that this transmissivity can be reduced as a function of the stepwise increase of the inflation pressure of the dilatometer from 0.5 to 5 MPa. The observed hydraulic properties of the EDZ around the borehole appear to be controlled by the load applied to the borehole wall. When the load applied to the borehole wall reaches 5 MPa, a reduction of the transmissivity of several orders of magnitude to about E-13 m^2/s of the axial flow path along the dilatometer is observed. Another effect is the delay of the pressure pulse in the neighbouring interval as a reaction to a pulse test in interval 1 (Bühler, 2005). It could be demonstrated that the delay of this pressure reaction in interval 2 depends on the applied load onto the borehole wall. An increased load delays the pressure reaction in the adjacent test interval.

3.4.2 HADES

In the framework of SELFRAC, two in-situ experiments (In-situ Experiments 3 & 4) were conducted at HADES URF (Mol, Belgium). The location of the experiments in HADES underground laboratory is given Figure 18. In situ experiment 3 aimed at studying the hydro-mechanical properties of Boom Clay around a freshly excavated gallery and their evolution with time. In situ experiment 4 studied the influence of borehole drilling and closure (collapse) on the host rock, using seismic and acoustic emission measurements. Full description of the experimental set-ups and results can be found in Frieg et al. 2006.

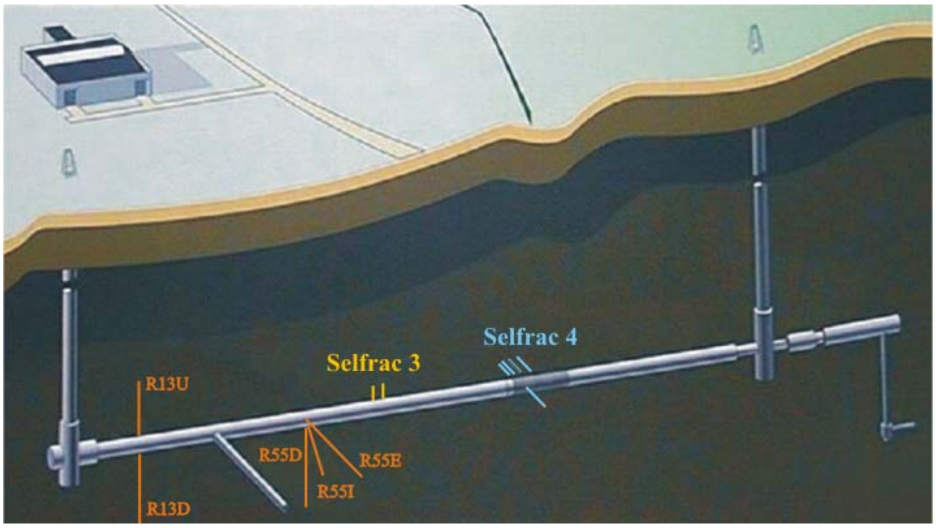


Figure 18: Experiments location at the HADES underground laboratory

3.4.2.1 In-situ Experiment 3: HM evolution around a repository scale drift

At about the same time as the start of the SELFRAC project, the HADES URF was extended with a new gallery (the *connecting gallery*), which was the first gallery ever to be constructed in a plastic clay at this depth using industrial techniques. The connecting gallery was finished in April 2002 and is roughly oriented North-South. Most of the observations described in this chapter were made in and around this gallery.

During the subsequent construction phases of the HADES URF, the presence of fractures was investigated. Especially during the last extension a lot of effort was put in to the characterisation of excavation induced fractures (Bastiaens et al., 2003; Mertens et al., 2004). A consistent fracturation pattern was recognised and a vertical cross-section is shown on Figure 19. The pattern consisted of two conjugated fracture planes. Dips were about 60° at mid-height of the gallery; at the fracture tip, dips could be as low as 30°. The distance between successive fracture planes usually was a few decimetres.

The shape of the fractures can be explained by the stress redistribution ahead of the excavation face, taking into account the in situ stress state and more specifically K_0 (~ 0.9). Furthermore, the presence of bedding planes may have influenced fracture orientation. Modelling and observations showed that fractures originated about 6-8 m ahead of the excavation face. Examination of clay cores indicated a radial extent of 1 m at most; the extent determined on vertical cores was somewhat lower than on horizontal or inclined cores. Field observations indicated fracture sealing but only limited fracture healing.

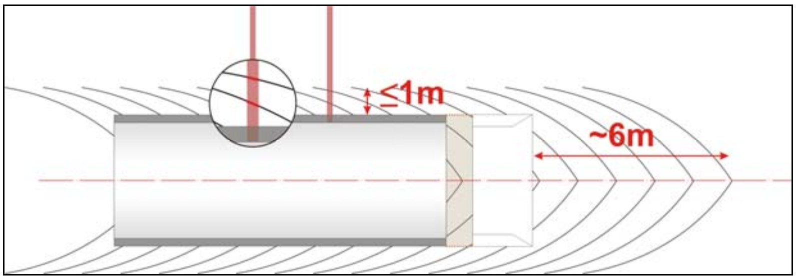


Figure 19: Vertical cross-section of observed fractures around the connecting gallery; their radial extent is up to 1 m and the fractures originate about 6 m ahead of the excavation face

Two identical multi-piezometers were installed vertically upward, with a distance of 1 m between them. For experimental reasons, the design of these piezometers was very specific and as a consequence the results are rather qualitative than quantitative. To examine the interaction between the two piezometers, a pressure change was applied at one filter and the response of the others was observed. In general, the response of filters on the same piezometer was immediate and strong and the response of filters on the other piezometer consisted of two components: a small, immediate reaction and a larger but postponed reaction. We could conclude that a strong hydraulic interaction exists between the two piezometers (at a distance of ~ 1 m). However there is no interconnected fracture network between them (at least at the tested intervals) as the reaction from one piezometer to the other is delayed. Moreover, the response is different from the imposed input.

In order to follow the evolution of pore pressure around the connecting gallery a network of 5 so-called reference piezometers was installed around the gallery, divided in two sections (Figure 20). Furthermore, two piezometers from the EC CLIPEX project were still available (Bernier et al., 2002): one above the connecting gallery (C2) and one next to the gallery (D2). Measurement results on 6th December 2004 (about 2 years and 9 months after gallery construction) of these and the SELFRAC piezometers (S62U and S63U) are shown on Figure 20; to make a meaningful comparison between the measurements at different locations, pore pressure is expressed as a percentage of the original undisturbed value at each location.

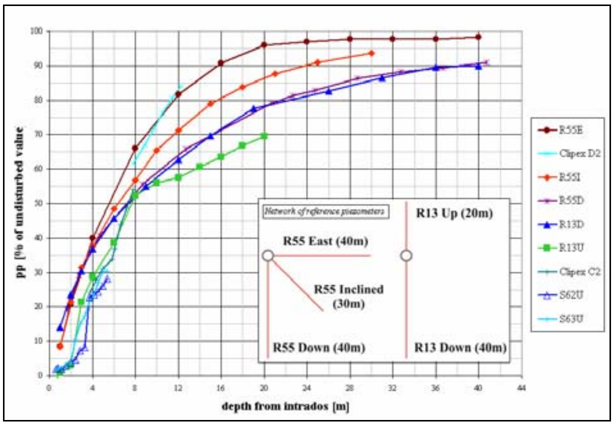


Figure 20: Overview of pore pressure measurements around the connecting gallery on 6 December 2004, expressed as a percentage of the original undisturbed value at each filter location. The layout of the two sections of the piezometric reference network is shown as well

It is clear that the pore pressure distribution around the connecting gallery is anisotropic; this can be explained by stress redistribution due to gallery excavation. Because $K_0 \sim 0.9$, the response of the host rock upon excavation will be different in a vertical plane than in a horizontal one. Mean stress will be higher in a horizontal plane than in a vertical one. Due to the low hydraulic conductivity, the immediate response of the host rock can be considered as undrained. As a consequence of the undrained response on the stress redistribution caused by gallery construction, pore pressures increase left and right of the gallery, and decrease above and below it. Re-equilibrium of pore pressures (dissipation of over- and underpressures) takes several years and depends mainly on hydraulic diffusivity. On this time scale (several years), the influence of hydraulic conductivity (cf. $k_H \sim 2 * k_V$) becomes important: the drainage by the gallery is larger horizontally than vertically.

Pore pressure could be measured as little as 40 cm into the host rock. This indicates that any fracture network existing beyond (at most) a few decimetres into the host rock is sealed. Moreover since no packers are used, the filters are sealed off by natural convergence of the borehole walls around the instrument.

Hydraulic conductivity around the connecting gallery was derived from constant head tests in steady state flow regime, Darcy's law was applied. Most measurements were performed on piezometers R55E and R55D. The results are shown on Figure 21. An increase of hydraulic conductivity is observed up to about 6-8 m into the host rock. The values outside this influenced zone are ~ 6.10^{-12} m/s for the vertical piezometer and ~ 4.10^{-12} m/s for the horizontal piezometer. About 1 year after the first measuring campaign on R55D, the first 5 filters were tested again. The obtained values were systematically lower, although not much. The first four filters of R55E were tested again as well; the obtained values were almost identical to those obtained one year before (slightly lower in the first two filters).

The values outside the influenced zone are consistent with in situ data obtained in previous experiments (De Cannière et al., 1994), although slightly higher. When measuring on a vertical piezometer, k_H is dominant; when measuring on a horizontal piezometer, k_H and k_V are more or less equally important. This explains the larger values of k obtained from the vertical piezometer. It is important to notice that even at the measuring points closest to the gallery; k is still only one order of magnitude larger than the undisturbed value.

Based upon these results and literature (Volckaert et al., 1995; Coll, 2005; Ortiz and Van Geet, 2005), we conclude that effective stress variation alone can account for the variation of hydraulic conductivity measured around the connecting gallery and that fractures do not play an important role in this case (cf. sealing); almost all measurements were performed further than 1 m into the host rock and thus well beyond the fractured zone.

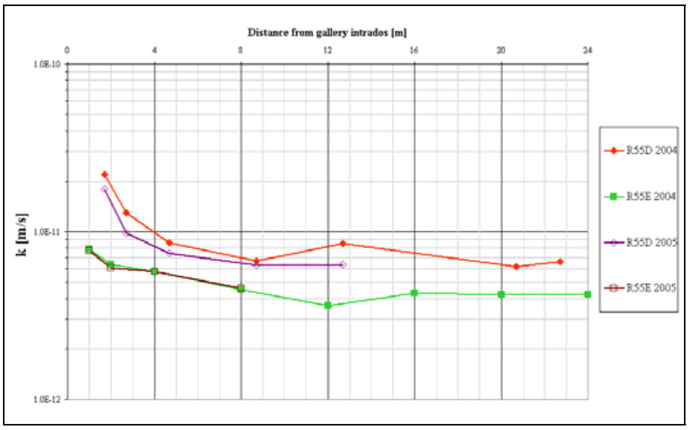


Figure 21: Hydraulic conductivity (k) around the connecting gallery (steady-state constant head tests)

In April 2002 and August 2004, two series of self-boring pressure-meter tests were carried out by *Cambridge in situ*. The connecting gallery was completed in early 2002, so the first series was carried out shortly after its completion. The equipment, personnel, drilling orientation and methods were the same for both boreholes; hence the only major difference between the two series of tests is the elapsed time since the construction of the gallery.

Results showed that total stress is influenced 6-8 m into the host rock and close to the gallery (Figure 22: Best estimate of total (vertical) stress based upon two series of self-boring pressure-meter tests conducted from the connecting gallery in 2002 and 2004); it increased during the interval between the two tests. Material parameters (e.g. shear strength and shear modulus) were only slightly influenced and restricted to a limited zone around the gallery (2-3 m); no significant change with time was observed. A value of 0.9 was determined for K_0.

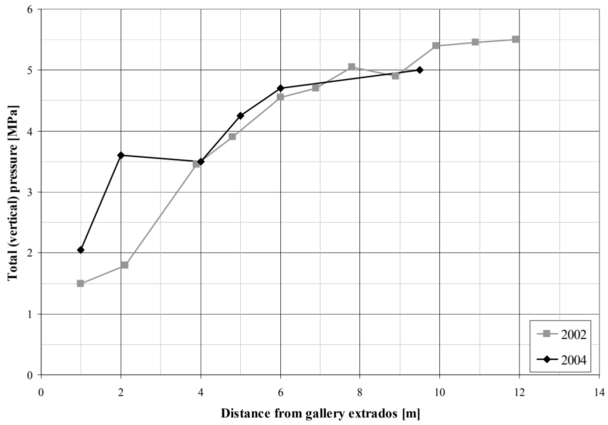


Figure 22: Best estimate of total (vertical) stress based upon two series of self-boring pressure-meter tests conducted from the connecting gallery in 2002 and 2004

We can conclude from in-situ test 3 that gallery construction in Boom Clay at an important depth (223 m in the case of the URF HADES) will always induce fractures. Fracturation is caused by stress redistribution which is inherent in tunnelling. Fracture extent can however be limited by using appropriate excavation and lining techniques. It was observed that these fractures seal but only heal partially.

In-situ test 3 has shown that no interconnected fracture network exists beyond (at most) a few decimetres into the host rock. Furthermore, hydraulic conductivity around the gallery seems not to be influenced by fractures. Higher k-values (up to about a factor 2) are observed up to 6-8 m and are probably caused by the lower level of effective stress in that zone. With time, k tends to become lower (when measured on a vertical piezometer) although further measurements are needed to confirm this. Unlike k, pore pressure is influenced up to an important extent by the presence of the gallery. The anisotropic aspect of pore pressure distribution can be explained by stress anisotropy (on the shorter term) and by anisotropic hydraulic conductivity of the host rock (on the longer term). Material parameters are only influenced slightly and up to a limited extent (2-3 m).

3.4.2.2 In-situ Experiment 4: Seismic survey of the closure of a borehole

This test was designed and interpreted by *GMuG (Gesellschaft für Materialprüfung und Geophysik)* in close association with EIG EURIDICE. The measuring system was developed for combined high-frequency seismic and acoustic emission measurements. Both receivers and transmitters used a piezoelectric element, embedded in cylindrical brass housing. Besides the main experiment which studied seismic parameters around an uncased (collapsing) borehole, a second one studied seismic parameters on a reinstalled (fractured) clay core.

3.4.2.2.1 Main experiment: collapsing borehole

The set-up consisted of four observation boreholes located around one central (collapsing) borehole. Each observation hole was equipped with three transmitters and two receivers at depths between 5.4 m and 7.6 m. In the central borehole four transmitters and four receivers were located (between 5.1 m and 7.9 m). Displacement sensors monitored the vertical and horizontal closure of the collapsing borehole. Figure 23a shows the lay-out of the experiment.

In a first phase, the four observation boreholes were installed (Dec. 2003) and the automatic measurements were started. After a stabilisation period, the central hole was drilled (July 2004). The measurements were evaluated with regard to seismic velocity and seismic amplitude of the P-waves for various travel paths in different directions. Figure 23b shows a typical result; transmission measurements on an oblique travel path between two observation boreholes are shown. A measurement gap is visible, during this time the system was updated. After the gap, some measurements were made and then the central hole was drilled.

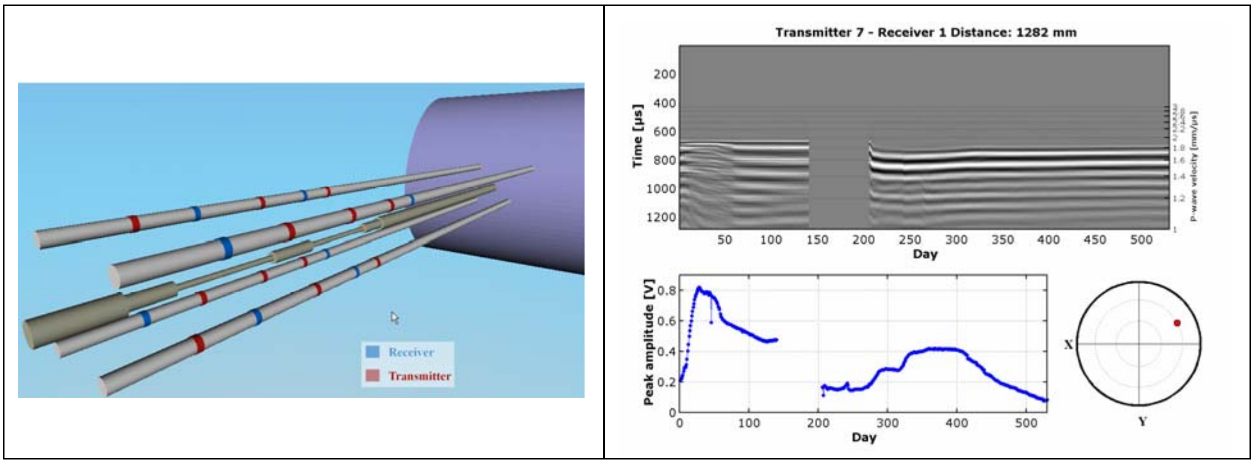


Figure 23: [a, left] Borehole layout in a three-dimensional projection showing transmitter (red marks) and receiver (blue marks) arrangement in observation boreholes. [b, right] Measurements between two observation boreholes (oblique travel path): the upper half shows the time history composed of the normalised waveforms on all days between 6 December 2003 and 23 May 2005. P-wave peak amplitudes are shown at the left-hand side at the bottom. The direction of the travel path is plotted on an equal-area, lower hemisphere projection (right-hand side at bottom)

In general, the waveforms of the transmission measurements between the observation boreholes (e.g. Figure 23b) showed a diffuse pattern of low frequency components at the beginning, this was accompanied by low peak amplitudes. In the first 30 to 50 days the peak amplitudes increased and the waveforms became more similar and a regular pattern appeared. This behaviour is attributed to the re-equilibrium of the host rock around the instruments and the sealing/healing of damage induced by the borehole drilling. Afterwards, amplitudes decreased in most of the travel paths. This phenomenon is not yet fully understood, one hypothesis is that this effect is linked to the evolution of effective stress at the interface between the instrument and the host rock. The drilling of the central hole did not have an important immediate effect on the amplitudes. The velocity however decreased following the drilling and increased again after some months.

The observations on travel paths with one sensor in the collapsing borehole were similar to those described above: diffuse pattern of low frequency components at the beginning of the measurements combined with increasing peak amplitudes. However, whereas the stabilisation period lasted 30-50 days in the case of a (lined) observation hole, it lasted some 100-150 days for the collapsing borehole. This is logical since more damage is allowed and it occurs over a longer period of time.

Few locatable AE events were recorded: only 38 over a period of 1.5 year but they confirm the results of the transmission measurements.

3.4.2.2.2 Extra experiment: instrumented core

Parallel to the main experiment, an extra experiment was carried out. During drilling of the collapsing borehole, a clay core was taken. A 0.567 m long section with two flat drilling induced fractures was selected. Its front and end were equipped with a seismic receiver and transmitter and 4 pore pressure cells were installed, two on each fracture plane (Figure 24a, b). After installation, transmission measurements were performed each night. Diffuse waveforms and low amplitudes were observed at first. After 38 days the velocity and frequencies suddenly increased. The amplitudes clearly started to increase at the same time and reached a maximum approximately 3 weeks later. After that point the seismic amplitudes continuously decrease to small values (Figure 24c).

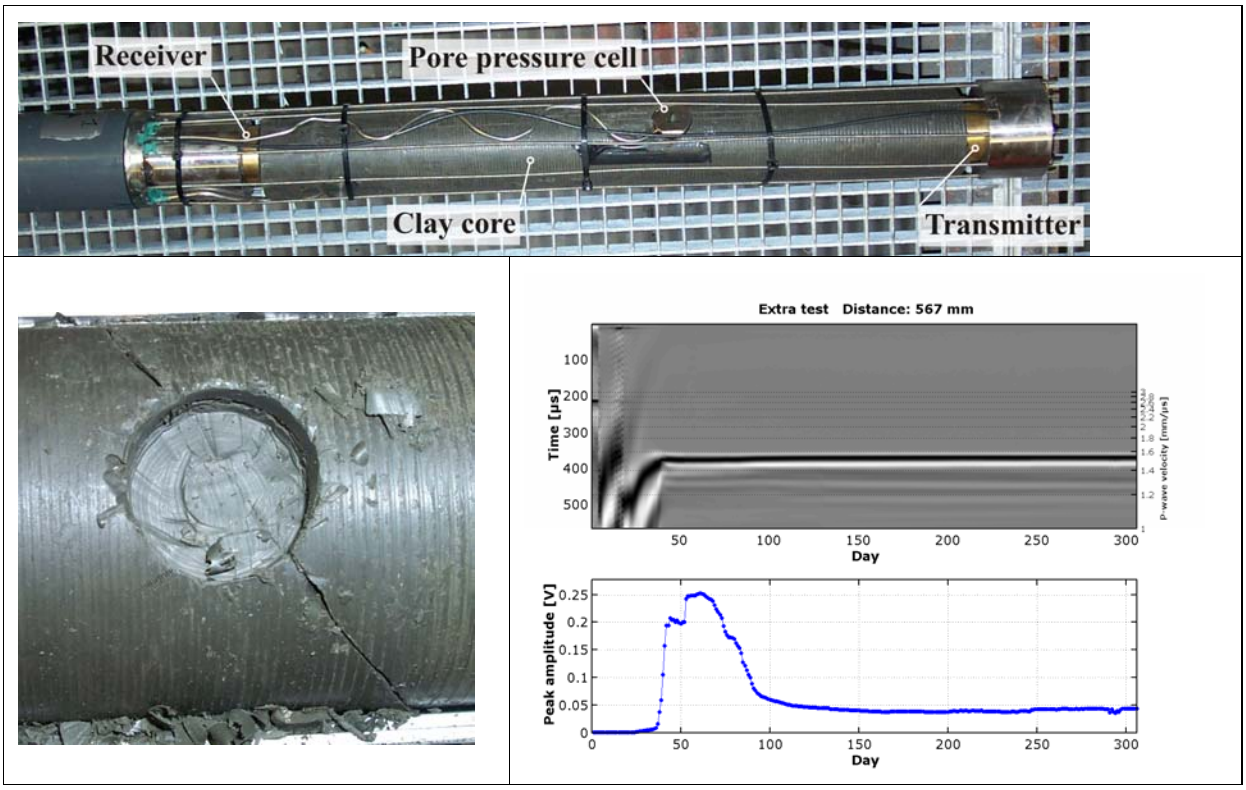


Figure 24: [a, top] Photo of the fractured clay core with transmitter, receiver, and pore pressure cells. [b, bottom left] Top view of the core at a pore pressure sensor location; a fracture is clearly visible. [c, bottom right] Signals of seismic measurements and corresponding P-wave peak amplitudes in axial direction through the Boom Clay core (between 1 July 2004 and 23 May 2005)

The amplitude evolution is similar to that of the main experiment. The sudden change of the seismic parameters can be explained by the assumption that the acoustic transmissivity in axial direction of the clay sample improved due to convergence of the borehole walls around the core and closure of the macroscopic fractures in the core. This sealing was tested by applying a pore pressure drop at one of the pore pressure sensors: although a fracture plane connected this sensor to another one, there was no direct hydraulic connection between them.

We can conclude from in-situ test 4 that long-term seismic and AE measurements are able to reveal disturbances and detect the evolution of sealing/healing processes with time in Boom Clay by means of measuring the associated variations in seismic velocities, damping of amplitudes and frequencies of the seismic signals. Collapsing of an uncased borehole entails a decompression of the surrounding clay which leads to an increase of deviatoric stresses and a decrease of pore water pressure. Both effects can lead to a decrease of seismic velocities and amplitudes which was observed in the present seismic measurements.

After the decompression phase, the stresses as well as the pore pressure tend to recover. Due to on-going reconsolidation, the acoustic transmissibility of seismic waves increases again in Boom Clay. The increasing trend could be observed rather in the seismic amplitudes than seismic velocities. Experiments on a reinstalled fractured clay core demonstrated that seismic parameters recover and fractures seal within several weeks or months.

3.4.3 Conclusions

The in-situ experiments performed in Opalinus Clay show that the effective hydraulic conductivity of the EDZ is expected to be relatively quick, within several years, lower than 10^{-10} m/s as soon as the bentonite backfill of the emplacement drifts becomes fully saturated and the expected swelling pressure will be build up.

The in situ experiments in Boom Clay have allowed to follow the evolution with time of the hydro-mechanical behaviour of Boom Clay around a gallery excavated by industrial technique and to quantify the effect of the sealing processes on the hydraulic conductivity evolution. The radial extent of the fracture zone around the gallery is about 1 m. However a slight increase of the hydraulic conductivity was measured up to 6-8 m into the host-rock. The results confirm that Boom Clay has important sealing properties. It was shown that two years after the excavation, the interconnected fractures zone was reduced from 1m to less than 60 cm around the gallery. The hydraulic conductivity in the sealed zone and beyond in the host-rock remains lower than 2.5 10^{-11}m/s.

It can be expected, for both Opalinus Clay and Boom Clay, that in the long term, the hydraulic conductivity of the EDZ will still decrease. It can therefore be assessed that the maximum hydraulic conductivity of the EDZ will be approximately one order of magnitude higher than that value of intact clays, which is in the case of Mont Terri in the overall range of 2 10^{-12} to 2 10^{-14} m/s (Heitzmann, 2004) and which is in the case of Boom Clay in the overall range of 2 10^{-12} to 4 10^{-12} m/s. Performance assessments models for different repository designs in different clay formations have demonstrated that, even for significantly higher hydraulic conductivity of the EDZ than the expected ones, the overall performance of the repository system would not be adversely affected and that radiation doses remain well below regulatory guidelines.

3.5 Modelling

One important objective of the SELFRAC project consisted in developing new constitutive and numerical tools that should give insight in the processes controlling fracturing (Process A) and sealing and/or healing of fractures (Process B).

Important efforts have been devoted to the constitutive modelling developments in the course of the SELFRAC project. The developments addressed theoretical and numerical aspects to simulate and /or predict both processes.

The involved teams are from L3S, LMS/G3S, LMR-EPFL and KUL.

Relying on the long time rich experience (from both experimental and numerical point of views) on the subject of strain localisation in Geomaterials (*the onset and development of Shear Banding, Cracking and Fracturation),* the team L3S has concentrated on the constitutive and numerical approaches development to predict the onset of strain localization in the context of hydromechanical coupling, namely Process A (§ 3.5.1).

On the contrary, LMS/G3S focused their effort on the constitutive law development dealing with the potential sealing/healing of materials, thus Process B (§ 3.5.2).

The modelling work performed by LMR-EPFL mainly aimed at interpreting the LEGEP experimental results to better understand and quantify the measured changes in permeability. For this purpose, Finite Element calculations of this test were performed (§ 3.5.3, Malinsky L. et al., 2006).

At KUL, the modelling work was performed to investigate whether it is possible to predict and quantify sealing/healing of a fracture from AE technique. In this way, the main objective of the modelling is, as at LMR-EPFL, to help the interpretation of the laboratory results (§ 3.5.3, Malinsky L. et al., 2006).

The developed/calibrated constitutive laws as well as corresponding numerical tools were afterwards applied to the modelling of the excavation of the connecting gallery at Mol (§ 3.5.4).

3.5.1 Development for the fracturing process modelling (Process A)

The review of the state-of-the-art on fracturing and their characterisations (Coll et al., 2006) revealed that the onset of failure and the mechanisms of propagation of discontinuities in a clayey material like the Boom Clay and the Opalinus Clay can be studied using both fracture mechanics and the bifurcation approach (shear band analysis).

The experimental studies described in Vervoort et al., 2006 have shown that strain localization is a major aspect of deformation processes in the two studied materials. This confirms the in situ observations. As overall deformation progresses in triaxial specimens under the increase of the deviatoric loading, at some loading level, one observes that deformation becomes *localised* i.e. gets concentrated in narrow zones called shear bands. Further loading after a while leads shear bands to turn into so called “strong discontinuities”, in other words cracks (in mode I or mode II, or combined).

In the frame of the SELFRAC project, the prediction of the onset of localisation was studied using the shear banding analysis, which is a bifurcation approach. For this, the hypoplastic CLoE constitutive model equipped with its fully non-linear bifurcation criterion, developed by L3S, is chosen and calibrated on the basis of the laboratory tests results. Moreover, the strain localisation analysis has been extended to a coupled hydromechanical context during the SELFRAC project. Indeed, a fully Hydromechanical coupled approach was necessary in order to take into account the situations of partial drainage encountered when strain localisation develops during laboratory tests or in the field.

The transition from shear bands to cracks is however a difficult matter to deal with as far as modelling is concerned, it needs to perform the so-called post-bifurcation analysis, which is still at the frontier of today's modelling know-how. Indeed, the strain localisation leads to the loss of uniqueness of the solution and mesh dependency for a Boundary Value Problem. Objective computations of the development of localised deformation in the post bifurcation regime call for further constitutive and numerical improvements. A part of the SELFRAC project consisted in developing new constitutive and numerical concepts in this post-bifurcation framework. As a response for this need, a second gradient theory has been developed and extended to the Hydro-mechanical coupling background and was successfully implemented in the Finite Element code (Lagamine) in the framework of the SELFRAC project.

3.5.1.1 Shear-banding analysis for CLoE model

In the theoretical framework of shear banding analysis, only the onset of shear banding is described. The result of the analysis is a local condition so-called **bifurcation criterion** which can be checked at any point in the problem. The criterion is met as soon as there is a real solution to a polynomial equation whose unknown is the orientation of the shear band with respect to the major principal stress direction, and the coefficients depend only on the constitutive equation.

CLoE model, is a constitutive model of the class of Hypoplastic models, whose main characteristics are to be an incrementally thoroughly non-linear model, not to make use of a decomposition of the strain rate into an elastic and a plastic part, and to allow to derive an analytical bifurcation criterion for the onset of shear banding, despite the strong non-linearity of the model. As described in Malinsky L. et al., 2006, the shear moduli used in the model can be calibrated using the experimental observations of shear bands in laboratory tests. More detailled information concerning the CLoE model can be traced through the Malinsky L. et al., 2006.

The parameters of the CLoE model can be calibrated from the laboratory tests. In the framework of SELFRAC, *a completely new interactive procedure* for calibration of the parameters on the pre-localization part of the tests has been built. Furthermore, in relation with the specificities of the Boom Clay and the Opalinus Clay, two new members of the model have been elaborated and brought into the CLoE model family:

- One is *CLoE-Clay*, which is devoted to normally consolidated clays, and incorporates significant changes in the definition of the basic stress paths used for the interpolation in the model. The new version allows for calibrating independently bulk modulus in isotropic loading and unloading. The classic version allows only calibrating bulk modulus in isotropic loading, while the modulus in isotropic unloading is imposed.
- The second new member, code number *CLoE-B* for the moment, is a more fundamental change in the family, as it abandons the assumption of a linear dependency of all the constitutive moduli on the mean effective stress. Instead, a non-linear relation is adopted, which induces major changes in the theory. The benefit expected from these changes is to reduce the stiffness of the isotropic response, which consistently proved to be much stiffer than the material characteristics due to theoretical constraints linked to consistency at the isotropic state, and of course to the linear dependency hypothesis (the latter induces automatically a exponential response on the isotropic compression stress path). It's worth to note that these changes would allow improving the hydromechanical coupled response of the material.

The calibrated CLoE model, equipped with its bifurcation criterion, was applied to model some laboratory tests and to predict the onset of the strain localisation.

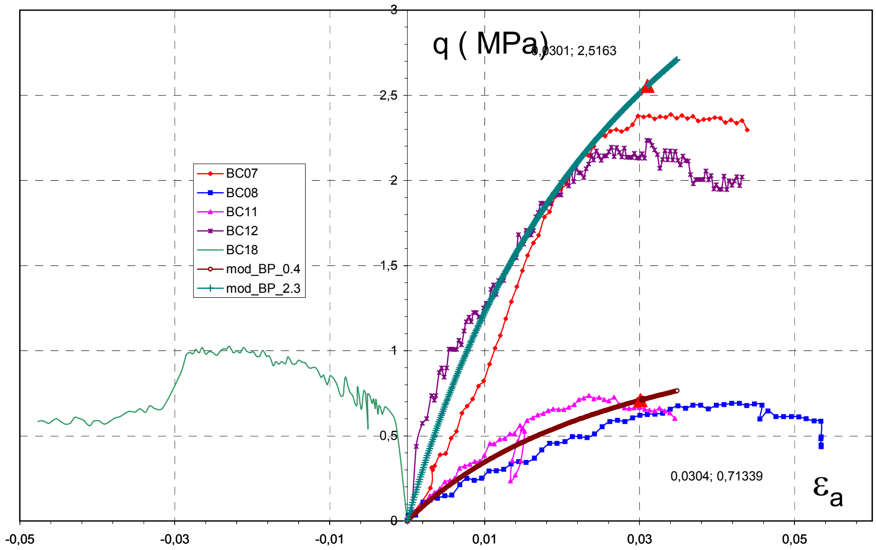


Figure 25: Boom Clay calibration on axisymmetric triaxial paths

The calibration for the Boom Clay is shown in Figure 25, which shows the comparison of the predicted stress-strain response with the experimental data, for two different values of the initial mean effective stress: 0.4 MPa and 2.3 MPa. For both levels of MES (Mean Effective Stress), the experimental database provides two repeated tests, which helps estimating the scatter of the experimental response. This scatter should be taken into account when assessing the model response versus the experimental data. The triangle at the end of the model response curve is the bifurcation point predicted by the bifurcation criterion; it indicates the onset of strain localisation in the test.

Figure 26 and Figure 27 present the calibration for the Opalinus Clay. Two sets of parameters have been elaborated, because it was difficult to obtain a satisfactory representation of the complete range of experimental responses recorded for different values of the mean effective stress. In Figure 26, the parameters are optimised to represent the low-pressure range, and it can be observed that the high pressure test is not so well modelled; in Figure 27 the high pressure range is better, with less good results for the lower pressure test. In both cases, the onset of strain localisation is properly modelled.

Numerical simulation results proved that CLoE model is able to model the response of the material on different stress paths, including strain localisation.

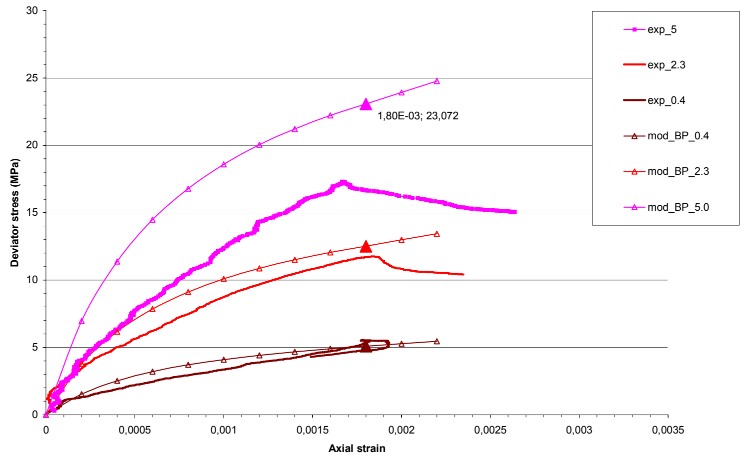


Figure 26: Opalinus Clay calibration on axisymmetric triaxial paths – low-pressure set

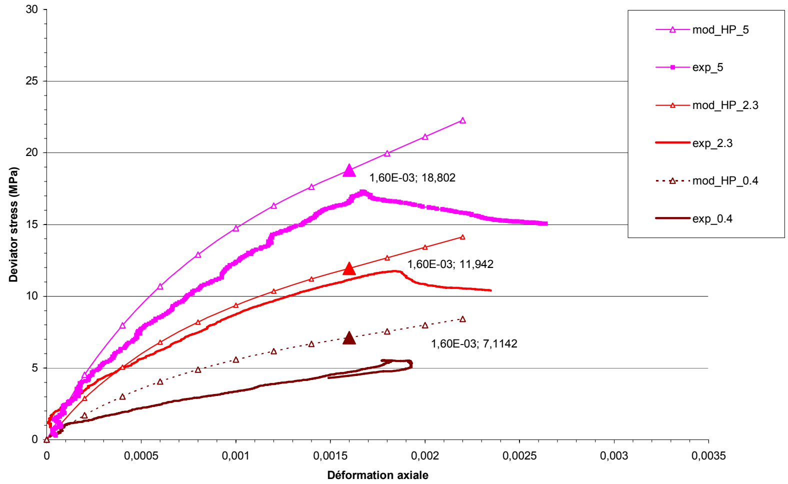


Figure 27: Opalinus Clay calibration on axisymmetric triaxial paths – high-pressure set

3.5.1.2 Strain localisation in HM-coupled analysis

Before SELFRAC, strain localisation in soils and rocks has been studied extensively in L3S for the last 20 years. A part of the experimental studies performed have been devoted to the response of specimens submitted to tests in undrained situation. It was shown that shear banding can take place in both contractive and dilative specimens, with some special features due to the coupling between the granular skeleton and the pore fluid. In SELFRAC project, strain localization was observed in both drained and undrained tests. Therefore, a study of the relevance of the bifurcation criterion in shear band mode in the case of hydro-mechanical coupling was performed. The relevance was assessed by numerical study of the response of the constitutive model in two kinds of a numerical integration: local, i.e. at the material point level and global, i.e. in boundary value problems analyzed by finite elements.

In current field problems involving argillaceous rocks like Boom Clay and Opalinus Clay, the situation of the material with respect to drainage is seldom perfectly drained or undrained: In quite a number of cases, partial drainage occurs, with some fluid flow taking place through the material, and simultaneously some pore pressure generation (possibly negative) developing. The latter is due to the restraint imposed by insufficient fluid flow to the pore volume change corresponding to the skeleton response to the (effective) stress. When the boundaries of the problem to be studied (e.g. stability of a gallery after excavation) are impervious (due to a lining for example), the problem can be said to be "globally undrained". Conversely, even if the boundaries are pervious, the time constants can be such that, in the short-term, no water flow can take place in the material anywhere: this case can be called "locally undrained".

As far as strain localisation is concerned, local drainage can take place (or not, depending on permeability and strain rate) between the incipient shear bands and the material close to the band, even if the problem is globally undrained. From a theoretical point of view, the question of the relevance of the localisation criterion has to be checked.

In the present state of affairs, when attempting to model complex problems like those we are dealing with in SELFRAC, it is worth checking if the observed behaviour of totally or partially undrained specimens can be properly captured by the hydro-mechanical coupled finite element formulation used, up to the onset of strain localization. In the sequel, a set of numerical simulations were performed to study the response of the **CLoE** constitutive model, equipped with its fully non-linear bifurcation criterion, implemented in the large strain finite element code Lagamine (University of Liège, GEOMAC department), in hydro-mechanical coupled simulations of totally and partially undrained biaxial experiments.

The main conclusions of this set of hydromechanical coupled simulations are the following:

The implementation of Hydro-Mechanical coupling in the FEM code Lagamine allows to reproduce the main features observed experimentally with respect to strain localisation in drained, undrained and partially undrained specimens subjected to biaxial tests in the laboratory. In particular, the resistance to localisation induced by pore pressure drop in dilatant specimens is properly modelled. Complex situations, like delayed drainage, can be studied numerically, and parametric studies can be performed to analyse and to design experimental programs.

3.5.1.3 Second gradient approach for post-bifurcation analysis

Geomaterials, especially those that exhibit some degradation in their strength and/or stiffness induce non objective computations, even if the computer code does not reveal any problem such as absence of convergence or discontinuity if we compare solutions of one step with the following. One way to clearly put forward such behaviour is to compute the same problem with two very different spatial grids: the solution is mesh dependent.

Some theories are known to be able to overcome this problem by incorporating an internal length. Amongst them the non local ones may be the most popular. However to the best of knowledge of the authors, there is no application of such theories to multiphasic materials like soft rocks or hard soils.

Another way is given by local theories involving some internal length. The most often used is the Cosserat theory which in fact has already been used in porous media (Ehlers and Volk 1998). However such a theory is well adapted for granular geomaterials such as sands, but is unable to properly regularise the post-localised regime of materials able to exhibit compaction bands.

The L3S started working on the local second gradient theory 10 years ago (Chambon et al. 1998). This theory is able to solve the problem of objectivity of computation in the post localised regime in any case (i.e. without restriction on the kind of localized bands encountered), and moreover as detailed in Malinsky L. et al., 2006), it was successfully applied to the poro-mechanics framework in the frame of SELFRAC project (Collin et al. 2005). The analysis is restricted to saturated soils or rocks filled by one compressible pore fluid. A coupled second gradient formulation has been proposed in the framework of microstructure continuum. A complete description of a large strain finite element coupled model using this local second gradient has been presented and first numerical results have been obtained for biaxial tests modelling. Numerical results show clearly the regularisation of the localisation pattern.

3.5.1.4 Conclusions

An important progress has been made in terms of the fracturing process modelling in the frame of the SELFRAC project. The strain localisation analysis, studied for more than 20 years for monophasic materials, was extended to a coupled hydromechanical analysis. The CLoE model implemented in the finite element code LAGAMINE has been calibrated for Boom Clay and Opalinus Clay on the basis of laboratory test results. Numerical simulations showed that such a theoretical framework is capable of fully describing at the local level and then predicting in FEM modelling of Boundary Value Problems (BVP) the HM behaviour of the material, including the occurrence of localised deformation and related local drainage phenomena.

Moreover, a local second gradient theory has been developed and extended to hydromechanical coupled porous media for the post-bifurcation analysis. This study is only a preliminary one and some further investigations are necessary in order to draw clearer conclusions on the influence of permeability. In the near future, more realistic constitutive models (non-associativity, yield function depending on mean stress, etc.) have to be used in coupled applications. It is however necessary to stress that these developments are necessary. There is presently no alternative solution to properly model those problems involving localised zones; i.e. the problems dealing with a possible rupture. Especially a complete study in order to predict the development of the localised zone with the influence of these zones on the seepage by a full coupled problem is of great interest for the future and this has to be able to predict the evolution of the EDZ.

3.5.2 Development for sealing/healing processes modelling (Process B)

The sealing/healing processes of the two studied materials have been extensively investigated deeply in laboratory and in situ in the framework of SELFRAC (see Vervoort et al. 2006 and Frieg et al. 2006).

In the case of Boom Clay, an artificial or induced fracture is sealed very fast as soon as submitted to a compressive state of stress. Some evidences of healing (restoration of the cohesion) are observed.

In the case of the Opalinus Clay a delayed sealing process is observed. Characteristic time of this time-dependent process certainly depends on the state of stress.

The literature review revealed that various models exist to describe the damage induced permeability variation for granite (Souley et al. 2001, Shao, et al. 1999). In addition a few healing models for salt are published (Stormont et el. 1992, Hou, 2003). Nevertheless, coupled constitutive modelling approaches dealing with the sealing/healing processes are still very limited.

A modelling framework has been proposed by LMS/G3S team in the frame work of SELFRAC project to model the time and stress dependent sealing of a previously damaged material.

3.5.2.1 Proposed sealing/healing constitutive model by LMS/G3S

The proposed model is an extension of the Hou model together with an extension of the Fauchet model to represent the sealing process. The strain tensor is split into an elastic part $\underline{\underline{\varepsilon}}^{e}$ and an irreversible part $\underline{\underline{\varepsilon}}^{irr}$ written as follows:

$$\underline{\underline{\varepsilon}}^{irr} = \underline{\underline{\varepsilon}}^{d} + \underline{\underline{\varepsilon}}^{h}$$

where $\underline{\underline{\varepsilon}}^{d}$ is the part of irreversible strain tensor modelling a damage responsible for the permeability increase. This mechanism can include for instance plasticity or visoplasticity. The second part of the irreversible strain tensor $\underline{\underline{\varepsilon}}^{h}$ is the sealing/healing strain tensor that is responsible for the permeability decrease. Several formulations of the sealing/healing strain rate have been studied and finally the following mathematical formulation is retained:

$$\overset{\bullet}{\varepsilon}{}_i^{h} = -\frac{1}{\eta_h}\left\langle -\left(\underline{\underline{\sigma}}'.\underline{n}_i\right).\underline{n}_i\right\rangle^{a}\left\langle \varepsilon_i^{irr}\right\rangle^{b}\left\langle -F^{d}\right\rangle^{c}$$

where ε_i^{irr} is the i^{th} Eigen value of the irreversible strain tensor, $\left(\underline{\underline{\sigma}}\underline{n}_i\right)\underline{n}_i$ is the normal stress in the direction of the i^{th} Eigen vector and F^{d} is the damage criterion (viscoplastic or plastic), η_h , a , b , c are positive constants to be determined from laboratory or in situ tests.

In this formulation, the sealing/healing strains can occur in the following conditions:

- the normal effective stress $\left(\underline{\underline{\sigma}}'\underline{n}_i\right)\underline{n}_i$ is compressive
- ε_i^{irr} is positive (cracks are opened)
- damage mechanism is not activated in the same time.

The minus sign insures that the sealing/healing mechanism tends to reduce the irreversible strain. Moreover, the larger is the compressive normal stress and the larger is the distance to the damage criterion, the faster is the sealing/healing.

The permeability is split in two parts:

$$\underline{\underline{k}} = \underline{\underline{k}}_0\left(\varepsilon^{e}\right) + \underline{\underline{\kappa}}\left(\underline{\underline{\varepsilon}}^{irr}\right)$$

The first term is that of the intact rock that can depend on the elastic strain or effective stress as shown by the laboratory experimental results and in situ testing.

For instance the LMS hollow cylinder permeability tests on Boom Clay lead to:

$K = k_0(1+a(\sigma'-\sigma_0'))$

with a = 0.42 $k_0 = 4 10^{-19} m^2$ $\sigma_0' = -4.5MPa$ (σ' negative in compression)

The second part is due to the irreversible strain.

This formulation allows modelling a delayed sealing when the stress dependent sealing mechanism is activated.

To conclude, theoretically, the proposed modelling framework is able to take into account most of the sealing or healing mechanisms identified during the project. These mechanisms are essentially the hydro-mechanical conditions leading to a change in the effective stress activating a time dependent sealing/healing mechanism. The irreversible strains that led to an increase in permeability can be reduced according to time when the effective stress is brought inside a sealing/healing domain. The permeability, depending on both irreversible and elastic strains, is decreased when the sealing/healing mechanism is activated. This formulation can naturally take into account the decrease in permeability associated with a contractant behaviour. The effect of clay swelling, identified as a contributory factor in the sealing and healing processes, can also be modelled if the mechanical boundaries are such that constrained swelling generates stresses favourable to the sealing or healing process.

However further efforts remain to be made:

- The proposed model is based on the theory of continuum mechanics. As pointed out, for Opalinus Clay a significant permeability variation is associated with a discontinuity at sample scale and not to a diffuse damage as observed for other rocks like salt or granite. The framework can be easily extended to the case of discrete discontinuities but since the fractures are discrete and taking into account the scale of the sample, the main difficulty is to identify from the laboratory data the behaviour of these fractures. This implies for example a good knowledge of the fracture geometry.
- In order to have some confidence in the modelling, the healing strain, assumed to be responsible for a delayed permeability decrease under a constant hydromechanical state, should be actually measured by an appropriate experimental device.
- A sufficiently large number of different hydromechanical loadings paths are needed to determine the introduced mathematical functions and parameters. At this time there is a lack of experimental data to achieve such a goal.
- Finally the approach is purely phenomenological and the microscopic phenomena are not yet well identified.

3.5.2.2 Conclusions

Based on the literature review and on the experimental results, a framework for the modelling of a time and stress dependent sealing process is proposed and discussed for the two studied clay rocks.

Such a model could be applied but is not indispensable for the Boom Clay in which sealing is almost instantaneous when the state of stress is compressive. In the case of the Boom Clay a classical poro-plastic or poro-viscoplastic contractant-dilatant model together with a porosity dependent permeability model is sufficient to account for the hydromechanically induced permeability variation in the compressive stress domain.

Conversely the proposed framework could be reliable for modelling the sealing process in Opalinus Clay provided that a single discontinuity or a well identified set of discontinuities are considered for the calibration of the model, but there is a lack of experimental data to achieve that goal.

The proposed constitutive model is promising to deal with both Process A and B. But further efforts are still needed for an application to a real-case modelling.

3.5.3 Modelling work supporting the interpretation of lab tests

3.5.3.1 Modelling work performed at LMR-EPFL

The modelling contribution of LMR-EPFL helped to interpret the LEGEP experimental results. Finite Element calculations with the Z_Soil program were performed in a two-dimensional axial-symmetric configuration, taking into account the differrennt steps of the experimental procedure and the hydro-mechanical coupling. For both Boom and Opalinus Clays, the FE computations provide results that match the laboratory flow measurements quite well. The increase in permeability after the tensile fracturing of Opalinus Clay samples is correctly modelled by means of a thin layer of more permeable elements. The plastic behaviour of Boom Clay, in comparison with the brittle behaviour of Opalinus Clay, could explain the small healing and the much faster sealing of fractures in Boom Clay when subjected to a normal stress close to the in-situ one.

3.5.3.2 Modelling work performed at KUL

Through the experimental program performed at K. U. Leuven, it is shown that the acoustic emission (AE) technique can be used to investigate at micro-scale damage and damage evolution, and hence also sealing and healing. The Kaiser effect in rocks is due to irreversible or partly reversible damage created by e.g. dislocations and microcracks (Holcomb, 1993). At the specimen's scale, the predominant mechanism of the Kaiser effect in brittle and semi-brittle rocks should be linked to microcrack growth. Sealing/healing of damage or of micro-fracturing should lead to a decrease in the Kaiser effect, expressed by a smaller Felicity ratio (Fr). In the experiments conducted at K. U. Leuven, a decrease in Felicity ratio (Fr) with increasing time delay between loading cycles was observed. This could be caused by sealing and/or healing processes. A similar trend is observed for both types of clay investigated.

A quantification of the different processes is at this time still difficult. This is due to the lack of direct observations of the processes at hand, through e.g. other experimental techniques or through modelling. Furthermore, a scale issue still remains. The acoustic technique is used here to study micro-fractures, and the upscaling of this effect is not necessarily straightforward.

The main question to be answered by the modelling is: if sealing/healing did occur, what would be the effect on the AE-activity in the model. Sealing/healing is introduced in the model by a shortening of the induced fractures or by an increase in their strength. Two approaches have been envisaged for the modelling: an analytical model based on the wing crack model and the boundary element code DIGS that allows explicit growth of micro-fractures (Malinsky L. et al., 2006). The various model parameters are determined based on calibration using uniaxial and triaxial loading tests. The clay strength properties have to be defined on micro-scale, but at the same time the simulation should result in a realistic macro value for e.g. UCS. The sealing/healing of a single crack is studied first, then the laboratory experiments are simulated. In the latter models, several micro-fractures occur.

The analytical model showed that a decrease in wing crack length leads to a sooner crack onset in a second cycle. Similar findings are reported using a boundary element code (DIGS). Here, a direct link to AE can be made (element activation = hit) and the influence of the different parameters can be studied. It is shown that sealing causes a slightly sooner onset of fracturing in a second cycle, but a Kaiser effect (clear inflection in the curve ‘Number of activated elements vs. Load’) at the same position. Complete sealing/healing of a certain percentage of fractures before a second cycle causes a change in shape of the curves, and a decrease in Kaiser effect, which is in agreement with experimental results. The influence of the various parameters (e.g.

cohesion) remains to be studied closer. A quantified direct link between modelling results and experimental results cannot be made at this time.

3.5.4 In-situ test simulations

Both LMS/G3S and L3S teams have focused their efforts on the modelling of the connecting gallery excavation process at Mol site (§ 3.4.2.1). In comparison to the numerical simulations realised in the frame of the EC CLIPEX project (Clay Instrumentation Programme for the Extension of an Underground Research Laboratory) (Bernier et al., 2002), the numerical simulations realised in the present SELFRAC project concern more aspects related to the strain localisation as well as the effects of the hydromechanical coupling in short and long-term (poro-viscoplastic analysis) on the hydraulic disturbance zone (HDZ), mainly the hydraulic diffusivity, which depends not only on the permeability but also on the deformability of the soil. Moreover, the effects of the anisotropy (in situ stress and permeability) have also been investigated numerically.

In LMS/G3S, numerical simulations have been carried out in order to assess the ability of poro-plastic and poro-viscoplastic models to predict the long-term hydromechanical disturbances in Boom Clay around the connecting gallery.

In L3S, the effort was put on two important points:

i) the modelling of the excavation process and

ii) the strain localisation prediction.

Detailed simulation results and corresponding analysis are given in Desrues et al., 2006.

3.5.4.1 Poro-ViscoPlastic analysis (LMS/G3S)

In LMS/G3S, one-dimensional simulations are first studied for a sensitivity analysis. In a second step, two dimensional simulations are performed to examine the effect of both initial in situ stress and permeability anisotropy on the hydraulic disturbance zone in Boom Clay around the connecting gallery.

The pore pressure distribution measured in piezometers and the total pressure evolution exerted on the lining inferred from strain measurements in the lining are compared to the numerical finite element simulations.

The constitutive laws used for the simulations are based on the poro-plastic and poro-viscoplastic models. The visco-plasticity behaviour of the Boom Clay is based on the work of Rousset (1988), while a type of the Mohr-Coulomb criterion was used for both viscoplastic yielding criterion and viscoplastic flow potential (non-associated viscoplasticity). The visco-plastic parameters are mainly identified from the laboratory tests performed by Rousset (1988), whereas the plastic parameters come from used parameters of Boom Clay for Mohr-Coulomb model.

1D sensibility simulations considered following aspects:

- Young’s modulus and Poisson ratio
- Instantaneous plastic behaviour
- Viscosity effect: stress dependency of viscosity parameters.

All these aspects are in fact related to the hydromechanical coupling effect, they influence essentially the hydraulic diffusivity (storability effect).

2D simulations aimed at investigating the anisotropy effects. Taking into account the initial anisotropic in situ stress with $\sigma_h = 0.9\sigma_v$ (σ_h and σ_v refer to horizontal and vertical stress respectively), the following permeability anisotropy cases are studied numerically:

- $k_h = k_v = 4.10^{-19}$ m^2
- $k_h = 2k_v = 8.10^{-19}$ m^2
- $k_v = 2k_h = 8.10^{-19}$ m^2

In all simulations, a special attention was paid to the hydraulic boundary condition at the gallery wall. Two different kinds of boundary condition have been used:

Unilateral boundary condition: No flow boundary condition until the pore pressure reaches the prescribed zero pore pressure at the gallery wall.

Q = 0 until P = 0, then P = 0

Convective boundary condition: The flux is proportional to the difference between the prescribed pore pressure and the computed pore pressure.

$Q = H(P-P_0)$ $\qquad$ $H = 10^{-4}$ m^3 MPa^{-1}s^{-1}

These two types of boundary conditions differ actually only at short term and when hydromechanical coupling is strong. Thus it plays a minor role when one is interested in the long-term behaviour.

As an example, Figure 28 presents the 1D simulation results considering different mechanical properties of the clay, on which a pure hydraulic calculation (so equivalent to rigid solid skeleton, infinite large modulus) is also put for comparison. The effect of the deformability of the soil has been put in evidence.

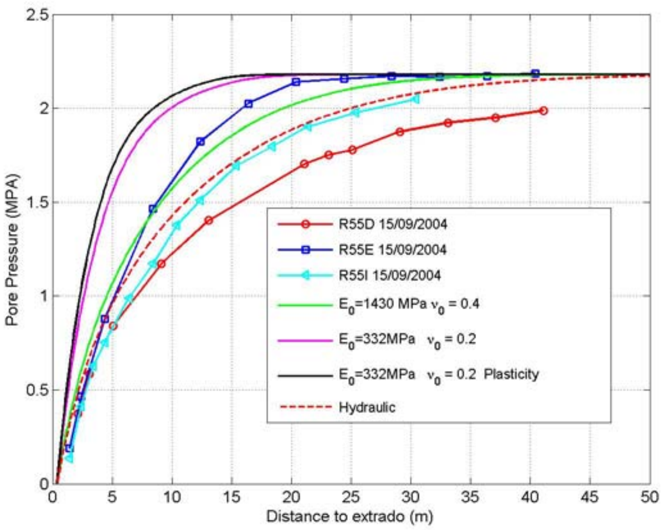


Figure 28: Comparison between the measured and simulated pore pressures 930 days after the excavation: influence of plasticity and elastic properties

The simulations realised in this project have put in evidence some important influencing factors with respect to the HDZ, even though not all conditions considered are realist. On the other hand, they revealed also the complexity of the hydromechancial behaviour of the Boom Clay. These simulations brought some light on the observations, but the anisotropy of the pore pressure distribution around the gallery is not fully understood.

In summary, the evolution of the total pressure exerted on the lining is quite well predicted by 2D simulations assuming an anisotropic initial state of stress if we account for material and geometrical parameters uncertainties. More questions remain concerning the prediction of the observed long-term pore pressure distribution, especially its anisotropy. Indeed the hydromechanical coupling phenomena induced by the initial in situ stress and the irreversible material behaviour could have been an explanation for that anisotropy but are efficient mainly at short term after the excavation whereas the long-term pressure field is mainly controlled by elastic properties and permeability. On the other hand the anisotropy in permeability as measured in laboratory tests cannot explain the pore pressure anisotropy.

3.5.4.2 Poro-hypoplastic analysis with progressive excavation and localisation analysis (L3S)

In L3S, the effort was put on two important points: i) the modelling of the excavation process and ii) the strain localization prediction. As for point i), a special modelling procedure was designed, taking into account the progressive excavation as a process in time, the placement of a lining behind the excavation front, the existence of an over-excavation with respect to the lining dimensions, and the hydromechanical coupling. Altogether, these features induce a time-dependent response, with unilateral contact condition to be considered as the clay comes in contact with the lining. An interesting point is the fact that, due to the deformation of the clay ahead of the front, grid points out of the excavation profile in the initial configuration may enter into the profile after the process has started; this real feature is taken into consideration in the modelling process, by determining the elements to be excavated on the basis of the actual position by the time the corresponding slice is processed. It is observed that a stationary state is reached after a while in the excavation process. Concerning the localisation prediction, the analysis was performed in every element all along the simulation, using a shear band analysis approach with a bifurcation criterion at the constitutive law level. The results indicate a localisation zone taking place at the tip of the excavation front, as was observed in situ during the excavation work.

Simulations were performed at L3S using the *FEM code Lagamine,* with the constitutive model CLoE. They provide among others pore pressure, stress and displacements evolution around the gallery and patterns of rupture.

The simulation results are compared with the experimental observations and with numerical results obtained in the framework of the *Clipex* EC project by other teams. Before these simulations, a series of numerical simulations of laboratory tests have been performed to check the performance of the integration of the constitutive law CLoE with the parameters determined for the studied materials (Malinsky L. et al., 2006).

Modelling hypotheses

The following main hypotheses have been considered:

- 2D - Axisymmetric conditions along the gallery axis
- Isotropic and homogeneous initial in situ stress and pore pressure state: variation with depth is neglected in the modelling zone
- A high permeability (1.10^{-12} m^2) is chosen for the lining in order to ensure drained conditions in the lining.

- The shafts are not considered
- The unilateral nature of the contact between the clay and the lining is modelled
- The excavation is supposed to be undrained, that is to say no water can flow through the boundaries.

Geometry

The modelling zone is corresponding to the region around the gallery and the head of the Test Drift (instrumented zone in the *Clipex* project). The modelling domain is 50 metres high (the gallery radius is 2.00 metres) and 90 metres long. The height excavated is 2.42 metres and the thickness of the concrete lining is 0.40 metre. The initial gap between the soil and the lining is 2 cm large.

A set of elements, meant to model the lining, is defined in the mesh, placed in the final position of the lining along the gallery after excavation. These elements have the mechanical properties representative of the lining (a concrete tube with young modulus = 15 GPa and Poisson Coefficient = 0.3), but they are activated only in due time in the modelling process. Before the activation, the lining elements do not play any role. The activation process is made by sub-sets corresponding to an excavation step.

Modelling procedure

Boom Clay is considered as a porous medium subjected to hydro-mechanical coupling. Two major stages are included in the modelling procedure of the excavation:

Stage1: stress release in the first 30 metres of the connecting gallery, starting from the second shaft, day 1. It is worth to note that a complementary study showed that 30 days instead of 1 day for stage 1 did not make significant changes in pore pressure and stress fields.

Stage 2: excavation of the gallery by 1-metre steps, up to reaching the Test Drift zone, with activation of the concrete lining simultaneously. The excavation speed is 2 m/day.

Main results

The simulation results (pore pressure, displacements, as well as the total stresses) are compared with the experimental observations and with numerical results obtained in the framework of the *Clipex* EC project by other teams. Generally, the present results are consistent with the site observations and the numerical results obtained in the frame of the Clipex project. In terms of pore pressure evolution and variation, the CLoE model with the set of calibrated parameters gives results, which are comparable to other classical models: it can't produce the large extended HDZ observed in – situ. The prediction of total stress evolution is quite consistent with a classical model. In addition, it seems to give larger displacements. The new member of the CLoE family, CLoE-B, may provide better results in terms of pore pressure as well as the displacement. As the attention of the simulations was put mainly on the process of strain localisation, also due to the limited time for the project, the efforts afterwards were concentrated on the bifurcation analysis during the excavation.

Indeed, with the own equipped bifurcation criterion, the CLoE model with actual parameters is able to catch the general feature of the strain localisation around the gallery.

Zones with potential strain localisation are detected behind the front over a length of 2.5 m in the axial direction. So-called "bifurcation crosses" illustrated in Figure 29 are displayed at the locations where the bifurcation criterion was found to be met. The green and the red arrows indicate the directions parallel to the two conjugate potential shear bands in this location. According to the results discussed in the constitutive package, these two directions are rather close, which is typical of brittle materials. In the numerical model used, no post-localisation strategy has been developed. The localisation prediction displayed is indicative of zones where rupture is likely to develop in a localised mode, nothing more.

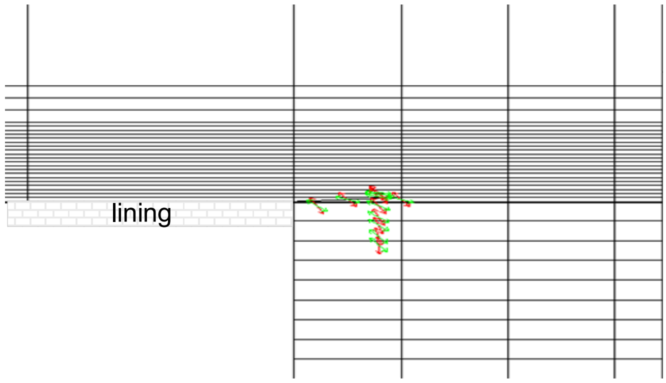


Figure 29: Localised zones due to stress release in the gallery (bifurcation crosses located at Gauss points)

The effect of different choices in modelling the installation of the lining, on the rupture mode, has been studied by performing two different simulations: in the first one, the lining is considered as instantaneously installed up to the actual position of the excavation front (Figure 30, top), while in the second there is a gap between the front and the lining (Figure 30, down), along which the soil is free to move. It can be seen that the localisation figure behind the front is not changed, but new localised zones appear along the non-supported wall: the process induces more localisation in the second case. A third computation with a finer mesh in the critical zone allows a more precise description of the localisation (Figure 30, right). It is worth to mention that no significant changes appear between these different simulations in the pore pressure and stress fields.

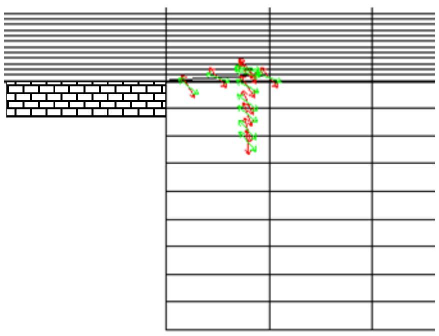
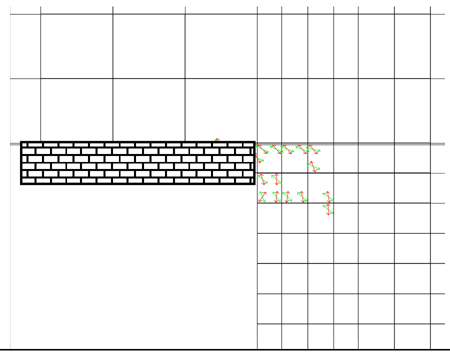
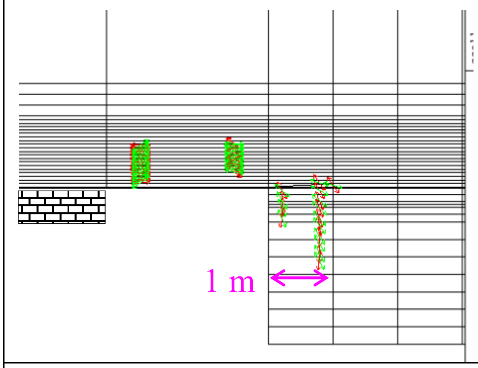
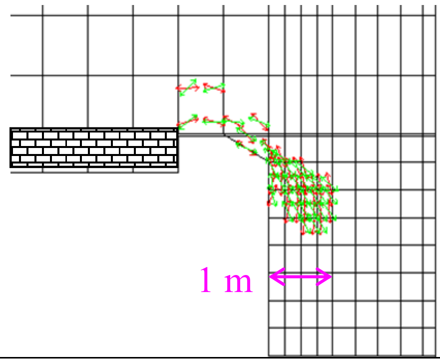
		The lining is placed up to the front
		There is a gap between liner and the front
Coarse mesh behind the front	Fine mesh behind the front	

Figure 30: Localised zones function of the mesh density and of the lining position

Conclusions

The modelling of the excavation process and the strain localisation prediction along the process have been performed using the finite strain Finite Element Code Lagamine, with the constitutive law CLoE. A special modelling procedure was designed, taking into account the progressive excavation as a process in time, the placement of a lining behind the excavation front, the existence of a over-excavation with respect to the lining dimensions, and the hydromechanical coupling. The results are consistent with the site observations, especially the localization are predicted. Further refinements would be necessary to reduce some discrepancies observed with respect to site data.

3.5.5 General conclusions

A lot of effort has been put on the modelling work during the SELFRAC project. On the one hand, new constitutive models and numerical tools were developed to model the fracturing and sealing/healing processes; on the other hand, numerical simulations have been used to help in interpreting and better understanding the laboratory tests.

An important progress has been made in terms of the fracturing process modelling in the frame of the SELFRAC project. The coupled strain localisation analysis as well as the coupled local second gradient model for the post-bifurcation analysis are of great interest for the future to predict the evolution of the EDZ.

A preliminary sealing/healing constitutive model has been proposed. Even though, at this stage, the application of this model still requires further efforts, especially in terms of parameter calibrations, it is very promising for the future to take into account both fracturing process and sealing/healing process.

The developed constitutive and numerical tools have been applied to model the in situ test. In practice, efforts were concentrated on the connecting gallery excavation at Mol site with special attention on the strain localisation and parametric sensibility analysis on the hydraulic disturbance zone. The modelling of the excavation process and the strain localisation prediction along the process have been performed in L3S using the finite strain Finite Element Code Lagamine, with the constitutive law CLoE. A special modelling procedure was designed, taking into account the progressive excavation as a process in time, the placement of a lining behind the excavation front, the existence of an over-excavation with respect to the lining dimensions, and the hydromechanical coupling. The results are consistent with the site observations, especially the localization predicted. The extent of the predicted localisation zone is however smaller than observed during the excavation of the connecting gallery. Further refinements would be necessary to reduce some discrepancies observed with respect to site data. The sensitivity analysis using poro-viscoplasticity analysis has put in evidence some important influencing factors with respect to the hydromechanical responses of Boom Clay to the excavation.

3.6 Conclusions on the long-term performance of a nuclear waste repository in respect of the EDZ around galleries and shafts

Performance Assessment is concerned with the potential radiological consequences of the disposal system. Hence, the analysis of the evolution of the repository system, of which the EDZ is a part, is an essential part of PA. The EDZ is initiated during the repository construction. Its behaviour is a dynamic problem, dependent on changing conditions that vary from open-drift period, to initial closure period, to the entire heating-cooling cycle of the decaying waste. Other factors concern the even longer-term issues of chemical reactions and biological activities.

The EDZ originates in the redistribution of stress which results from excavation and/or excavation method. Fracturing of the rock cannot be avoided in practice. However it is possible to limit the extents of the fractures by controlling the convergence. During the consolidation phase sealing occurs; the open fractures close progressively. During the exploitation phase, sealing can be slow or even inhibited due to the ventilation of the galleries which may prevent a full saturation of the host-rock. Open fractures favour the oxidation of the host rock. However, this oxidation phenomenon has only been observed along fracture planes. Within the clay matrix, no evidences of oxidation were found. After the closure of the disposal galleries, the system will progressively resaturate and suction will disappear. The backfilled galleries will be saturated with water. Favourable conditions to sealing will be recovered. In case of indurated clay host rock bentonite backfill material generating swelling pressure are considered to accelerate the sealing process. Consequently, it can be assumed that the remaining open fractures will seal and reducing chemical conditions will progressively be restored within the EDZ. Once saturation is attained, the thermal load may cause an increase of stresses because of thermal expansion of solids and pore water and heating-related reduction of strength. This process could further increase the size of the EDZ but in turn, heating may also increase creep rates and thus accelerate the closure of open fractures and reduce the influence of EDZ. Peak temperatures within the EDZ should normally be reached after a few tens of years. A subsequent slow cooling phase will follow. Finally, EDZ and engineered barriers will chemically interact.

Safety assessment calculations for Opalinus Clay and Boom Clay showed little impact of the EDZ on radionuclide release even for very conservative assumptions. The different experiments realised in the frame of SELFRAC show that the effective hydraulic conductivity of the EDZ in the considered rocks will be lower than 10^{-10} m/s within several years as soon as the bentonite backfill of the emplacements drifts become fully saturated and the expected swelling pressure will be build up. The hydraulic parameters of natural fractures in Opalinus Clay indicate that the long-term conductivity of the EDZ is even lower. Consequently, the maximum hydraulic conductivity of the EDZ is expected to be approximately one order of magnitude higher than the value of intact rock, which is in the case of Mont Terri in the overall range of 2×10^{-12} to 2×10^{-14} m/s (Heitzmann, 2004). Moreover the assumption that the presence of an EDZ provides a fast path for the escape of solutes from the canister to the geosphere and biosphere is now recognized to be an oversimplification. It may be true that the EDZ is, at least over a period of time, a zone of relatively high permeability, but whether flow can take advantage of it to transport solute to the accessible environment requires an evaluation of the total flow system. Thus, if the high-permeability zone is surrounded by low-permeability regions, or the hydraulic gradient is sufficiently low, there will be an insufficient supply of flowing water in the EDZ to negatively impact the repository performance.

Consequently EDZ should not be considered as a critical issue for the performance assessment of radioactive waste repositories in argillaceous formations. For the safety cases, however, a profound scientific knowledge on the subject is necessary. The results of the SELFRAC project undoubtedly strengthen the sound scientific background. Nevertheless there are some important remaining questions, mostly related to the response of the EDZ to gasses and chemical and thermal changes through time. The understanding of these phenomena is also necessary to support future safety cases.

3.7 Possible remedial actions to the EDZ for different repository stages

A careful site specific repository design may reduce the EDZ extent. Different aspects have to be considered: opening geometry, excavation method, lining stiffness, rapid lining setting, selection of the strongest areas, orientation of the gallery with respect to bedding planes, possible discontinuities, in-situ stresses, and shaft location.

The EDZ may be limited during the shaft and gallery excavation. This is especially important for Boom Clay. Damage intensity and the extent of the EDZ can be controlled and minimised by setting a support immediately after the excavation. The support provides a back pressure, or confinement pressure, which significantly changes the stress distribution in the rock mass (when compared to the no-support case). When a support is set at the gallery wall, radial compressive stresses are larger, and deviatoric stresses are smaller — to some extent, preventing the onset of damage. The support design is based, on one hand, on geomechanical considerations and, on the other hand, on performance assessment requirements (e.g. minimise the amount of steel, cement etc. left in a repository during the post-closure phase) The lining (or rock bolting) must be set at an early stage, immediately behind the gallery front, before the strains generated by the excavation can develop fully. For this reason, both the lining stiffness (i.e. steel or concrete stiffness and lining thickness) and the lining setting time (or bolting scheme) must be selected carefully.

Gallery shape and size are also important. A circular shape is considered to be the optimal shape for a gallery in an isotropic medium, but the anisotropy of rock properties and/or of in situ stresses may lead to a different conclusion. When the two main horizontal stresses are significantly different (a situation that exists in the Mont Terri URL), it is better to orient the gallery axis parallel to the axis of the maximum compressive principal stress. The so-called “size effect” may also be of importance: rock sample strength and rock mass strength often are significantly different, and EDZ extension may be greater than anticipated. In addition, the extent of the EDZ can be significantly larger when larger openings are considered.

In plastic clays, the convergence can be limited during the excavation thanks to a continuous excavation procedure and to the use of a well designed shield. The use of a full face tunnelling boring machine could perhaps reduce the convergence further. Also the liner could be installed within the shield, thus completely avoiding a short unsupported tunnel section behind the shield. However a minimal over excavation remains necessary. A sufficient clearance between the shield of the boring machine and the rock wall is indeed needed to allow a well-controlled and smooth excavation process.

As it is crucial to ensure the excavation controllability, technological constraints (e.g. strength of the support) require at least a minimal convergence during excavation.

The repository design and operation must ensure that emplacement cells (boreholes, small diameter tunnels) remain open for a short period of time and are adequately ventilated. As all processes are highly coupled, the operational conditions, including lining or support, monitoring, air humidity control, should be treated as a whole.

Post excavation remediation may be used. Assuming that the swelling pressure can be controlled, a swelling backfill may contribute to fracture closing within the EDZ. Fluid flow through the EDZ is considered by PA as unlikely, as sealing process is expected to be effective. Even if the EDZ would not seal, the flow along the EDZ would be limited by the availability of water from the intact rock. Nevertheless, optional plugs (EDZ cut-off) within disposal galleries are considered, at least as a conservative measure, in the overall repository design in order to e.g. limit interactions between various repository zones, increase the resilience of the repository to intrusion, and avoid gas migration. The objective is to cut off the damaged/fractured zone in selected areas to prevent bypassing of the engineered seals through more permeable zones. It is a difficult task, as additional damage may be created when constructing a cross-cut.

4 Assessment of results and conclusions

The results of SELFRAC have been assessed by a panel of eight international experts. The panel was composed by representatives of universities, research centres and national radioactive waste management agencies: Politecnico di Torino, IT; CERMES, FR; ANDRA, FR; SCK•CEN, BE; ONDRAF/NIRAS, BE; and NAGRA, CH. The conclusions of the panel are reported hereafter (Bernier et al. 2006)

The definition of EDZ and EdZ may be kept as defined in the Cluster EDZ conference proceedings (Davies and Bernier 2005). Damage in PA is different from damage in geomechanics. The definitions try to establish a bridge between both and form therefore a general consensus on the significance of EDZ and EdZ in PA. However, the definitions should be deepened for each type of rock and each site. In particular, the terms “significant” and “major” should be quantified and the time effect should be discussed for each particular site.

The proposed “sealing” and “healing” definitions are directly related to discontinuities and to the EDZ. It is proposed to enlarge the definition to the reduction of the permeability in the rock matrix and to the thermal sealing processes. The consideration that complete “healing” means a total recovery of the initial state has been accepted.

Although significant progress is made to describe and understand most of the observed processes, clear and comprehensive answers on the origin and development of the fracturing are still missing. However, from a PA point of view, understanding the development of fracturing is not of a critical importance. It is the state of the fractures and their evolution with time that is important to assess the impact on the transport properties. Important parameters for PA are therefore the porosity and the overall effective permeability of the intact and damaged rock.

In both clays sealing processes are clearly observed. In some cases indication for partial healing is provided for plastic clays. For Boom Clay the sealing process is very quick, while the process is significantly slower for Opalinus Clay, but still fast enough from a PA point of view since very long periods of time are considered and complete isolation of the waste is assured by the canisters for 1 000 to 10 000 years. The main sealing processes identified so far are swelling, disintegration, creep and consolidation.

Microfocus X-ray computer tomography reveals to be a very valuable technique to visualise the sealing process and its evolution with time. The acoustic emission technique can be used to investigate damage and damage evolution at micro-scale. However, further research is needed to be able to quantify these processes. It could be interesting to realise the same type of tests considering the influence of temperature.

It can be stated considerately that the tests performed within the SELFRAC project have allowed characterising and quantifying the sealing processes in Boom Clay as well as in Opalinus Clay. The processes of healing, if at all applicable to clay formations, are not yet fully understood.

Although a detailed process understanding on the micro-scale is not critical for PA, a sound scientific understanding is advantageous to adequately support the arguments presented within a safety case.

Although models already explain some of the features that are observed, there is still some effort to make before numerical modelling will be able to model the ongoing processes at the timescale and spatial scale of a repository.

Considering the results of the SELFRAC project:

- EDZ is for NAGRA not a critical issue for the performance assessment at this moment. For the safety cases, however, a profound scientific knowledge on the subject is necessary
- EDZ is also not a critical issue for ONDRAF/NIRAS at this moment. There are, however, some important remaining questions, mostly related to the response of the EDZ to gas production and chemical and thermal changes through time. The understanding of these phenomena should be investigated more deeply in the following years to support the next safety cases,

undoubtedly, the results of SELFRAC strengthen the sound scientific background of the arguments presented within safety cases.

Finally, it is worthwhile to mention that the interest of the study performed in the framework of SELFRAC project is not only of concern to nuclear waste repository, but can also have some very important industrial importance, for instance, in the oil industry (e.g. borehole stability, shaft stability, reservoir exploitation, feasibility assessment of the CO_2 sequestration) as well as in the mining industry or in the engineering industry (tunnelling stability, etc.).

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