

Optimization of concrete lining for the geological storage of radioactive waste in clay formations (en ce compris une introduction à la méthodologie de la recherche)

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Optimization of concrete lining for the geological storage of radioactive waste in clay formations

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Abstract

The long-term geological disposal of radioactive waste requires the construction of deep underground repositories consisting of extensive networks of galleries, each supported by a concrete lining designed to ensure structural integrity over very long timescales. Given the scale of such infrastructure, optimising the lining design and, in particular, its thickness represents a significant challenge: an oversized lining entails unnecessary material costs, while an undersized one may be subjected to pressures exceeding its capacity, compromising the safety of the repository.

This work investigates the mechanical behaviour of concrete linings installed in Boom clay, the host rock studied for the Belgian deep geological repository, using the finite element code *LAGAMINE* developed at the University of Liège. The modelling strategy is deliberately progressive: starting from a simple isotropic, purely mechanical configuration validated against analytical solutions derived from the convergence-confinement method, complexity is gradually introduced through stress anisotropy and hydro-mechanical coupling. The model represents the excavation of a circular gallery at a depth representative of the HADES underground research laboratory (225 m), with a 40 cm thick concrete lining separated from the surrounding clay by an annular gap.

A parametric study is then conducted to assess the influence of the main governing parameters such as the gap size between the lining and the rock, the initial stress state, the coefficient of earth pressure at rest K_0 , and the deconfinement profile on the contact pressure acting on the lining. The results show that larger gap sizes delay the onset of contact and reduce the equilibrium contact pressure, while higher initial stresses and lower values of K_0 increase the anisotropy of the loading around the lining. The introduction of hydro-mechanical coupling reveals a significant time-dependent component to the lining loads, driven by the progressive dissipation of excess pore pressures in the Boom clay over timescales of decades. These findings provide quantitative relationships between key design and geological parameters and the resulting lining loads, which can serve as practical guidelines for the optimisation of gallery linings in deep geological repositories.

Résumé

Le stockage géologique à long terme des déchets radioactifs nécessite la construction de centres de stockage souterrains profonds, composés de vastes réseaux de galeries, chacune soutenue par un revêtement en béton conçu pour garantir l'intégrité structurelle sur des échelles de temps très longues. Étant donné l'ampleur de ces infrastructures, l'optimisation de la conception du revêtement, et en particulier de son épaisseur, représente un défi majeur : un revêtement surdimensionné entraîne des coûts matériels inutiles, tandis qu'un revêtement sous-dimensionné pourrait être soumis à des pressions dépassant sa capacité, compromettant ainsi la sécurité du centre de stockage.

Ce travail étudie le comportement mécanique des revêtements en béton installés dans l'argile de Boom, la roche hôte étudiée pour le stockage géologique profond belge, à l'aide du code aux éléments finis *LAGAMINE*, développé à l'Université de Liège. La stratégie de modélisation est délibérément progressive : partant d'une configuration mécanique purement isotrope et validée par rapport à des solutions analytiques dérivées de la méthode convergence-confinement, la complexité est progressivement introduite par le biais de l'anisotropie des contraintes et du couplage hydro-mécanique. Le modèle représente l'excavation d'une galerie circulaire à une profondeur représentative du laboratoire souterrain de recherche HADES (225 m), avec un revêtement en béton de 40 cm d'épaisseur, séparé de l'argile environnante par un espace annulaire.

Une étude paramétrique est ensuite menée pour évaluer l'influence des principaux paramètres gouvernants, tels que la taille de l'espace entre le revêtement et la roche, l'état de contrainte initial, le coefficient de pression des terres au repos K_0 et le profil de déconfinement sur la pression de contact agissant sur le revêtement. Les résultats montrent que des espaces plus grands retardent le début du contact et réduisent la pression de contact à l'équilibre, tandis que des contraintes initiales plus élevées et des valeurs plus faibles de K_0 augmentent l'anisotropie de la charge autour du revêtement. L'introduction du couplage hydro-mécanique révèle une composante temporelle significative des charges sur le revêtement, pilotée par la dissipation progressive des surpressions dans l'argile de Boom sur des échelles de temps de l'ordre de plusieurs décennies. Ces résultats établissent des relations quantitatives entre les paramètres clés de conception et géologiques et les charges résultantes sur le revêtement, pouvant servir de lignes directrices pratiques pour l'optimisation des revêtements de galeries dans les centres de stockage géologiques profonds.

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Chapter 1

Introduction

This Chapter introduces the global context surrounding this work. In particular, it details the role of nuclear energy in the world and in Belgium.

1.1 Global context

In 2025, global electricity production reached a record-high of approximately 32 000 TWh. According to International Energy Agency (2026), this growth is mainly driven by emerging countries, with China making up most of the new demand in the past ten years. At the same time, as shown in Figure 1.1, developed economies such as the US or Europe experienced a stagnation of their demand. However, forecasts predict that their demand will soon start to rise, led by the growing share of electric vehicles, data centres and many others.

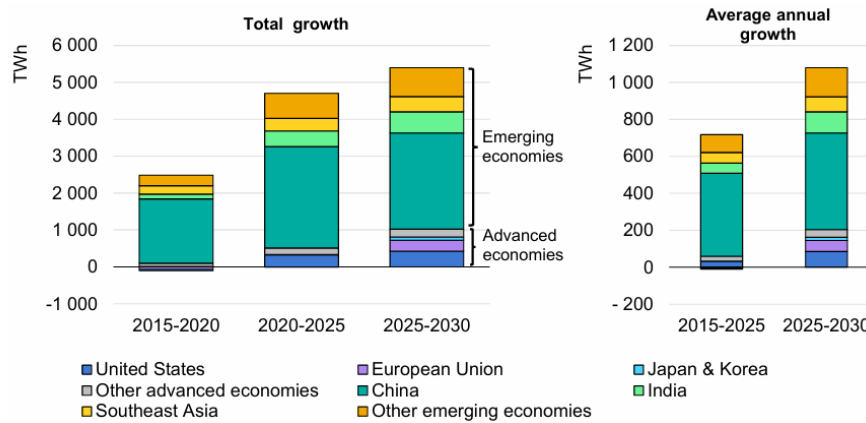


Figure 1.1: Global electricity demand growth by region, 2015-2030 (International Energy Agency (2026)).

From the industrial revolution to the present days, there has been a strong coupling between energy production and CO_2 emissions (Figure 1.2), partly due to historically carbon-intensive ways of producing energy (coal, fossil fuels,...).

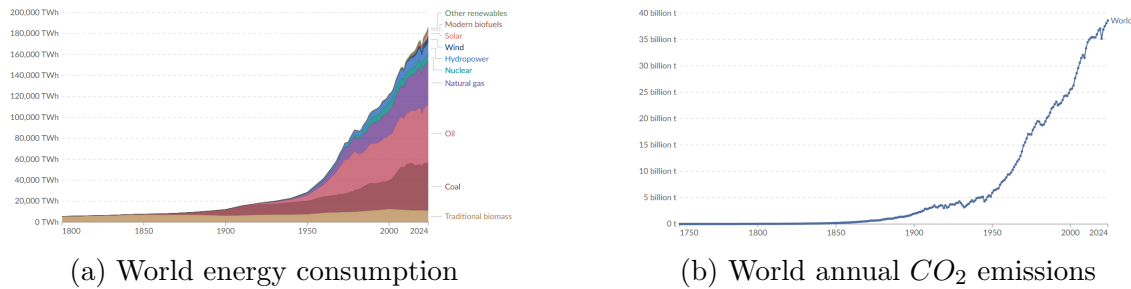


Figure 1.2: Relation between energy consumption and CO_2 emissions (Ritchie, Rosado, and Roser (2020)).

In recent years, most developed economies have begun a transition from CO_2 emitting energy sources to low-carbon sources such as nuclear energy, photovoltaic energy or hydro-energy. Figure 1.3 illustrates three different realities:

- In developed economies (US, Europe,...), despite a stagnating energy consumption, CO_2 emissions are decreasing, meaning that a shift is being operated in energy sources towards cleaner technologies, in addition to deindustrialization.
- In China, despite a fast growth of energy consumption, CO_2 emissions begin to stabilize, partly due to the rapid increase in low-carbon and renewables sources (nuclear, wind, solar).
- Other emerging countries (India, Southeast Asia) are increasing both their energy consumption and their CO_2 emissions, still relying on fossil fuels.

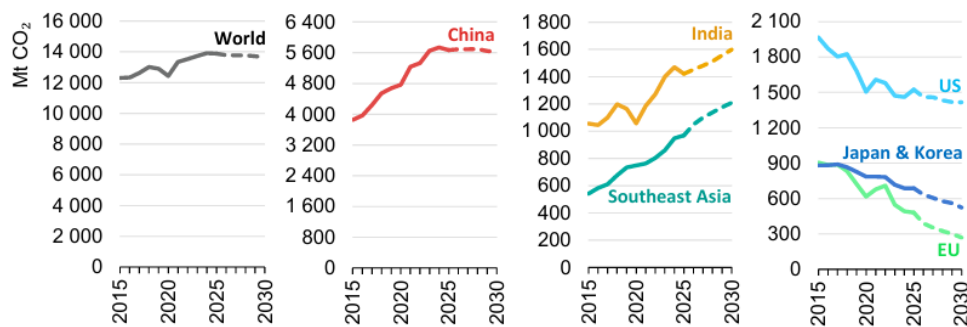


Figure 1.3: CO_2 emissions related to electricity production in various parts of the world (International Energy Agency (2026)).

1.2 Nuclear energy

In 2025, nuclear energy accounted for approximately 10% of the world's electricity (Figure 1.4). In 2024, about 440 reactors were operating in the world and more than half of reactors under construction were located in China (World Nuclear Association (2025b)).

Nuclear power exhibits several advantages:

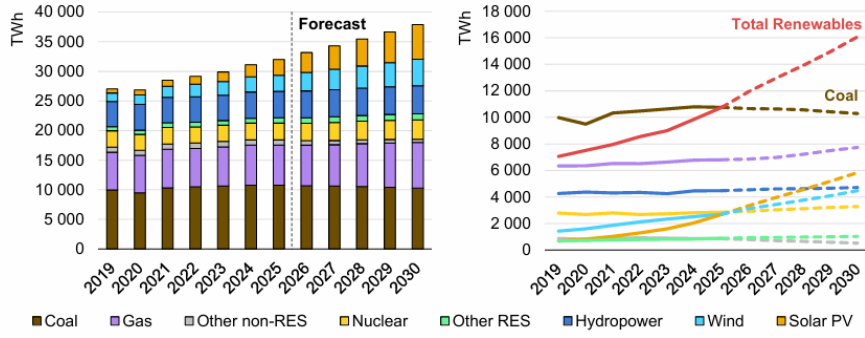


Figure 1.4: World electricity generation (International Energy Agency (2026)).

- It produces very low CO_2 emissions during its operating life compared to other energy sources. Indeed, electricity is generated by the fission of atoms such as uranium or plutonium, leading to a rise in temperature, evaporation of water and the rotation of a turbine, generating electricity. The contrast between France and Germany perfectly illustrates the impact of nuclear power on a national carbon footprint (the electricity productions sources are shown in Figure 1.5). France, which derives about 70% of its electricity from nuclear energy, possesses one of the most decarbonized electricity mixes in Europe, with emissions per kilowatt-hour produced among the lowest of industrialized countries at around $28g(CO_2)_{eq}/kWh$ (Electricity maps (2026)). Conversely, Germany finalized its nuclear exit in 2023. Even with massive investment in wind and solar, it remains heavily dependent on gas and coal to compensate for the intermittency of renewables and the absence of a nuclear baseline, keeping its carbon intensity at a level significantly higher than that of its French neighbour with approximately $388g(CO_2)_{eq}/kWh$ (Electricity maps (2026)) despite having a renewable energy share about 20 percentage points higher than France.

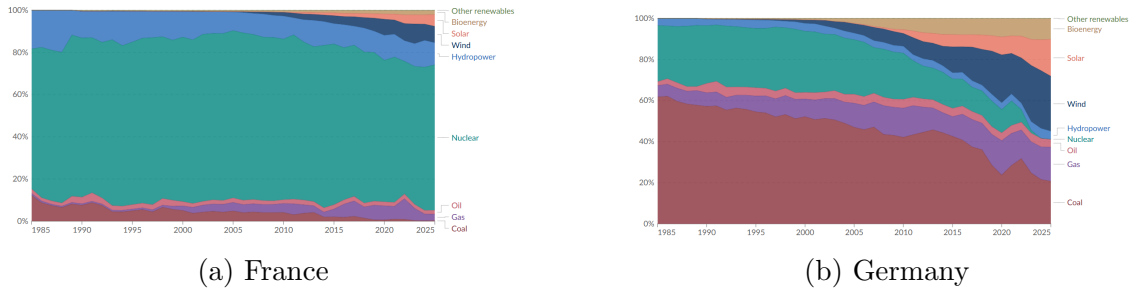


Figure 1.5: Relative share of each electricity production source in the country, France and Germany, Ritchie and Rosado (2020).

- In contrast to wind turbines or photovoltaic panels, it is not an intermittent source. The production can be controlled and does not experience any variation depending on the temperature or other meteorological conditions.
- It does not occupy much surface compared to renewable sources such as photovoltaic panel or wind turbines.
- It produces very low quantities of waste relative to the energy produced.

A downside of nuclear power is that it produces radioactive waste. Radioactive waste encompasses all radioactive materials that have no more use. After a period of time characterized by the half-life of the material, the waste loses its radioactivity and decays into non-radioactive waste. However, most radioactive elements have a very long half-life, reaching tens of thousands of years (World Nuclear Association (2025a)).

The more radioactive an isotope is, the shorter its half-life. Depending on factors such as their level of radioactivity, it is possible to classify waste in three levels (World Nuclear Association (2025a)):

- Low level activity (LLW). It generally consists of clothes or tools that contain very low amount of radioactivity measured in billions of Becquerel per ton [GBq/t] (4 GBq/t of α or $< 12 \text{ GBq/t}$ of β - γ activity).
- Intermediate level waste (ILW). This level of radioactivity is not sufficiently high to be stored in disposal facilities and only requires shielding. This category regroups waste such as metal fuel cladding, contaminated material from reactors or resins.
- High level waste (HLW). It is the highest level of radioactive waste and is capable of generating important heat, requiring cooling and shielding in adequate facilities. HLW is the product of the fission taking place in the reactor.

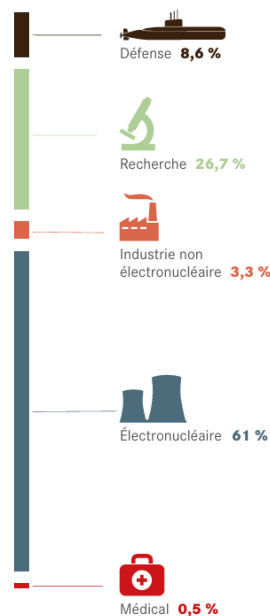


Figure 1.6: Distribution of the origin of existing waste (ANDRA (2026)).

According to ANDRA (2026), the main source of radioactive waste in France is the production of electricity through nuclear energy, representing 61% of the waste being produced (Figure 1.6). Figure 1.7 shows that the vast majority of the waste (over 90%) is either of very low activity or short-lived and is consequently responsible for far less than 1% of radioactivity. Most radioactivity comes from high level waste

which makes up for over 97% despite representing only 0.2% of all produced waste in France. In Belgium, ONDRAF (2024) numbers are summarized in Table 1.1.

	LLW	ILW	HLW
Volume [m^3]	158 156	9 825	2 979
Nuclear power	61.5%	17%	99.9%
Passive	24%	70%	0%
Research and medical sector	11.5%	6%	0%
Other	3%	7%	0.1%

Table 1.1: Origin of nuclear waste in Belgium.

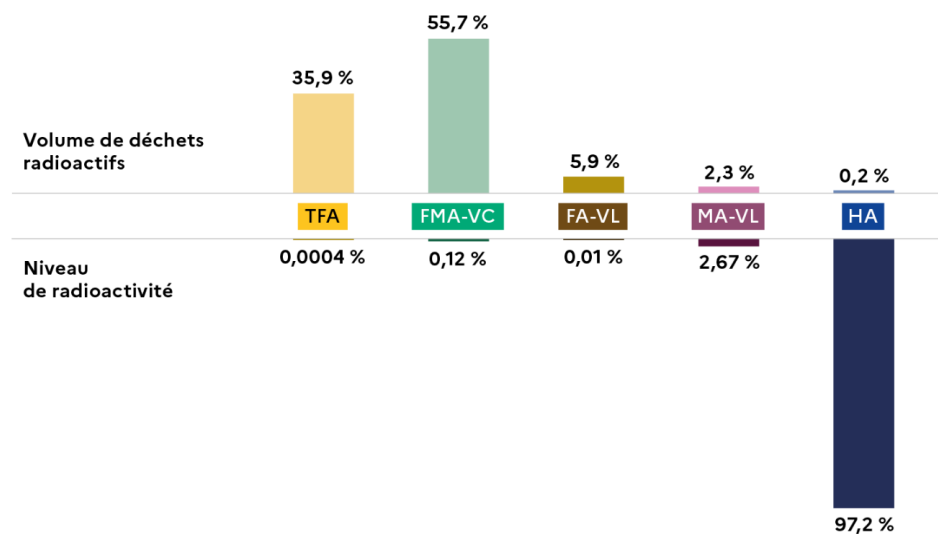


Figure 1.7: 2024 annual report on radioactive waste in France. TFA: very low activity, FMA-VC: low/intermediate activity and short-lived, FA-VL: low activity and long-lived, MA-VL: intermediate level and long-lived, HA: high activity. (ANDRA (2026)).

In Belgium, the first nuclear reactor began operating in 1974. At its peak, the country operated seven reactors across two sites: four reactors in Doel, Flanders and three in Tihange, Wallonia. In 2003, Belgian government voted a nuclear phase-out set to take place between 2015 and 2025 when each reactor would reach a lifetime of 40 years. This has been a subject of controversy and debate across the years and, as of today, five of the seven reactors have been shut down (Doel 3 in 2022, Tihange 2 in 2023, Doel 1, Doel 2 and Tihange 1 in 2025) while the life of operation of the last two reactors has been extended until 2035.

Figure 1.8 shows that, from the 80's, nuclear power has represented between 10 and 20% of consumed energy in Belgium. When looking only at electricity production, in 2024, nuclear power generated 40% of electricity, ahead of renewables sources by approximately 6% (SPF Economie and Waterbley (2025)). Following the 2024 elections, Belgium's new coalition government announced plans to extend the operational life of existing reactors until 2045 and expressed interest in expanding nuclear capacity through the construction of new small modular reactors, signalling a significant shift in the country's energy policy.

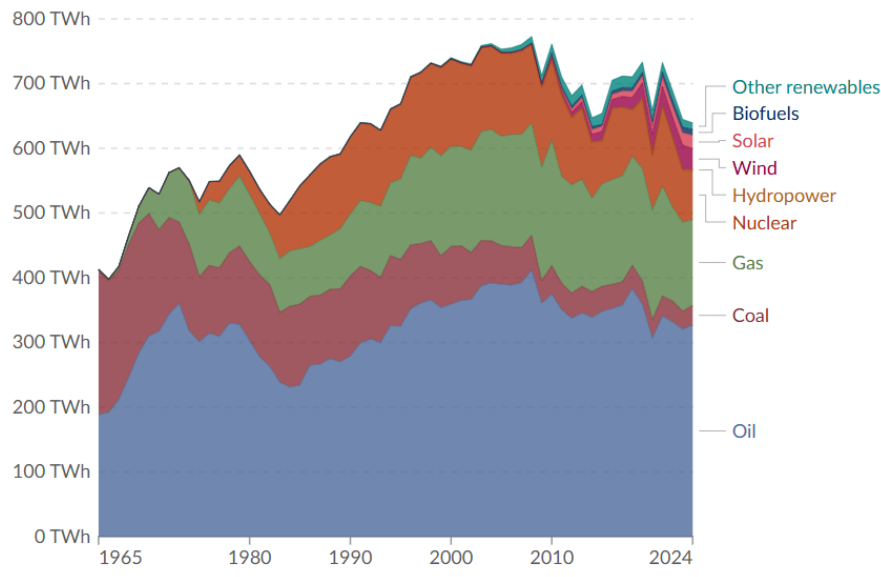


Figure 1.8: Energy consumption by source, Belgium (Ritchie and Rosado (2025)).

1.3 Nuclear waste storage

Because of the dangers mentioned previously, waste must be stored safely. Most short-lived and LLW can be sent to surface-level facilities, as explained previously, this category alone represents more than 90% of the waste volume. The effects of radioactivity can be quantified with three different units:

- The Becquerel (Bq), equal to the number of disintegration per second. It measures the activity.
- The Gray (Gy), equal to the level of energy (in J) absorbed by kg of matter.
- The Sievert (Sv), which quantifies the effect of radioactivity on a biological organism.

The most critical storage concerns HLW, which accounts for a planned volume of $2979 m^3$ (a little bit more than an Olympic swimming pool, ONDRAF (2025)). Because this kind of waste generally exhibits high levels of heat and radioactivity, the first step is to allow them to cool and decay, usually in water or in dry casks, sometimes even on-site of the reactor (World Nuclear Association (2025b)). Worldwide, the preferred option for the long term handling of HLW is deep geological facilities.

Because the half-life of radioactive elements can last several hundred thousand years, there is a need for a passive solution that ensures that the burden of storage is not passed to the next generations. This is why deep geological storage seems the most adapted, as waste is located several hundred meters deep in stable geological formations.

The waste is protected by several layers, with an example illustrated in Figure 1.9. These include both natural and engineered barriers, with the goal that no

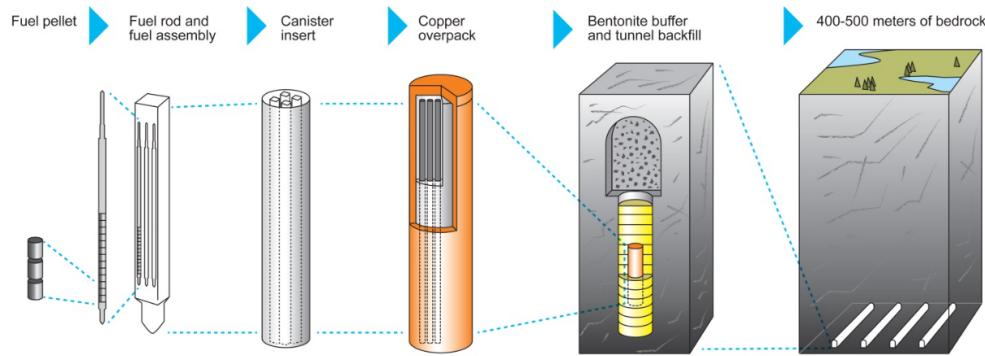


Figure 1.9: The multiple layers isolating waste from the biosphere (World Nuclear Association (2025b)).

maintenance from next generations is needed (World Nuclear Association (2025b)). In Belgium, Levasseur, Van Geet, and Sillen (2018) describes a design of the Belgian concept made of different layers (Figure 1.11):

- The overpack, prevents the contact with water
- The buffer, in concrete to assure a high-pH environment and prevent overpack degradation
- The envelope, prevents the contact with elements from the rock (chlorides, thiosulphates, sulphides)
- The surrounding rock, which can be, for example, a clay (as Boom clay in Belgium) with very low permeability and other characteristics slowing down the migration of the radionuclides in case of leak. This layer is classified as a natural barrier, while others are classified as engineered barriers.

According to Collin (2025), argillaceous rocks exhibit the following advantages and disadvantages:

- | | |
|---|--|
| + Low permeability | - Low strength, leading to the necessity of installing support |
| + No economic value | |
| + Significant radionuclides sorption capacity | - Sensible to chemical changes such as oxidation |
| + Possibility of self-healing properties | |

Once at depth, the storage is subjected to multiple physics processes (mechanical, hydraulic, temperature, chemical). Furthermore, each component has interactions with the others. Temperature changes can affect the properties of fluids (viscosity, density), thus modifying the flow of water or can induce thermal strains, generating more mechanical stresses. Changes in water pressure impact effective stresses (due to Terzaghi principle) and water flow is influenced by temperature (convection).

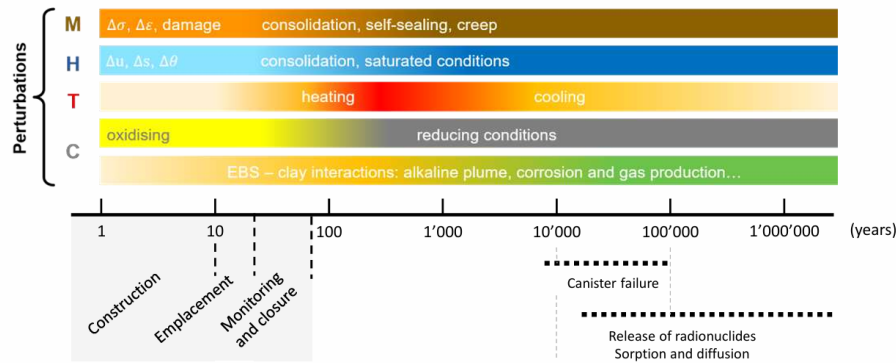


Figure 1.10: Perturbations caused by MHTC processes (Collin (2025)).

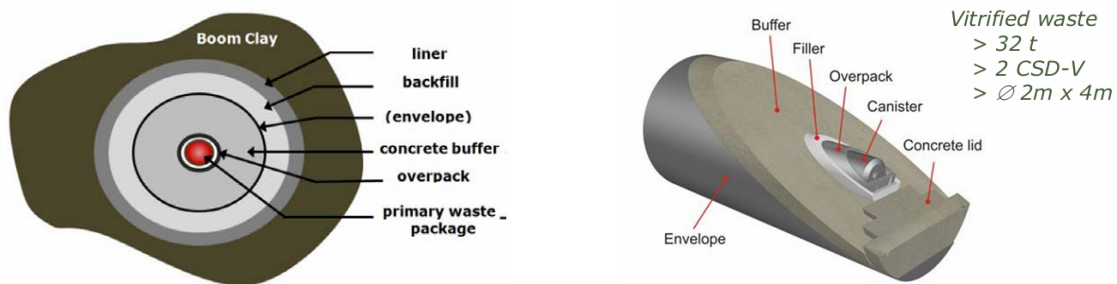


Figure 1.11: Reference design for the Belgian supercontainer (Levasseur, Van Geet, and Sillen 2018).

Changes in the stress state cause deformations and modifications of the porosity which changes the permeability and the storage capacity of the rock (Collin (2025)).

According to ONDRAF (2025), the cost of a deep geological disposal facility in Belgium is estimated at approximately 12 billion euros (based on a single facility, located at a depth of 400 meters in poorly-indurated clay). The feasibility of this project has been estimated in Pirot et al. (2013), with a funding based on agreements between ONDRAF and the producers that have delegated the handling of their waste to ONDRAF. This very high cost highlights the necessity of optimising the construction of all elements involved in the construction of deep geological facilities, especially the gallery in the context of this work. In France, the *CIGEO* project is estimated to lead to the construction of 270 km of galleries at depths of about 500 m for a projected cost of 37 billion euros according to Cigéo (2026).

In Belgium, research regarding the feasibility and safety of geological disposal of radioactive waste is notably performed in a lab, the HADES underground research laboratory (URL).

1.4 Objectives of the work

The construction of a deep geological repository involves the excavation of several kilometres of galleries, each requiring a structural lining to ensure long-term stability. Given the scale of such infrastructure, even a small reduction in lining thickness

translates into substantial cost savings when applied across the entire facility. At the same time, an undersized lining may be subjected to pressures exceeding its design capacity, compromising the safety and integrity of the repository over the very long time scales involved. The central challenge is therefore to understand, with sufficient precision, what pressures the lining will actually experience, and how these depend on the key parameters of both the lining design and the surrounding geological medium.

To address this question, the finite element code *LAGAMINE*, developed at the University of Liège, is used to simulate the excavation of a circular gallery in Boom clay under conditions representative of the HADES underground research laboratory. The modelling approach is deliberately progressive: starting from a simple isotropic and purely mechanical configuration, which is validated against analytical results obtained through the convergence-confinement method, complexity is gradually introduced through stress anisotropy and hydro-mechanical coupling. A parametric study is then conducted, varying the gap between the lining and the rock, the initial stress state, the coefficient of earth pressure at rest, and the deconfinement profile. The goal is to establish quantitative relationships between these parameters and the resulting contact pressure on the lining, which can ultimately serve as practical guidelines for optimizing the design of concrete linings in deep geological repositories.

One of the main goals of this work is to answer the following question: how do the gap size between the concrete lining and the surrounding Boom clay, the initial stress state, and the hydro-mechanical behaviour of the host rock jointly govern the contact pressure acting on the lining, and what quantitative relationships can be derived to guide the optimization of gallery linings in deep geological repositories?

Chapter 2

Literature review

This Chapter presents the main concepts necessary for studying the behaviour of gallery linings in deep geological storage. It covers excavation methods, analytical calculation approaches, and the properties of Boom Clay, the host rock considered in Belgium.

2.1 Tunnel excavation methods

Tunnel sections can be excavated in full or half section. Recent techniques favour the former as they are able to work on the entire section. As highlighted in Collin (2024), Panet and Sulem (2022) or Schoen (2025), the main techniques used in deep tunnel excavation are: drill and blast, roadheader excavation or the use of a tunnel boring machine (TBM).

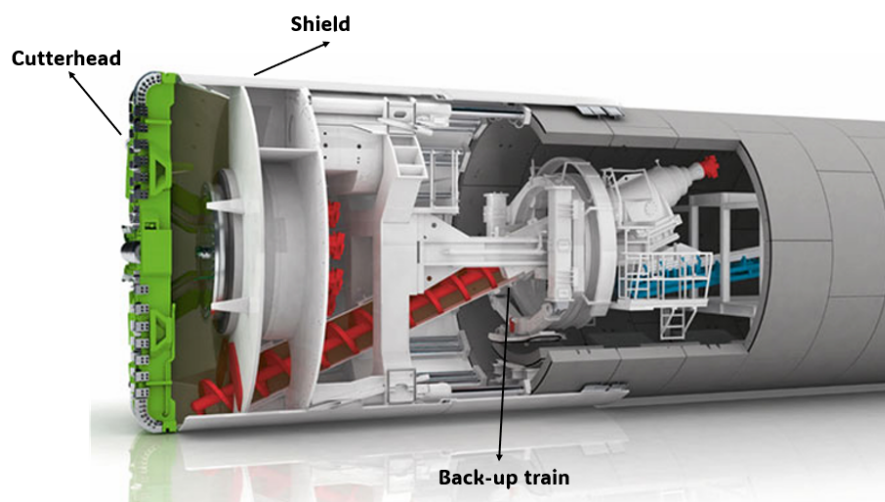


Figure 2.1: Illustration of a TBM (Panet and Sulem (2022)).

Tunnel boring machines are particularly widely used because they allow for both rapid progress and the simultaneous installation of the tunnel lining. TBMs are made of different parts (visible in Figure 2.1):

- The cutterhead which is excavating the ground. The composition of the cutterhead depends on the type of rock that the head encounters. It is possible to use materials such as bentonite if it is necessary to stabilize the tunnel front.
- The shield, protecting the machine itself or the workers.
- The back-up train, with all the elements required to keep the TBM working (removal of the excavated material, installation of the lining,...)

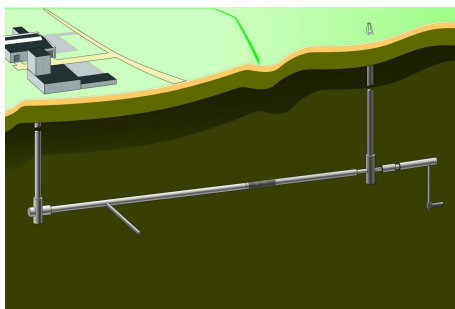
Tunnel digging at high depths can induce the development of an Excavation Damaged Zone (EDZ). In these zones, permeability greatly increases, notably due to the development of cracks (Tsang, Bernier, and Davies (2005)). This is especially problematic in a context of nuclear waste storage, where deep geological facilities are located in formations partly selected for their low permeability.

In Belgium, deep geological facilities are studied in the Hades underground research laboratory (Hades URL).

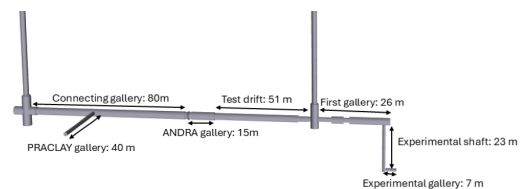
2.1.1 HADES URL

The HADES URL, whose acronym stands for High Activity Disposal Experimental Site, is located 225 metres deep in the Boom Clay formation at the SCK CEN site in Mol, Belgium. It is operated by EURIDICE, a partnership between ONDRAF/NIRAS, the Belgian agency responsible for the safe management of radioactive waste, and SCK CEN, the Belgian Nuclear Research Centre. HADES was the first underground research laboratory specifically designed to study the deep geological disposal of radioactive waste, and it remains a central tool in the Belgian research programme on this topic to this day (HADES lab (2026)).

The knowledge gained at HADES directly informs the design and optimisation of tunnel linings intended for use in deep geological repositories, making it a reference facility of great relevance to this work.



(a) The two shafts.



(b) The galleries.

Figure 2.2: View of the underground research laboratory, HADES lab (2026).

The construction of the laboratory was carried out in multiple steps, using different techniques and linings for all galleries. First, from 1980 to 1982, the soil was frozen down to -25°C (Bastiaens and Bernier (2006)) to ensure that the first shaft

would not close immediately after the excavation, which was performed manually. Then, from 1983 to 1984, the first 26-meter-long gallery was excavated, still manually and in frozen clay, with supports consisting of cast iron rings. In 1984, the experimental shaft and gallery were excavated to investigate creep measurements based on the first gallery, as well a new type of supports (concrete blocks which turned out to be as effective but cheaper). The shaft has since been sealed off. In 1987, the test drift and the ANDRA gallery were excavated, with steel ribs being used as supports for the latter (Hades lab (2026)). In a second, more industrial, phase (Figure 2.2), the laboratory was extended with a second shaft, a connecting gallery and the PRACLAY gallery, which is a prototype of a disposal facility (Industrial expansion of HADES lab in the 90's and '00s (n.d.)).

In the first shaft, the lining is made of concrete poured while Boom clay was frozen in two layers of 40 cm with a PVC-membrane in between them (for an internal diameter of 2.65 m). In the first gallery (internal diameter of 3.5 m) cast iron segments of 20 cm were used. The lining of the experimental shaft and gallery (internal diameter of 2 m) was installed in unfrozen clay and consists of a 30 cm layer of unreinforced concrete blocks. The test drift has been made with different types of supports: rings made of 64 concrete blocks with a thickness of 60 cm in the first part of the gallery and steel sliding ribs in the last 12 meters. Each steel ring includes four profiles connected with clamps, allowing the system to slide if pressure becomes too high in order to reduce the diameter and the pressure (Bastiaens and Bernier (2006)).

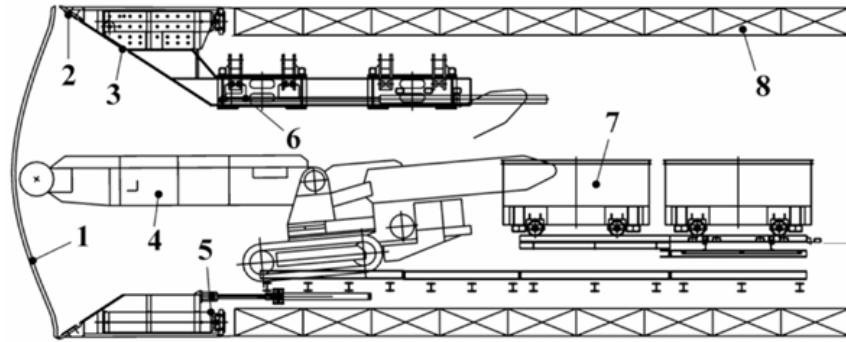


Figure 2.3: "Schematic view of the tunnelling equipment. 1: clay face; 2: cutting head; 3: tunnelling shield; 4: roadheader; 5: hydraulic jack; 6: bird-wing erector; 7: wagon; 8: wedge-block lining." Bastiaens and Bernier (2006).

In the second phase, i.e. the construction of the second shaft, the connecting gallery and the PRACLAY gallery, soil was only frozen in the sand layers on top of Boom clay. The lining of the shaft consists of a primary and a secondary layer. The primary layer in frozen soil is a 20 cm layer of shotcrete. In unfrozen clay, the primary lining is a reinforced concrete foundation. The secondary lining is made of prefabricated concrete rings with an 8 mm thick outer steel casing. All rings are welded to each other. Hot asphalt was poured between the layers to fill the gaps between the primary and secondary layers. In the connecting gallery, the excavation was performed by means of an industrial excavation technique (Figure 2.3) equipped

with a shield and a roadheader. After every meter of excavation, a 1-meter-wide ring was placed. After the passage of the equipment, the final lining consists of a 40 cm thick ring made of 12 segments (Bastiaens and Bernier (2006)).

In the connecting gallery, according to X. Li et al. (2023), the bending moments in the concrete are very low because of the presence of a high radial isotropic stresses. This means that the lining is considered to be working only in compression. Because the keys closing the rings are pushed between the segments, slightly extending them, and that the shield allows the rings to have a radius very close to the excavation radius, it is not necessary to use any kind of grouting to fill the gaps between the lining and the soil. This helps to limit the extent of the damaged zone around the gallery (X. Li et al. (2023)).

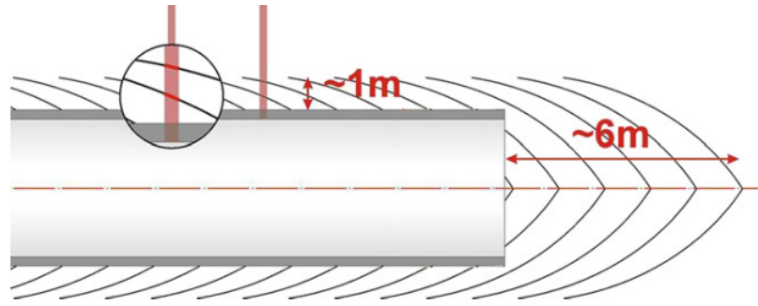


Figure 2.4: Extent of the EDZ around the galleries at HADES URL (Van Marcke and Bastiaens (2010)).

Around the connecting and the Praclay galleries, Van Marcke and Bastiaens (2010) reported the development of an excavation damaged zone visible in Figure 2.4. Fractures appeared in a radius of one meter around the gallery and reached up to six meters up front of the TBM. The configuration of the fractures consists of two planes, with the upper dipping towards the excavation front and the lower dipping towards the back of the excavation.

2.1.2 Lining

Once the excavation is over, the lining can be placed. The closer it is to the tunnel front, the less the ground will converge but the more it will be stressed. The main possibilities are steel supports, concrete supports or the New Austrian Tunnelling Method (NATM).

Steel supports consist of passive bolts (point anchored, friction or fully grouted bolts), active bolts (prestressed bolts), steel ribs or steel lagging sometimes associated with the use of shotcrete to link the different elements (Figure 2.5).

There are several types of concrete supports. Shotcrete (Figure 2.6a) can be projected directly on the walls of the tunnel in association with steel reinforcements. Although less common, concrete cast behind a formwork can be used, either on its own or with a metallic support. Lastly, precast concrete segments (Figure 2.6b) are often used by tunnel boring machines. The TBM places itself the elements as it progresses, allowing the supports to operate very early.

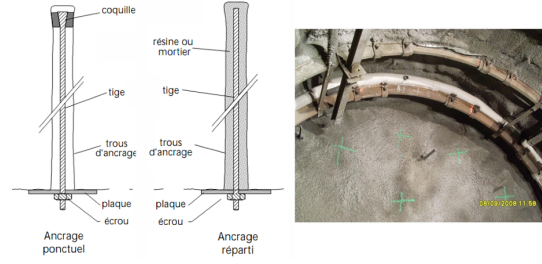


Figure 2.5: Illustration of different steel supports. From left to right: Point anchored bolt, friction bolt and steel ribs, Collin (2024).



(a) Projection of shotcrete.



(b) Concrete precast elements.

Figure 2.6: Concrete lining, Collin (2024).

Another possibility is the use of the NATM (Figure 2.7). By combining shotcrete and radial bolts, the ground is allowed to undergo small displacements, leading to a redistribution of the stresses which participates in the support of the ground. Bolts with glass fibres are also placed at the front of the excavation to stabilize it.

2.2 Analytical computational methods

The principle behind the convergence confinement method (described in Panet and Sulem (2022)) is to replace the three-dimensional problem of a tunnel face advancing through a ground mass with a two-dimensional plane strain equivalent, which is easier to describe and to analyse. This substitution is made possible by the use of a variable called the deconfinement ratio, which captures how the three-dimensional stress redistribution around the tunnel face translates into a progressive relaxation of stresses at the tunnel wall in a cross-sectional analysis. It accommodates various ground behaviours such as elastic or elastoplastic laws. Overall, the convergence confinement approach is a method frequently discussed in articles such as Brown et al. (1983), Corbetta, Bernaud, and Nguyen Minh (1991), Vlachopoulos and Diederichs (2009), Bernaud and Rousset (1996), Carranza-Torres and Fairhurst (2000) or Nguyen and Guo (1996).

This method considers an isotropic and homogeneous rock mass and an isotropic

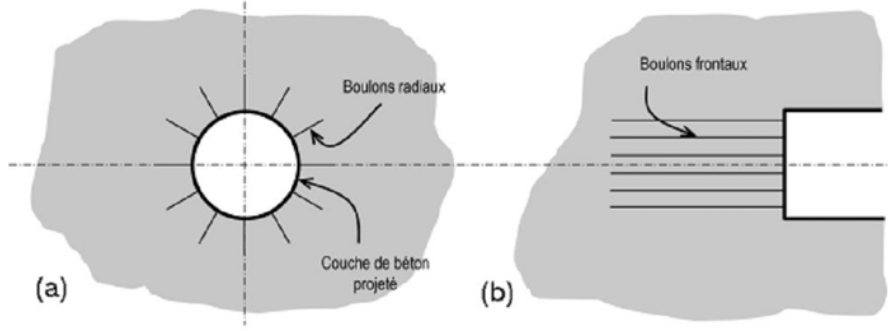


Figure 2.7: New Austrian Tunnelling Method (Collin (2024)).

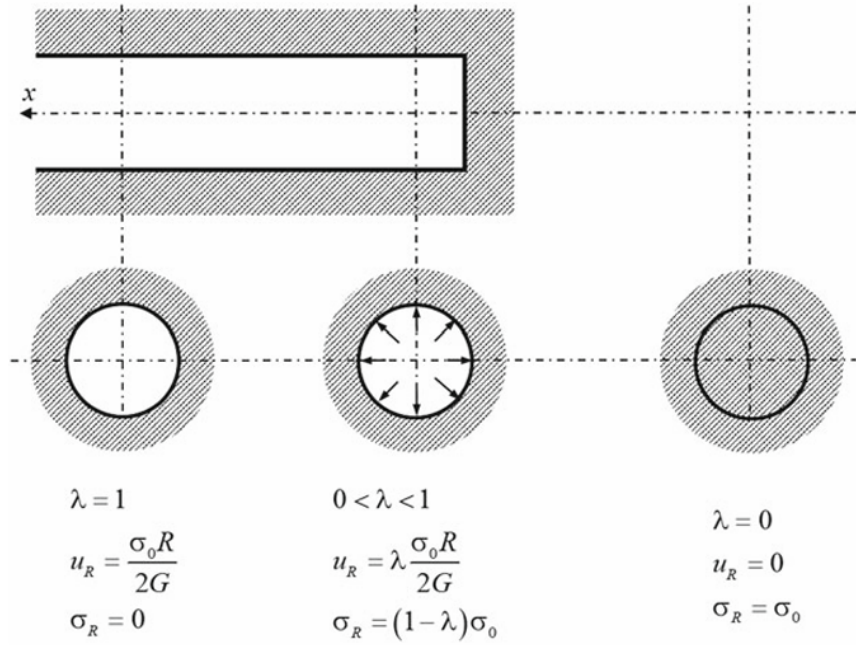


Figure 2.8: Progression of the excavation front according to convergence confinement method (Panet and Sulem (2022)).

initial stress state which constitutes a big limitation. This is why, in this work, it will be used primarily to compare the results obtained for a very simple study case, before moving on to more complete finite element techniques.

2.2.1 Elastic medium

Figure 2.8 illustrates the different steps of the method. In the initial undisturbed state, $\lambda = 0$ and the stress on the tunnel wall is equal to the in-situ stress σ_0 . As the excavation progresses and the face moves away from a given cross-section, the deconfinement ratio increases towards 1, representing the progressive stress release. The stress state applied to the tunnel wall in the plane strain model is thus given by $\sigma = (1 - \lambda)\sigma_0$. Before the excavation, there is no displacement and, as the excavation progresses, displacement increases until an equilibrium is reached between

the support and the rock mass.

For isotropic materials, Hooke's law which expresses the relationship between the stress σ_{ij} and the strain ε_{ij} can be written:

$$\sigma_{ij} = \frac{E}{1+\nu} \left(\varepsilon_{ij} + \frac{\nu}{1-2\nu} \varepsilon_{kk} \delta_{ij} \right) \quad (2.1)$$

$$= 2 G \varepsilon_{ij} + \lambda_L \varepsilon_{kk} \delta_{ij} \quad (2.2)$$

where E is the Young modulus, ν is the Poisson coefficient, $\lambda_L = \frac{E\nu}{(1+\nu)(1-2\nu)}$ is the first Lamé coefficient, $G = \frac{E}{2(1+\nu)}$ is the second Lamé coefficient (also called the shear or Coulomb modulus), ε_{kk} is the trace of the deformation tensor and δ_{ij} is the delta of Kronecker.

The strain can also be expressed as a function of the displacement $u_i(x_j)$:

$$\varepsilon_{ij} = \frac{1}{2} \left(\frac{\partial u_i}{\partial x_j} + \frac{\partial u_j}{\partial x_i} \right). \quad (2.3)$$

By switching to polar coordinates, it appears that Equation 2.3 is limited to two terms:

$$\varepsilon_{rr} = \partial_r u(r) \quad (2.4)$$

$$\varepsilon_{\theta\theta} = \frac{u(r)}{r}. \quad (2.5)$$

As the strains, the stresses are limited to two terms in polar coordinates:

$$\sigma_{rr} = \lambda_L \left(\partial_r u(r) + \frac{u(r)}{r} \right) + 2 G \partial_r u(r) \quad (2.6)$$

$$\sigma_{\theta\theta} = \lambda_L \left(\partial_r u(r) + \frac{u(r)}{r} \right) + 2 G \frac{u(r)}{r}. \quad (2.7)$$

The equilibrium equation then becomes:

$$\frac{1}{r} [\partial_r (r \sigma_{rr}) - \sigma_{\theta\theta}] = 0 \quad (2.8)$$

By using Equations 2.6 and 2.7, Equation 2.8 becomes:

$$(\lambda_L + 2G) r \partial_r \left[\frac{1}{r} \partial_r (r u(r)) \right] = 0 \quad (2.9)$$

Equation 2.9 can be solved with two boundary conditions. When the radial distance r is equal to the radius of excavation of the tunnel R , $\sigma_{rr} = (1 - \lambda) \sigma_0$ and, when infinitely far from the tunnel, $(r \rightarrow \infty) = 0$. The displacement is then given by:

$$u(r) = \lambda \frac{\sigma_0}{2G} \frac{R^2}{r} \quad (2.10)$$

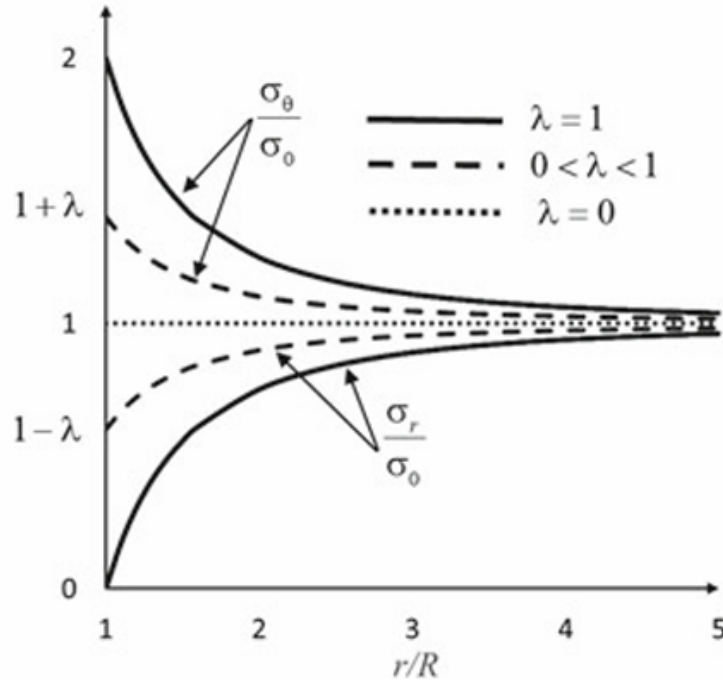


Figure 2.9: Radial and orthoradial stresses versus radial distance (Panet and Sulem (2022)).

Figure 2.10 illustrates that displacement decreases as the distance from the tunnel walls increases. When far enough from the tunnel, the rock remains undisturbed.

Using Equation 2.10 in Equations 2.6 and 2.7 gives:

$$\sigma_{rr} = \left(1 - \lambda \frac{R^2}{r^2}\right) \sigma_0 \quad (2.11)$$

$$\sigma_{\theta\theta} = \left(1 + \lambda \frac{R^2}{r^2}\right) \sigma_0 \quad (2.12)$$

Figure 2.9 is the representation of Equations 2.11 and 2.12 and shows that the stresses converge towards σ_0 with the radial distance. As it was the case with Figure 2.10, it can be observed that initial stress state remains undisturbed when far enough from the tunnel. The deconfinement rate λ also influences the evolution of the stresses: when $\lambda = 0$, the deconfinement has not yet begun and the stress state is still at its initial value, when λ increases, the stress gets further from its initial value.

When a lining is introduced at a distance d from the excavation front, it is possible to use the pressure vessel equation for thin walls under a pressure p_s and for a thickness e to link the pressure p_s acting on the lining (which is coming from

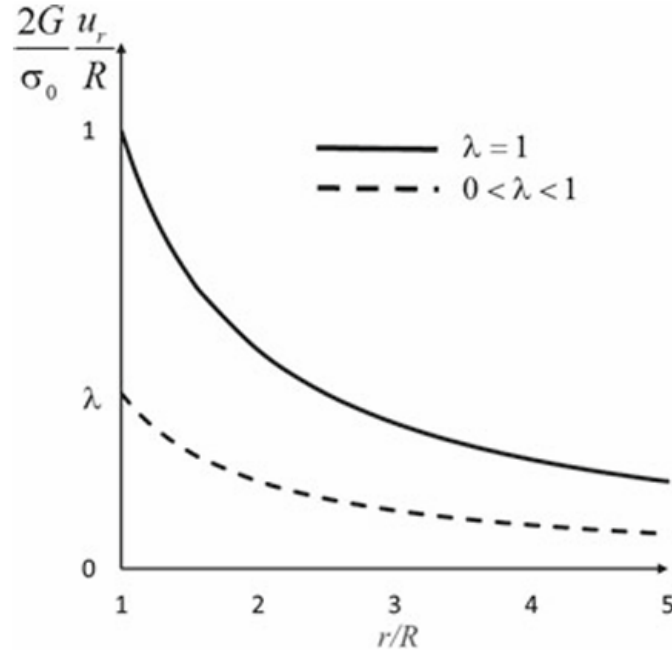


Figure 2.10: Radial displacement versus radial distance (Panet and Sulem (2022)).

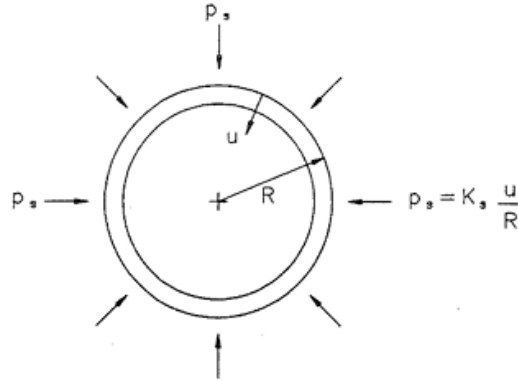


Figure 2.11: Pressure acting on the lining (Collin (2024)).

the rock mass) to the radial displacement of the walls u :

$$\sigma_{\theta\theta} = \frac{p_s R}{e} \quad (2.13)$$

$$\Leftrightarrow p_s = \frac{e}{R} \sigma_{\theta\theta} \quad (2.14)$$

$$\Leftrightarrow p_s = \frac{e}{R} E \varepsilon_{\theta\theta} \quad (2.15)$$

$$\Leftrightarrow p_s = K_s \frac{u(R)}{R} \quad (2.16)$$

where $K_s = \frac{e}{R} E$. Equation 2.16 does not account for the bending rigidity of the lining and is only valid for a radial loading.

Previous equations do not allow to account for the flexural rigidity of the lining or for non-radial loadings. These limitations can be lifted by establishing Equations

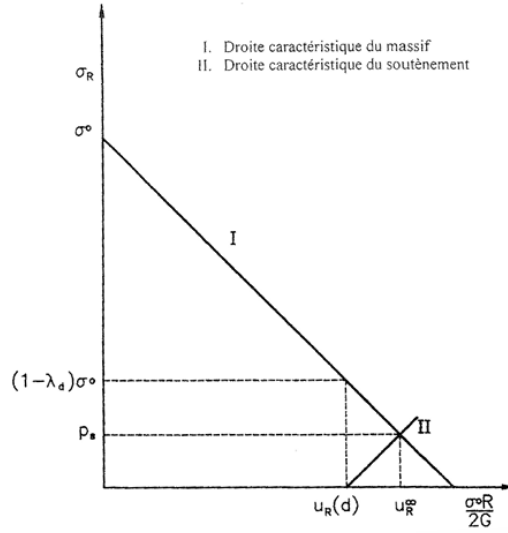
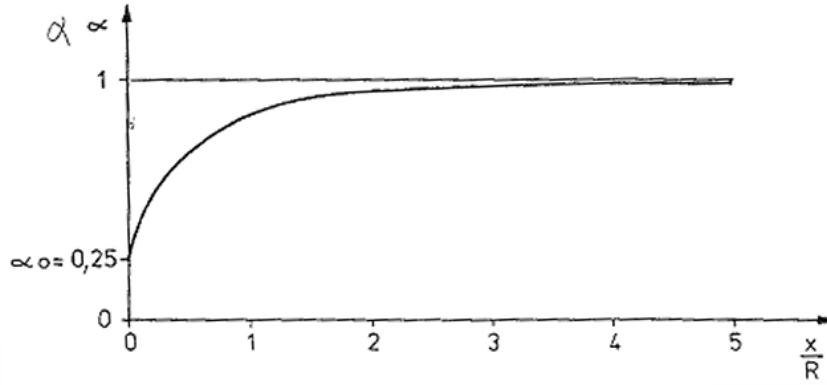


Figure 2.12: Characteristic curves of the rock and the support (Collin (2024)).

Figure 2.13: $\alpha(x)$ with parameters $\alpha_0 = 0.25$ and $m = 0.75$ (Collin (2024)).

2.17 and 2.18.

$$\sigma_R = K_{sn} \left(\frac{u}{R} + \frac{1}{R} \frac{dv}{d\theta} \right) + K_{sf} \left(\frac{1}{R} \frac{d^4 u}{d\theta^4} + \frac{2}{R} \frac{d^2 u}{d\theta^2} + \frac{u}{R} \right) \quad (2.17)$$

$$\tau_{R\theta} = -K_{sn} \left(\frac{1}{R} \frac{du}{d\theta} + \frac{1}{R} \frac{d^2 v}{d\theta^2} \right) \quad (2.18)$$

where σ_R and $\tau_{R\theta}$ are the normal and shear stress on the lining, u and v are the radial and orthoradial displacements, $K_{sn} = \frac{E}{1-\nu^2} \frac{e}{R}$, $K_{sf} = \frac{E}{1-\nu^2} \frac{I}{R^3}$ and $I = \frac{e^3}{12}$.

The equilibrium (visible in Figure 2.12) is found when the rock mass and lining (placed at distance d from the excavation front, when a displacement u_d already happened) curves intersect, i.e. when:

$$\sigma_R - K_{sn} \frac{u(R) - u_d}{R} = \sigma_R + 2G \frac{u(R)}{R} - \sigma_0 \quad (2.19)$$

If the lining is placed at a great distance from the excavation front, the displace-

ment u_d is greater and the intersection happens for lower pressures. The bigger the rigidity of the lining, the higher the pressure at intersection.

All previous developments concerned the transverse plane to the tunnel. However, it is necessary to consider the longitudinal plane in order to calculate the displacement u_d . In sections that have not yet been excavated and are located far from the face ($x < -2R$), the displacement is near zero. In sections that have already been excavated and are located far from the face ($x > 4R$), the displacement stabilizes at $u_{R,\infty} = \frac{\sigma_0 R}{2G}$. For all distances in between ($-2R < x < 4R$), a formula based on numerical finite element simulations has been developed and is shown in Figure 2.13:

$$u_R(x) = \alpha(x) \frac{\sigma_0 R}{2G} = [\alpha_0 + (1 - \alpha_0)a(x)] \frac{\sigma_0 R}{2G} \quad (2.20)$$

where $a(x) = 1 - [\frac{mR}{mR+x}]^2$ with α_0 and m two numerical parameters typically equal to 0.25 and 0.75. The link between the longitudinal and transverse planes is thus ensured by the use of the deconfinement rate λ and the function $\alpha(x)$.

2.2.2 Elasto-plastic medium

If the rock mass is considered to have an elasto-plastic behaviour, plasticity will develop as soon as a specific value λ_e of the deconfinement rate is reached. As illustrated in Figure 2.14, a plastic zone characterized by radius R_p develops around the excavation. The value of the deconfinement rate λ_e at which plasticity appears depends on the plasticity criterion used. For the Tresca criterion described in Section 3.2.1.1:

$$\lambda_e = \frac{c}{\sigma_0} = \frac{1}{N} \quad (2.21)$$

where c is the cohesion of the soil and $N = \frac{2\sigma_0}{\sigma_c}$ with σ_c the uniaxial strength is called the stability number, based on Broms and Bennermark (1967).

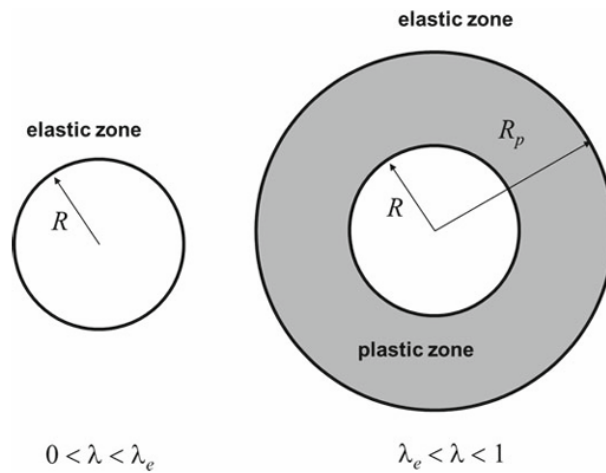


Figure 2.14: Development of plastic zone of radius R_p around the excavation (Panet and Sulem (2022)).

Depending on the value of the stability number, it is possible to distinguish three situations represented in Figure 2.15:

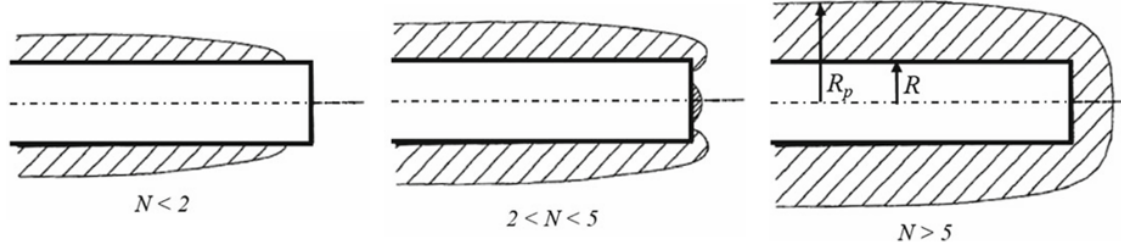


Figure 2.15: Plastic zone around the excavation depending on the stability number (Panet and Sulem (2022)).

- $N < 2$: the plastic zone is only located behind the face of the excavation. The front is stable.
- $2 < N < 5$: parts of the excavation face are in the plastic zone, either due to an excess of radial or orthoradial stresses depending on the zone.
- $N > 5$: the excavation is completely included in the plastic zone. The front is unstable and requires stabilization.

This plastic zone can be called the Excavation Damaged Zone (EDZ) and is linked to the redistribution of the stresses as the deconfinement ratio increases. Around the excavation, radial stresses decrease until becoming zero while the tangential stresses increase, leading to local failure (Blümling et al. (2007)).

The formation of the Excavation Damaged Zone is linked to several parameters: anisotropy of stresses or material, parameters due to the tunnel (the support or, as explained before, the geometry or the deconfinement ratio λ_e) or possible inhomogeneities in the rock (Blümling et al. (2007)).

In the context of radioactive waste storage, the development of the EDZ mainly causes issues due to the change in hydraulic properties. According to Tsang, Bernier, and Davies (2005), irreversible fractures of different scales occur in this zone, increasing properties such as permeability by one order of magnitude or more. That increase in permeability contributes to reducing the efficiency of the natural barrier whose role is to slow down the potential migration of radionuclides.

When the elastic deconfinement rate is reached and $R = R_p$, the formulas for stresses and displacements change for $\lambda > \lambda_e$ and become (by supposing an isochoric behaviour in plasticity and a Tresca criterion):

$$R_p = R e^{\frac{\lambda - \lambda_e}{2\lambda_e}} \quad (2.22)$$

$$u(r) = \lambda_e \frac{\sigma_0}{2G} \frac{R_p^2}{r} \quad (2.23)$$

$$\sigma_r = (1 - \lambda + 2\lambda_e \ln \frac{r}{R}) \sigma_0 \quad (2.24)$$

$$\sigma_\theta = \left(1 - \lambda + 2\lambda_e \left(1 + \ln \frac{r}{R}\right)\right) \sigma_0 \quad (2.25)$$

2.3 Boom clay

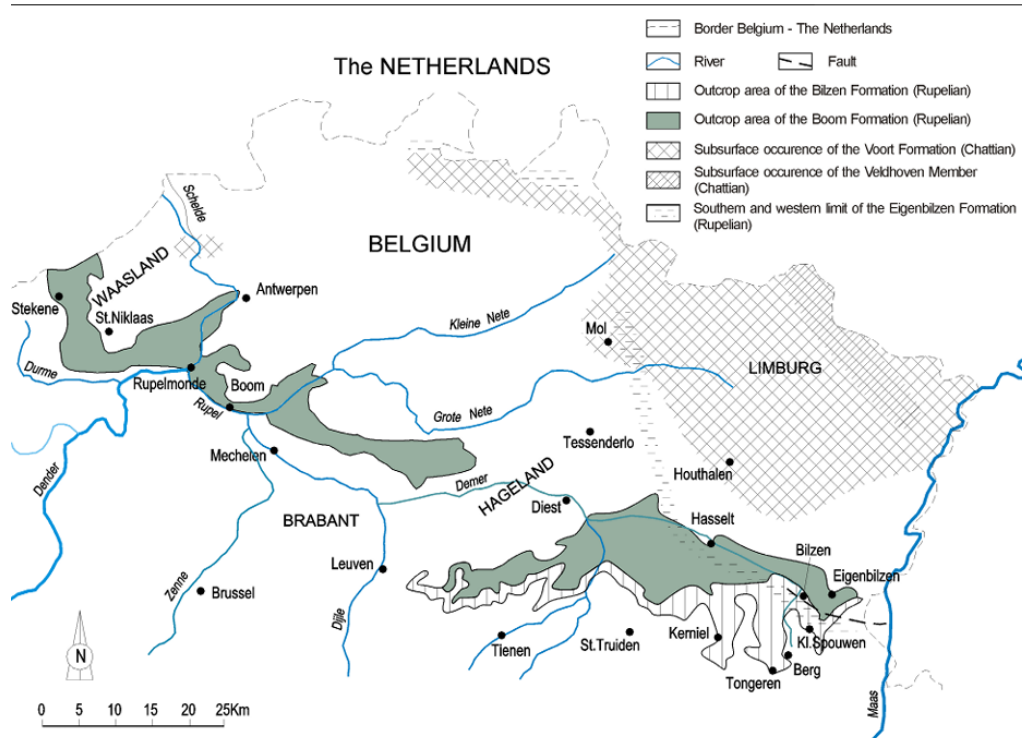


Figure 2.16: Campine area and the presence of Boom clay (Vandenberghe, De Craen, and Wouters (2014)).

Given that the first case study uses the HADES URL in Mol, Belgium (as described earlier), the material surrounding the excavation is Boom clay. Boom clay is a clay with marine origins that was deposited at the bottom of the sea during the Rupelian stage, between about 33.9 and 28.1 million years ago. This period also saw the onset of the Oligocene, when Earth's climate shifted from a mainly greenhouse regime in the Eocene to a cooler, partly glaciated world where ice was growing at the South Pole causing sea-level fluctuation and changes in the volume of water on Earth (Vis and Verweij (2014)). According to Vandenberghe, De Craen, and Wouters (2014), from a tectonic perspective, the Rupelian was marked by important Alpine deformation and the development of a several-hundred-metre-deep foreland basin between the European plate and the growing Alpine chain. The marine transgression that led to the deposition of Boom Clay and other Rupelian clays in relatively deep water in northwestern Europe is therefore attributed primarily to regional subsidence due to Alpine evolution, rather than to a global sea-level rise (because, as described previously, sea-level were rather declining at that time).

Boom clay is therefore located in the Campine region (constituted of northern Belgium and southern Netherlands) where it presents a northeast dip with a height of 200 m for a length of about 40 km, leading to depths of around 200 to 300 m along the border between Belgium and the Netherlands (Figure 2.17). The thickness of the formation varies between 80 and 140 m depending on the exact location. It is also worth noting that Boom clay in the Campine region has never been buried much deeper than it currently is (Vandenberghe, De Craen, and Wouters (2014))

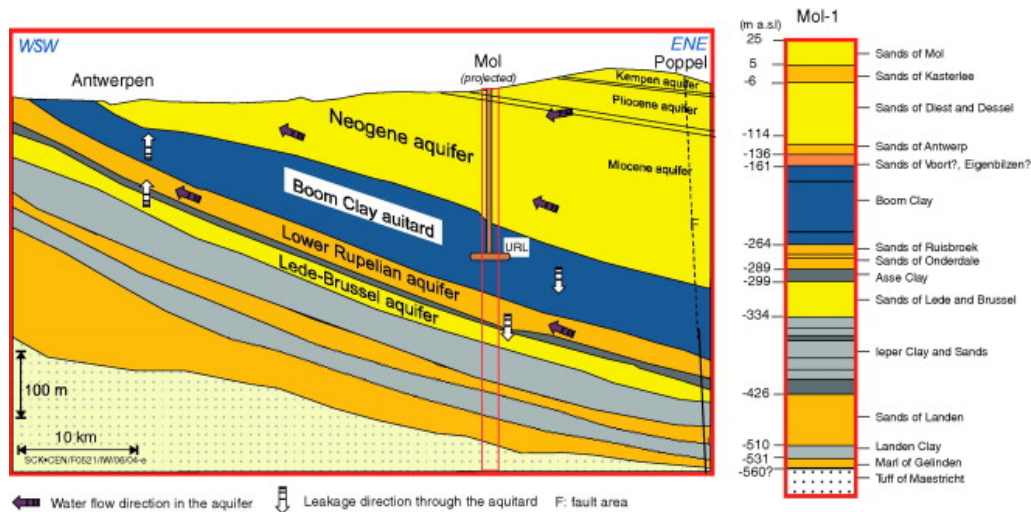


Figure 2.17: View of the dip of the Boom clay layer and hydraulic properties (Wemaere, Marivoet, and Labat (2008)).

and is only slightly overconsolidated, has a high swelling pressure and a very low permeability (Dizier, Guangjing, and Xiang-Ling (2018)). During several millions of years, Boom clay was covered by other sediments such as sands (Dao (2015)).

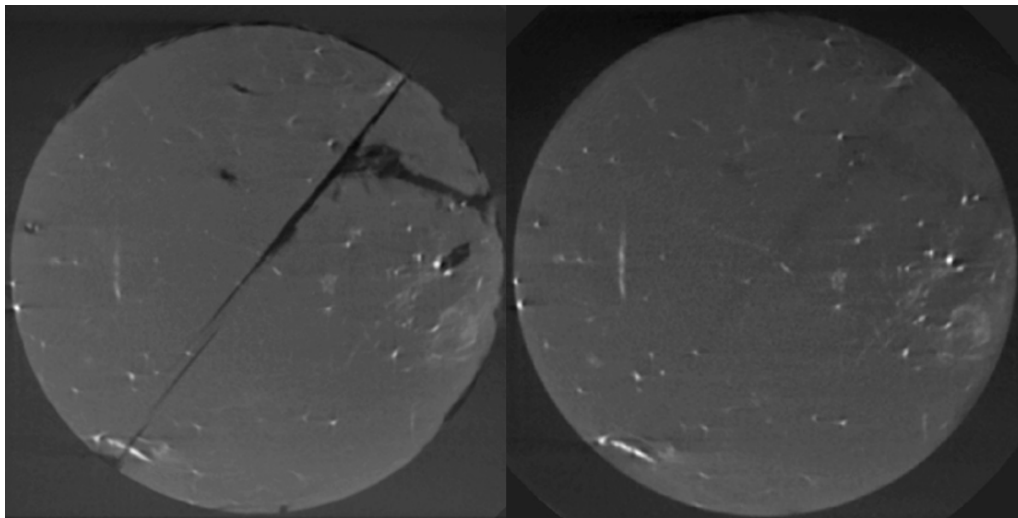


Figure 2.18: Same sample before and after 4.5 hours of hydraulic conductivity measurements, highlighting the sealing capacity of Boom clay, Bastiaens, Bernier, and X. L. Li (2007).

Regarding radioactive waste storage, an interesting property of Boom clay is its self sealing capacity, which is the result of three different phenomena: swelling, consolidation and creep. The sealing capacity of Boom clay is high, healing was also noticed, although lower (Bastiaens, Bernier, and X. L. Li (2007)). Sealing contributes to lowering the permeability, slowing down the potential migration of radioactive waste. Figure 2.18 shows an experiment conducted by Bastiaens, Bernier, and X. L. Li (2007) where the same sample was photographed before and after saturation, highlighting the very fast sealing processes of Boom clay.

The main properties of Boom clay are summarized in Table 2.1, which is based

on different studies.

Parameter	Bésuelle et al. (2014)	Bernier, X. L. Li, and Bastiaens (2007)	Konstadinou et al. (2025)
E [MPa]	150/500	300	—
ν	—	0.125	—
c [kPa]	80	300	—
φ [°]	18	18	—
ψ [°]	4 – 11	0 – 10	—
q_c [MPa]	2.5	—	—
n [%]	36.5	39	—
k [m ²]	3×10^{-19}	—	—
w [%]	20 – 25	—	20 – 40
w_p [%]	26	—	17.8 – 20.1
w_L [%]	60	—	56 – 80
PI [%]	33	—	—

Table 2.1: Boom clay main parameters.

Specifically at the Hades URL, measurements found values for the stress state in the ground: total stresses of $\sigma_{xx} = \sigma_{zz} = 3.825$ MPa and $\sigma_{yy} = 4.5$ MPa with a uniform pore water pressure $p_{w0} = 2.25$ MPa.

The granulometric curve of Boom clay is shown in Figure 2.19. According to Lima (2011), the uniformity coefficient is equal to $C_u = \frac{d_{60}}{d_{10}} \approx 7$ and the curvature coefficient is equal to $C_c = \frac{(d_{30})^2}{d_{60} d_{10}} \approx 1.1$, which means that the soil has a well-spread grain size distribution ($C_u > 3$) and is well graded ($C_u > 3$ and $1 < C_c < 3$).

Francois et al. (2012) derived anisotropic parameters for Boom clay displayed in Table 2.2.

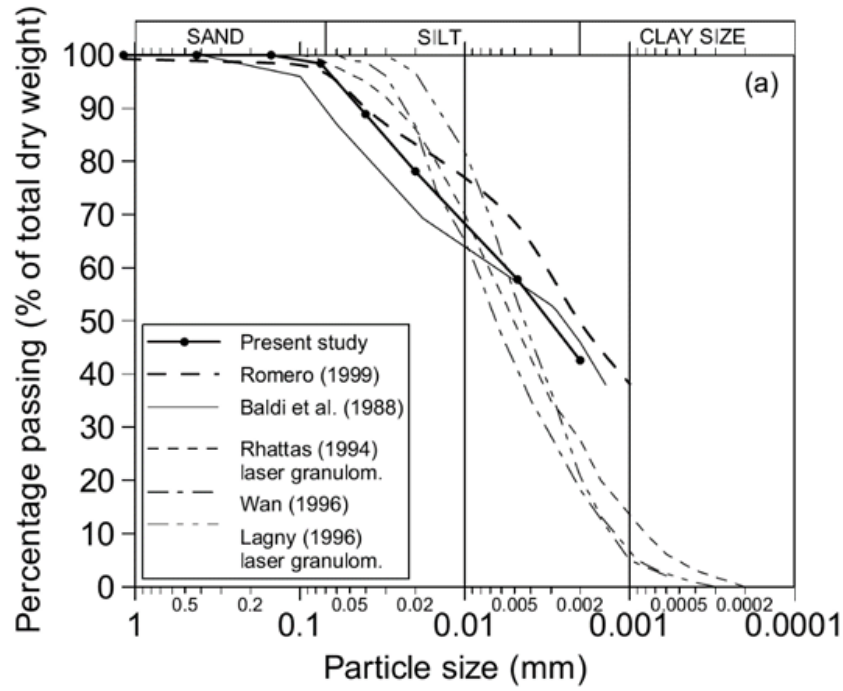


Figure 2.19: Granulometric curve of Boom clay, Lima (2011).

Parameters		
Young elastic modulus (MPa)	$E_{//}$	400
	$E_{\perp}$	200
Poisson ratio (—)	$\nu_{///}$	0.125
	$\nu_{//\perp}$	0.125
Shear modulus (MPa)	$G_{//\perp}$	178
Initial cohesion (kPa)	c_i	255 (0°)
		240 (45°)
		330 (90°)
Final cohesion (kPa)	c_f	85 (0°)
		80 (45°)
		110 (90°)
Softening parameter (—)	B_c	0.01
Initial friction angle (°)	ϕ_i	5
Final friction angle (°)	ϕ_f	18
Hardening parameter (—)	B_p	0.01
Dilatancy angle (°)	ψ	0

Table 2.2: Value of anisotropic parameters for Boom clay with a Drucker-Prager criterion, Francois et al. (2012).

2.4 Conclusion

This literature review highlights the mechanisms governing the interaction between the rock mass and the surface layer. It also provides the essential theoretical and geotechnical framework for the development of the numerical model presented in the following chapters.

Chapter 3

Description of the finite element model

This Chapter describes the finite element model developed using the software *LAGAMINE* to represent the excavation of a gallery in Boom Clay. The equilibrium equations, constitutive laws, and material parameters used are detailed.

3.1 Balance equations

All materials in the model are considered as a porous medium, represented in Figure 3.1. Such medium is made of three phases: solid, liquid and gas.

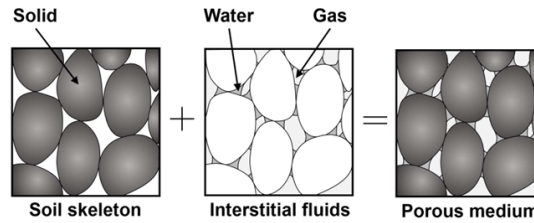


Figure 3.1: Porous medium consisting of three phases (solid, liquid and gas), Bar-nichon (1998).

As described in Corman (2023), the solid phase is the soil skeleton and is often considered to be incompressible, meaning that they do not deform but rather rearrange themselves in the medium. The place not occupied by the solid skeleton is characterized by the porosity of the material, which can be open or closed. These voids can be filled by water or gas, thus affecting the saturation of the porous medium.

The problem, studied through the finite element software *LAGAMINE*, is governed by four balance equations. To close the system, constitutive equations are required to relate the field variables to the material properties of each component (concrete lining, Boom Clay, etc.) as defined in the software.

All equations presented in this section are further explained and described in Corman (2023).

3.1.1 Momentum equation

The momentum equation expresses Newton's second law: the rate of change of momentum equals the sum of all forces acting on the system. From the equation of motion, it is possible to write Equation 3.1 which becomes Equation 3.2 and describes the density of a porous medium considering the contributions of solid, water, and gas phases.

$$\frac{\partial \sigma_{ij}}{\partial x_j} + \rho g_i = 0 \quad (3.1)$$

$$\rho_{\text{mix}} = \rho_s(1 - \phi) + \rho_w S_{rw} \phi + \rho_g(1 - S_{rw})\phi \quad (3.2)$$

where $\phi = \frac{\Omega_w}{\Omega}$ is the porosity, with Ω being the current volume of a given mass of skeleton and Ω_w the corresponding porous volume, ρ_s is the solid grain density, ρ_w is the water density, ρ_g is the gas density, and S_{rw} is the water degree of saturation.

3.1.2 Mass balance equations

Mass balance equations describe that, for any control volume, the accumulation of mass of a given phase equals what flows in minus what flows out, plus any source or sink. Mass is neither created nor destroyed within a phase, unless it transfers to another phase.

The solid mass balance equation can be written as Equation 3.3.

$$\dot{M}_s = \frac{\partial}{\partial t} (\rho_s(1 - \phi)\Omega) = 0 \quad (3.3)$$

where M_s is the solid mass inside the current porous material configuration Ω , ρ_s is the density of solids and ϕ is the porosity.

The liquid mass balance equation can be written as Equation 3.4.

$$\underbrace{\frac{\partial f_{w_i}}{\partial x_i}}_{\text{Liquid water}} + \frac{\partial}{\partial t} (\rho_w \phi S_{rw}) + \underbrace{\frac{\partial f_{v_i}}{\partial x_i}}_{\text{Water vapour}} + \frac{\partial}{\partial t} (\rho_v \phi S_{rg}) - Q_w = 0 \quad (3.4)$$

where f_{w_i} and f_{v_i} are the mass flows of liquid water and water vapour, Q_w is the liquid water and water vapour source/sink term, ρ_w and ρ_v are the densities of liquid water and water vapour and S_{rw} and S_{rg} are the degrees of saturation in water and gas.

Finally, the gas mass balance equation can be written as Equation 3.5.

$$\underbrace{\frac{\partial f_{g_i}}{\partial x_i}}_{\text{Dry gas}} + \frac{\partial}{\partial t} (\rho_g \phi S_{r_g}) + \underbrace{\frac{\partial f_{dg_i}}{\partial x_i}}_{\text{Dissolved gas}} + \frac{\partial}{\partial t} (\rho_{dg} \phi S_{r_w}) - Q_g = 0 \quad (3.5)$$

where f_{g_i} and f_{dg_i} are the mass flows of dry gas and dissolved gas, Q_g is the dry gas and dissolved gas source/sink term, ρ_g and ρ_{dg} are the densities of gas and dissolved gas and S_{r_w} and S_{r_g} are the degrees of saturation in water and gas. In this work, during all simulations, the degree of freedom corresponding to gas will be fixed to atmospheric pressure.

3.2 Constitutive equations

The constitutive equations are used to link the parameters of the balance equations to the variables of the problem which are the displacement u_i , the water pressure p_w and the gas pressure p_g .

3.2.1 Mechanical model

The expression of the effective stress is given by Equation 3.6.

$$\sigma'_{ij} = \sigma_{ij} - b_{ij} (S_{rw} p_w \delta_{ij} + (1 - S_{rw}) p_g \delta_{ij}) \quad (3.6)$$

where σ'_{ij} is the effective stress, σ_{ij} is the total stress, b_{ij} is the Biot coefficient tensor, S_{rw} is the residual water saturation, p_w is the water pressure, p_g is the gas pressure, and δ_{ij} is the Kronecker delta (equal to 1 if $i=j$, 0 otherwise).

3.2.1.1 Plasticity

There are several models that can govern the development of plasticity in the materials. The simplest model one can imagine is certainly a linear elastic behaviour, illustrated in Figure 3.2.

An elastic model is a relationship between stress and strain characterized by the Young modulus E (in MPa), the Poisson coefficient ν and eventually the elastic limit f_y (in MPa). Upon unloading, deformations are completely recovered. The elastic law can be linear, in which case it is a simple line of slope E corresponding to Hooke law $\sigma = E \times \varepsilon$ or it can be non-linear (such as the hyperbolic law used in Duncan and Chang (1970)).

In a plastic model, the main difference is that, upon unloading, residual deformations appear. Materials such as steel combine elastic and plastic laws: for small stresses the behaviour of steel is elastic, until the elasticity limit f_y is reached and a plastic behaviour with hardening appears.

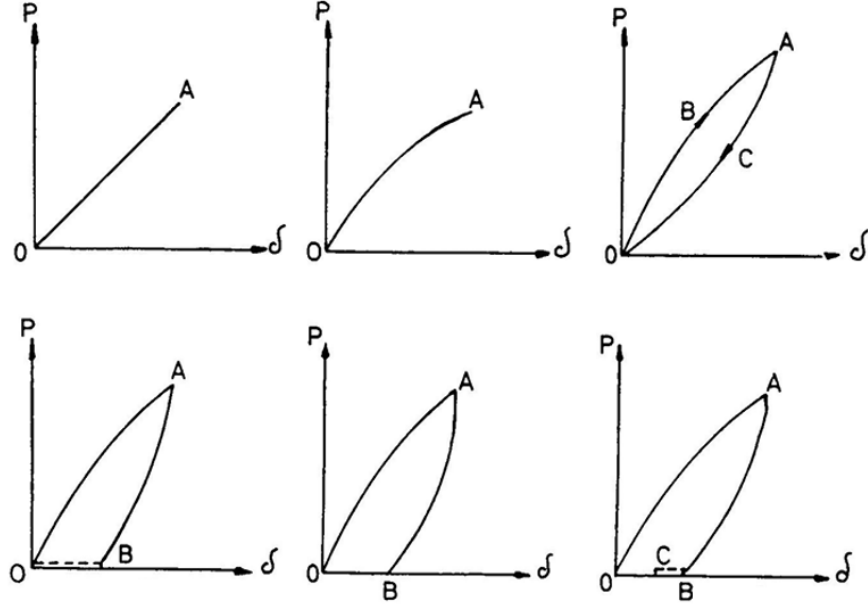


Figure 3.2: Examples of material laws. From left to right, top to bottom: linear elastic, non linear elastic, linear elastic with internal friction, viscoelastic, plastic, partially reactive plastic (Courard (2023)).

In an elastoplastic medium, it is necessary to determine the yield criterion that is used. The possibilities include, for example, the Tresca criterion, the Mohr-Coulomb criterion, the Drucker-Prager criterion or the Hoek-Brown criterion, all described in Panet and Sulem (2022).

The Tresca criterion is only based on cohesion (c [Pa]) and is adapted mainly for cohesive soils. It is given, for principal stresses σ_1 and σ_3 , by Equation 3.7.

$$\sigma_1 - \sigma_3 = 2c \quad (3.7)$$

The Mohr-Coulomb criterion adds the friction angle (ϕ [°]) and is defined by Equation 3.8.

$$\sigma_1 - K_p \sigma_3 = \sigma_c \quad (3.8)$$

where the uniaxial strength σ_c and K_p are given by Equations 3.9 and 3.10.

$$\sigma_c = \frac{2c \cos \phi}{1 - \sin \phi} \quad (3.9)$$

$$K_p = \frac{1 + \sin \phi}{1 - \sin \phi} \quad (3.10)$$

The Hoek-Brown criterion is mostly used for rock masses and is given by:

$$\sigma_1 = \sigma_3 + \sigma_c \sqrt{m \frac{\sigma_3}{\sigma_c} + s} \quad (3.11)$$

where the parameters are based on the Rock Mass Rating (RMR) and on the parameters I_m and I_s which are equal to 1 for an undisturbed rock mass and respectively

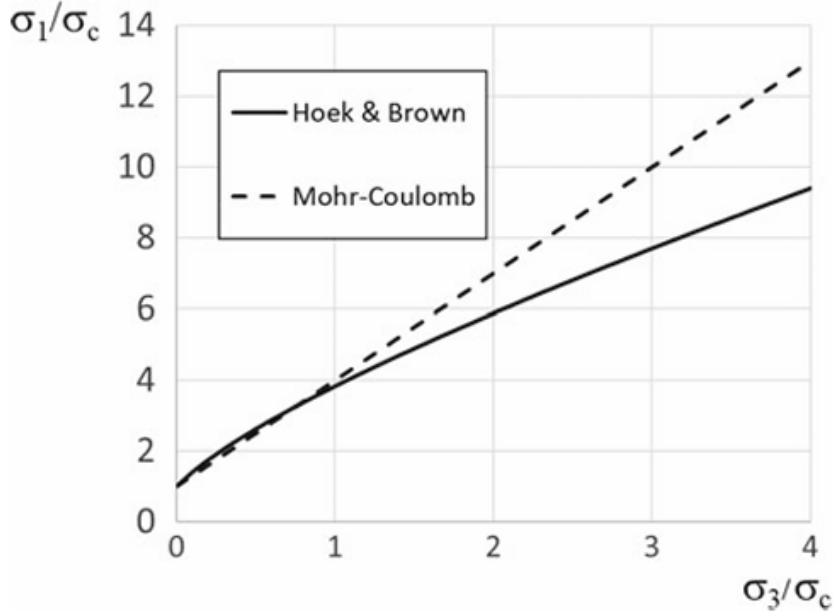


Figure 3.3: Mohr-Coulomb for $K_p = 3$ and Hoek-Brown for $m = 7$ and $s = 1$, Panet and Sulem (2022).

equal to 2 and 1.5 for a disturbed rock mass:

$$m = m_r e^{\frac{RMR-100}{14} I_m} \quad (3.12)$$

$$s = e^{\frac{RMR-100}{6} I_s} \quad (3.13)$$

with m_r being the value of parameter m corresponding to a sound unfractured rock ($RMR=100$). A comparison of the Mohr-Coulomb and the Hoek-Brown criterion is shown in Figure 3.3.

In the principal stresses plane, the Tresca and Mohr-Coulomb criteria are shapes which base is a hexagon. This lack of smoothness can be corrected by considering either the Von Mises (for Tresca, Figure 3.4) or the Drucker-Prager (for Mohr-Coulomb, Figure 3.5) models. With the two first invariants of the stress tensor being:

$$I_1 = \sigma_1 + \sigma_2 + \sigma_3 \quad (3.14)$$

$$J_2 = \frac{1}{6}[(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2] \quad (3.15)$$

The Von Mises criterion is given by (depending on whether the circle is inscribed or circumscribed):

$$\sqrt{3J_2} - k = 0 \quad (3.16)$$

$$k = \sqrt{3}c \quad \text{or} \quad k = 2c \quad (3.17)$$

And finally, the Drucker-Prager criterion is given by (depending on whether the cone is inscribed, circumscribed in compression or circumscribed in tension):

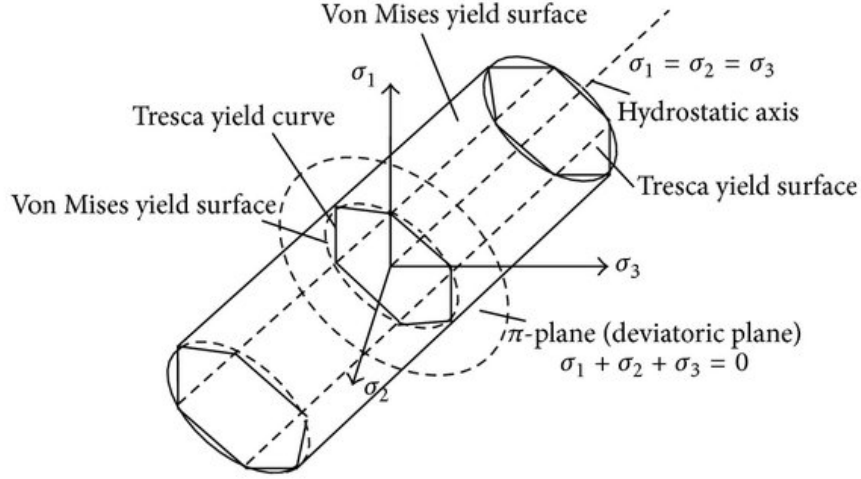


Figure 3.4: Comparison of the Tresca and Von Mises models, Elliotis (2013).

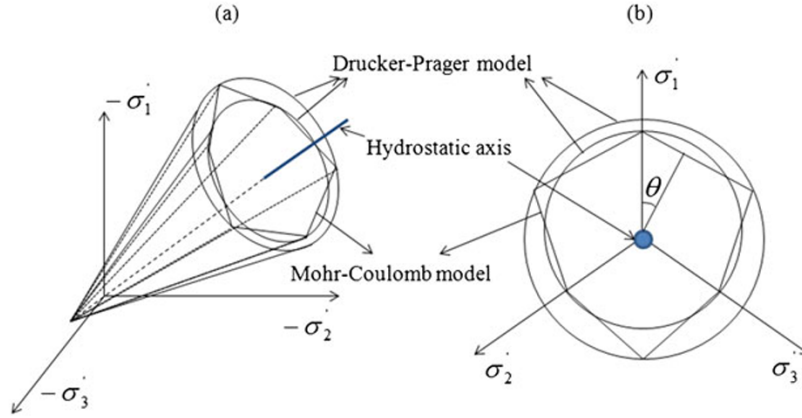


Figure 3.5: Mohr–Coulomb and Drucker–Prager models on (a) the principal effective stress space and (b) on the deviatoric plane, Kim, Sonnenthal, and Rutqvist (2012).

$$\alpha I_1 + \sqrt{3J_2} - k = 0 \quad (3.18)$$

$$\alpha = \frac{\tan \phi}{\sqrt{3 + 4\tan^2 \phi}} \quad \text{or} \quad \alpha = \frac{2\sin \phi}{3 + \sin \phi} \quad \text{or} \quad \alpha = \frac{2\sin \phi}{3 - \sin \phi} \quad (3.19)$$

3.2.2 Hydraulic model

The model used for the hydraulic behaviour of the materials is extensively described in Corman (2023). It is based on the advection of water or gas according to generalized Darcy's law (Equation 3.20):

$$q_i = -\frac{k_{ij}^{\text{int}} k_r}{\mu} \left(\frac{\partial \rho}{\partial x_j} + \rho g_j \right) \quad (3.20)$$

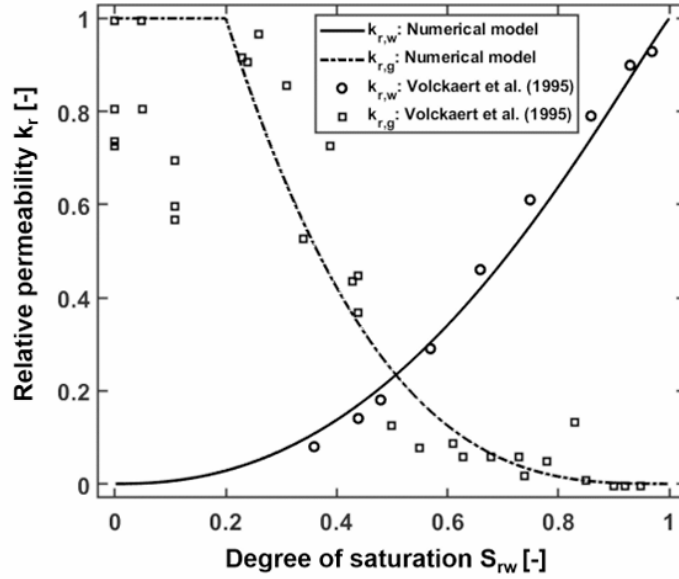


Figure 3.6: Evolution of the relative permeabilities of water and gas, Corman (2023).

where q is the specific discharge, p the pressure, g the gravity of Earth, ρ the density, k_r the relative permeability, μ the dynamic viscosity, k_{ij}^{int} the anisotropic tensor of intrinsic permeability which is a diagonal matrix whose elements are, in order, $k_{//}$, $k_{\perp}$ and $k_{//}$.

The change in water and gas relative permeability depending on water degree of saturation is respectively given in Equations 3.21 and 3.22 and is displayed in Figure 3.6 (Corman (2023)).

$$k_{r_w} = \sqrt{S_{r_w}} \left(1 - (1 - S_{r_w}^{\frac{1}{\mathcal{M}}})^{\mathcal{M}} \right)^2 \quad (3.21)$$

$$k_{r_g} = (1 - S_e)^{\mathcal{L}} \quad (3.22)$$

$$S_e = \frac{S_{r_w} - S_{r_w,res}}{1 - S_{r_w,res} - S_{r_g,res}} \quad (3.23)$$

where k_r is the relative permeability, S_{r_w} the degree of saturation of water, $S_{r_w,res}$ the residual saturation degree of water, $S_{r_g,res}$ the residual saturation degree of gas and $\mathcal{M}$ and $\mathcal{L}$ are numerical parameters fitted to correspond to the experimental data.

The relation between the degree of saturation of water and the capillary pressure is given in Corman (2023) by Equation 3.24 and is illustrated in Figure 3.7.

$$S_{r_w} = S_{r_w,res} + (S_{r,max} - S_{r_w,res}) \left(1 + \left(\frac{s}{P_r} \right)^{\mathcal{N}} \right)^{\frac{1}{\mathcal{N}} - 1} \quad (3.24)$$

where S_{r_w} is the degree of saturation of water, s the capillary pressure (also called suction), $S_{r_w,res}$ the residual degree of water saturation, $S_{r_w,max}$ the maximum degree of water saturation, P_r the parameter of gas entry pressure and $\mathcal{N}$ a numerical parameter fitted to correspond to experimental data.

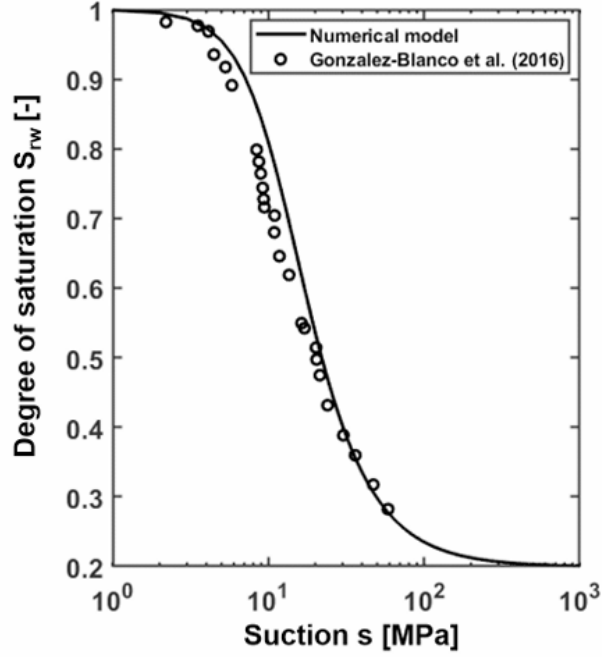


Figure 3.7: Water retention curve, Corman (2023).

3.2.3 Interface model

The contact between two elements is governed by Equations 3.25 and 3.26.

$$\dot{p}_N = K_N \dot{g}_N \quad (3.25)$$

$$\dot{\tau}_T = K_T \dot{g}_T \quad (3.26)$$

where p_N is the normal contact constraint, τ_T is the tangential contact constraint, K_N and K_T are penalty coefficients, $\dot{g}_N$ and $\dot{g}_T$ are the normal and tangential velocities.

The maximum sliding is limited by Equation 3.27.

$$\tau_{max} = \mu p'_N \quad (3.27)$$

where τ_{max} is the maximum shear stress and μ is the Coulomb friction coefficient.

The hydraulic behaviour of the interface is governed by Equations 3.28 and 3.29.

$$f_{w,N} = \rho_w T_w \Delta p_w \quad (3.28)$$

$$f_{w,L} = -\rho_w \frac{k_l}{\mu_w} \nabla p_w + \rho_w g \nabla z \quad (3.29)$$

where $f_{w,N}$ and $f_{w,L}$ are the fluid flows in the normal and longitudinal directions, ρ_w is the density of water, T_w is the transmissivity of water, p_w is the water pressure, k_l is the hydraulic conductivity in the longitudinal direction, μ_w is the dynamic viscosity of water, g is gravitational acceleration and z is the elevation.

Even if the equations related to gas pressure have been given here, that degree of freedom will later be blocked in all simulations.

3.2.4 Parameters

This section presents the parameters used in the constitutive equations for the concrete lining, Boom clay and the interface between those two.

3.2.4.1 Concrete lining

A perfect linear elastic law is used for the mechanical behaviour of concrete. The hydraulic behaviour is based on the water - air seepage - thermal coupled - vapour 2nd gradient law described in Corman (2023). The value of most parameters is given in Table 3.1.

Concrete mechanical law			
Young modulus	E	49.24×10^9	Pa
Poisson ratio	ν	0.2	—
specific mass	ρ	2650	$kg\ m^{-2}$
Concrete hydraulic law			
intrinsic permeability	k_x	4.5×10^{-18}	m^2
porosity	n	0.1	—
water dynamic viscosity	$\mu_{w,0}$	1×10^{-3}	$Pa\ s$
air dynamic viscosity	$\mu_{a,0}$	10^{-5}	$Pa\ s$
water density	ρ_w	1000	$kg\ m^{-3}$
air density	ρ_a	1.17361	$kg\ m^{-3}$
parameter for $k_{r_w,0}$	$\mathcal{M}$	0	—
parameter for $k_{r_a,0}$	$\mathcal{L}$	0	—
residual saturation degree	$S_{r_w,0}$	0	—
field saturation degree	$S_{r_w,field}$	1	—

Table 3.1: Mechanical and hydraulic parameters for concrete.

3.2.4.2 Boom clay

The plasticity criterion that has been used for Boom clay is the Mohr-Coulomb criterion described in Section 3.2.1.1 while the hydraulic law is the same as the one used for concrete.

Parameter	Symbol	Value	Unit
Clay mechanical law			
Young modulus	E	3×10^8	Pa
Poisson ratio	ν	0.125	—
dilatancy angle	ψ	0	$^\circ$
specific mass	ρ	2639	$kg\ m^{-2}$
initial Coulomb angle	ϕ_i	5	$^\circ$
final Coulomb angle	ϕ_f	18	$^\circ$
cohesion	c	0.3	MPa
Clay hydraulic law			
intrinsic permeability	k_x	4×10^{-19}	m^2
	k_y	2×10^{-19}	m^2
porosity	n	0.39	—
water dynamic viscosity	$\mu_{w,0}$	10^{-3}	$Pa\ s$
air dynamic viscosity	$\mu_{a,0}$	10^{-5}	$Pa\ s$
water density	ρ_w	1000	$kg\ m^{-3}$
air density	ρ_a	1.17361	$kg\ m^{-3}$
parameter for $k_{rw,0}$	$\mathcal{M}$	1.1	—
parameter for $k_{ra,0}$	$\mathcal{L}$	3	—
residual saturation degree	$S_{rw,0}$	0.2	—
field saturation degree	$S_{rw,field}$	1	—

Table 3.2: Mechanical and hydraulic parameters for Boom clay.

3.2.4.3 Interface

Fault mechanical law			
penalty coefficient on the contact pressure	K_N	3×10^9	—
penalty coefficient on the shear frictional stress	K_T	3×10^9	—
Coulomb friction coefficient	μ	0.2	—
Fault hydraulic law			
longitudinal intrinsic permeability	k_{l0}	4.5×10^{-18}	m^2
porosity	n	0.39	—
water dynamic viscosity	$\mu_{w,0}$	10^{-3}	$Pa \cdot s$
air dynamic viscosity	$\mu_{a,0}$	10^{-5}	$Pa \cdot s$
water density	ρ_w	1000	$kg \cdot m^{-3}$
air density	ρ_a	1.17361	$kg \cdot m^{-3}$
parameter for $k_{r_w,0}$	$\mathcal{M}$	1.1	—
parameter for $k_{r_a,0}$	$\mathcal{L}$	3	—
residual saturation degree	$S_{r_w,0}$	0.2	—
field saturation degree	$S_{r_w,field}$	1	—
water transverse transmissivity if contact	$T_{t,c}$	10^{-10}	$\frac{m}{s \times Pa}$
water transverse transmissivity if no contact	$T_{t,nc}$	10^{-10}	$\frac{m}{s \times Pa}$

Table 3.3: Mechanical and hydraulic parameters for the contact interface.

3.3 Conclusion

This Chapter presented the necessary mechanical, hydraulic and interface formulations to construct a representative model of the rock and lining behaviours. This basis will be used to perform the numerical simulations.

Chapter 4

Description of the finite element simulation

This chapter describes the geometry and the simulations that will be performed to study the excavation of a nuclear waste storage gallery similar to what has been presented in previous chapters. Throughout the work, the model starts very simply and complexity is progressively added. The excavation of a gallery is a 3D problem, however, such simulations are complex. For this reason, a simpler, 2D, model is built. According to the convergence confinement method discussed in Section 2.2, the link between the advancement of the excavation front and the transverse section that is represented is done by considering a deconfinement rate λ going from 0 to 1 corresponding to the progression of the excavation front from $-2R_{excavation}$ to $+4R_{excavation}$.

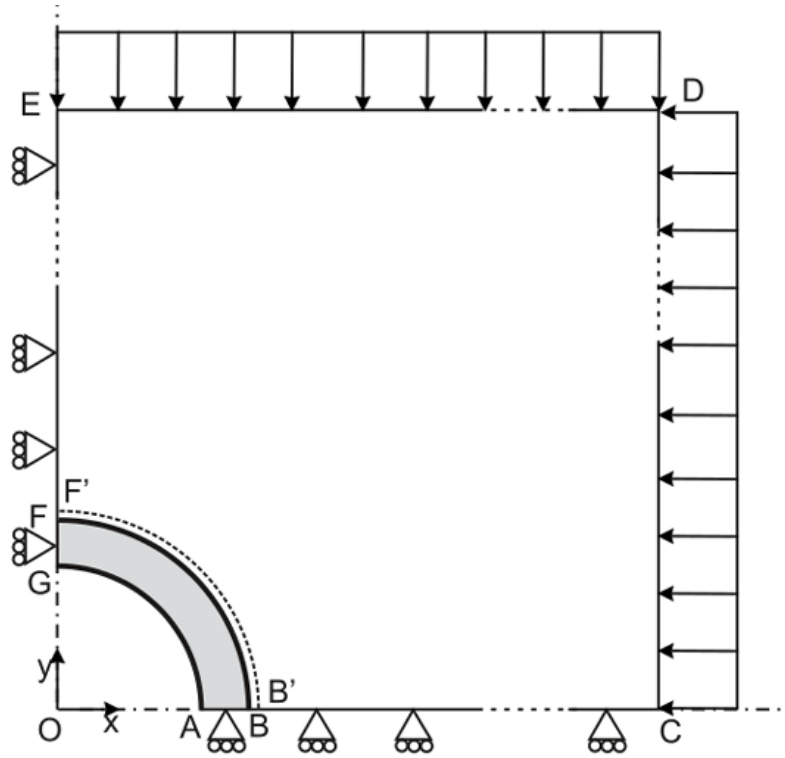
4.1 Reference configuration

4.1.1 Problem statement

In this section, several configurations, which are inspired from the real-life case of the connecting gallery in the Hades URL (from Section 2.1.1), are described. That gallery, which is surrounded by Boom clay, was built at approximately 225 meters to study the possibility of deep geological nuclear waste storage in such conditions. The software *LAGAMINE* is used to model the gallery and the different configurations further explained in the following.

4.1.2 Numerical model implementation

The finite element software *LAGAMINE* is used to model a 100x100m square made of Boom clay showed in Figure 4.1. Because the configuration is symmetrical, it is not necessary to represent the entirety of the problem. For this reason, only a quarter of the plane is constructed and sides EO and OC are respectively fixed along

Figure 4.1: Model used in *LAGAMINE*

y and x. The radius of the excavation (OB') is set to $R_{excavation} = 2.445 \text{ m}$. The concrete lining is not placed directly against the rock. Instead, a gap is left between them (visible in Figure 4.1 with the segments FF' and BB'). The size of this gap will be one of the parameters evaluated later. Although the thickness of the concrete lining will stay constant, the radius of the gallery will change, leading to changes in the value of the gap between the concrete lining and the rock.

The domain is meshed as displayed in Figure 4.2 over a square with sides of 100 meters. As illustrated in Figure 4.2a, the mesh is made of three zones:

- 0 to 10 meters, where the mesh is the finest to allow a better understanding and representation of the impact of the excavation in the zone just around the gallery. This zone is shown in Figure 4.2b and includes the concrete lining with a width of 40 cm and the gap which size will be studied.
- 10 to 50 meters. Although this area is further away from the excavation, it will be demonstrated that the initial state of the rock is still influenced by the excavation of the tunnel, albeit in a limited way.
- 50 to 100 meters, where the mesh is coarser as it is located very far from the excavation and requires less precision when looking at the results.

The mesh is made of 24,143 nodes and 6,120 elements are used to model the concrete lining, the surrounding Boom clay and the interface between those two according to their mechanical and hydraulic behaviour. Five degrees of freedom can be represented: x and y displacement, water pressure, gas pressure and temperature.

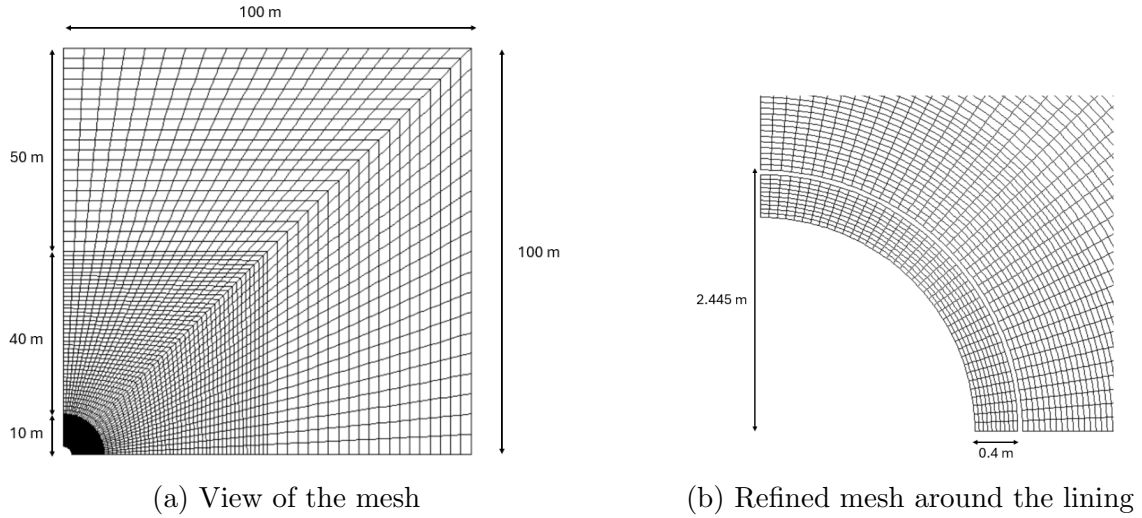


Figure 4.2: View of the different zones of the mesh.

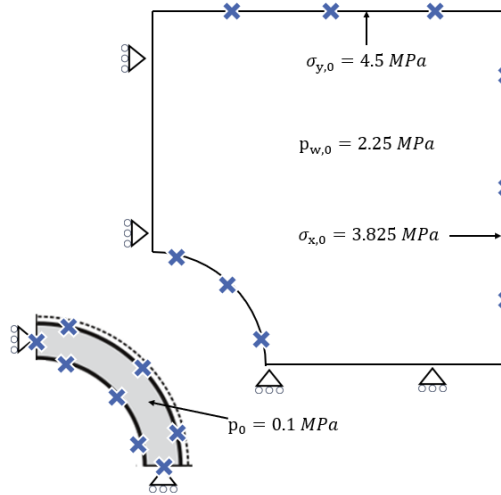


Figure 4.3: Representation of the initial and boundary conditions, with blue crosses meaning that the water pressure is fixed along those borders.

As shown in Figure 4.3, water pressure is initially set at $p_{w,0} = 2.25 \text{ MPa}$ everywhere in Boom clay while it is set at 0.1 MPa inside the lining to match atmospheric pressure. Along the top (ED) and right (DC) borders, distributed stresses respectively equal to $\sigma_{v,0}$ and $\sigma_{h,0}$ are applied to replicate the initial stress state. The vertical stress is equal to $\sigma_{v,0} = 2.25 + p_{w,0} = 4.5 \text{ MPa}$. The horizontal stress $\sigma_{h,0}$ depends on the vertical stress according to the following formula: $\sigma_{h,0} = K_0 \times \sigma'_{v,0} + p_{w,0}$ where K_0 is the coefficient of anisotropy. If the stress state is perfectly isotropic, $K_0 = 1$. In the reference configuration used to model Boom clay in this work, anisotropy is considered and K_0 is set to 0.7. However, the coefficient of anisotropy is rather poorly known and subject to high uncertainty, which explains that the value of $K_0 = 0.7$ will be changed to better understand the effects caused by higher or lower anisotropy. By considering $K_0 = 0.7$, the horizontal stress becomes $\sigma_{h,0} = 3.825 \text{ MPa}$.

For reasons of symmetry, x displacement along the left (EF') border is blocked.

Similarly, the bottom (CB') border is fixed in the y direction.

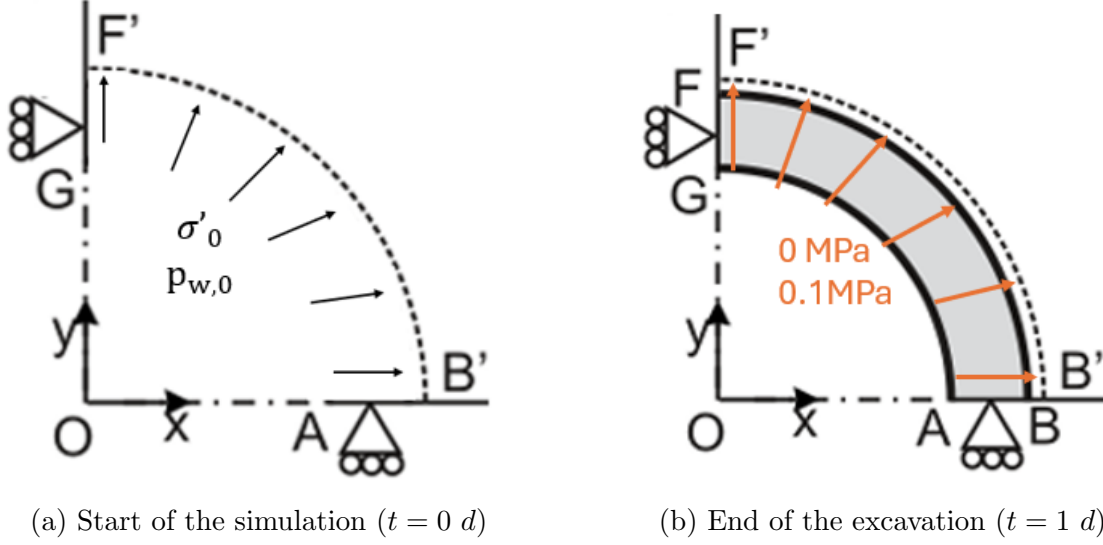


Figure 4.4: Evolution of the boundary conditions along the radius of excavation.

The last border corresponds to the intrados of Boom clay (F'B'). As depicted in Figure 4.4, the stresses and water pressures decrease from their initial value ($\sigma'_{h,0} = 1.575 \text{ MPa}$, $\sigma'_{v,0} = 2.25 \text{ MPa}$ and $p_{w,0} = 2.25 \text{ MPa}$) to $\sigma'_h = \sigma'_v = 0 \text{ MPa}$ and $p_w = 0.1 \text{ MPa}$. That decrease simulates the excavation of the rock to install the lining. At the start of the simulation, the rock has not yet been excavated and the stress and water pressure are still at their original value σ_0 and $p_{w,0}$. After one day, when the excavation is over, the distributed force is equal to 0 as there is no more rock applying pressure on the circular arc B'F'. The pressure applied by water is also equal to 0 but it has been replaced by air which applies atmospheric pressure, approximately equal to 100 kPa . The rate at which these values change can be controlled and is set at a linear change between $t = 0 \text{ d}$ and $t = 1 \text{ d}$ in the reference configuration displayed in Figure 4.5. As explained in Section 2.2, that progressive deconfinement aims to reproduce the coming of the tunnel boring machine in the longitudinal direction. In the reference configuration, the excavation lasts one day. Panet and Sulem (2022) states that a specific section is influenced by the excavation when the excavation front arrives at distance $x = -2R_{\text{excavation}}$ and until it reaches a distance $x = 4R_{\text{excavation}}$. Because the radius of excavation is equal to 2.445 m and the excavation lasts for 24 hours, this corresponds to an excavation speed of $\frac{6R_{\text{excavation}}}{24} = \frac{6 \times 2.445}{24} = 0.61125 \text{ m/h}$. Later, different durations and deconfinement rates will be tested.

The last two degrees of freedom that have not been discussed yet are the temperature and the gas pressure. In this work, they will permanently remain blocked at $T = 289.65 \text{ K}$ and $p_g = 10^5 \text{ Pa}$.

All parameters describing the behaviour of the different elements has been explained in Chapter 3.

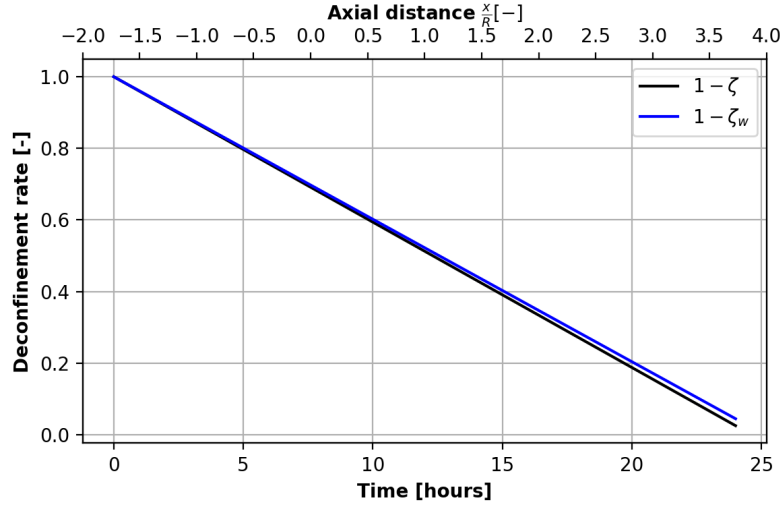


Figure 4.5: Linear rate of excavation with a speed of 0.611 m/h.

4.1.3 Loading implementation

To study the development of the pressure on the lining, three different phases will be simulated: the excavation, the drainage and the stabilization. Excavation corresponds to what has been described in Section 4.1.2, with the excavation of the gallery and the deconfinement of the section over 24 hours.

This is followed by a drainage phase lasting 22 years during which the boundary is moved from the intrados of Boom clay to the intrados of the concrete lining (as illustrated in Figure 4.6). This new boundary conditions represents the state imposed in the gallery, where the ventilation corresponds to atmospheric pressure. During the drainage phase, the gallery is kept ventilated (to allow, for example, people to work or circulate inside). This means that there are no stresses applied on border AG and that pressure is kept at 0.1 MPa to match atmospheric pressure. All previous boundary conditions the x and y displacement or the water pressure on the top, bottom, left and right borders are left unchanged for this phase and the next one.

Finally, the simulation ends with a period of stabilization during which the boundary conditions along AG are deleted and water pressure is let free to evolve. To determine the necessary duration of this phase, several possibilities are tested. Two of them are displayed in Figure 4.7a. It seems that the correct duration would

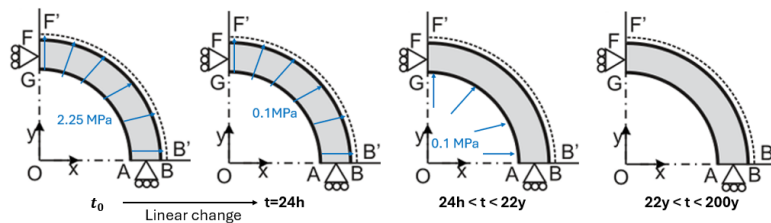


Figure 4.6: Evolution of the boundary conditions across all phases

be the one after which the water pressure is allowed to return to its initial value of $p_{w,0} = 2.25 \text{ MPa}$ after having been blocked at 0.1 MPa for 22 years. However, the time needed to restore these pressures is fairly high and, as shown in Figure 4.7, increases of more than 100 years barely increase the value of water pressure. For this reason, the duration of the stabilization phase is set to 200 years, which seems a good compromise between simulation time and precision. This period of stabilization corresponds to the period during which radioactive waste is stored inside the gallery.

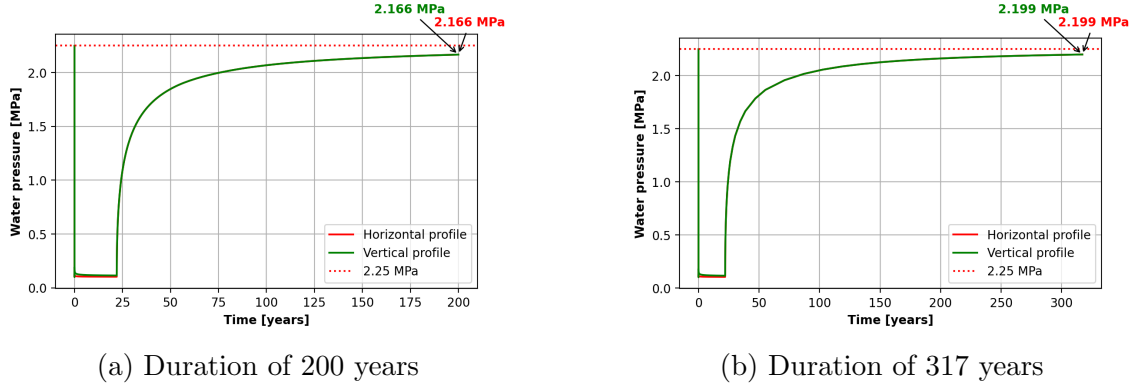


Figure 4.7: Comparison of the pressures acting on the lining after 200 and 317 years.

Overall, throughout this work, the model starts very simply and complexity is gradually added. First, only the excavation phase is considered for a purely mechanical (without water pressures) and isotropic ($\sigma_v = \sigma_h$) case. This configuration is particularly adequate to compare results with the convergence confinement analytical method, thus validating the numerical model. Then, it is possible to introduce anisotropy in the stress state by setting the ratio between horizontal and vertical stress $K_0 = \frac{\sigma_h}{\sigma_v}$ to 0.7. Next, it is possible to introduce an hydro-mechanical behaviour by setting water pressure to 2.25 MPa . Finally, the model becomes complete by combining both the anisotropy and the water pressures.

4.2 Optimisation-oriented parametric study

Once all configurations have been studied, a parametric study can be conducted. First, different gaps are tested. By default, the outer wall of the lining is placed at 2.4 m , meaning that the gap between concrete and Boom clay is equal to 4.5 cm . The new sizes of the gap include 1 cm , 6 cm and 9 cm . As a reminder, the size of the gap in the 2D problem corresponds to the distance from the excavation front at which the lining is placed in the 3D configuration. Further distances from the front (and thus higher values of the gap) lead to higher displacements but lower stresses when the contact with the lining happens.

Afterwards, for all gaps, anisotropy is considered by testing values for the anisotropy coefficient K_0 of 0.5, 0.6, 0.7, 0.8 and 1. The variation of the coefficient of anisotropy is interesting as its exact value at such depths is not well known. Then, for each gap, the initial stress state is modified to simulate lower or greater

depth of burial. From the default value of $\sigma_{v,0} = 2.25 \text{ MPa}$ ($\approx 225 \text{ m}$), values of $\sigma_{v,0} = 2 \text{ MPa}$, $\sigma_{v,0} = 2.5 \text{ MPa}$ and $\sigma_{v,0} = 5 \text{ MPa}$ are tested.

Later, the time of excavation is reduced to half a day or doubled to two days. Changing the excavation time is equivalent to considering different rates of progress of the tunnel boring machine. The shape of the deconfinement curves on the imposed stress and the decrease of water pressure are also modified, from a linear process to a steeper change, happening at different periods of the day.

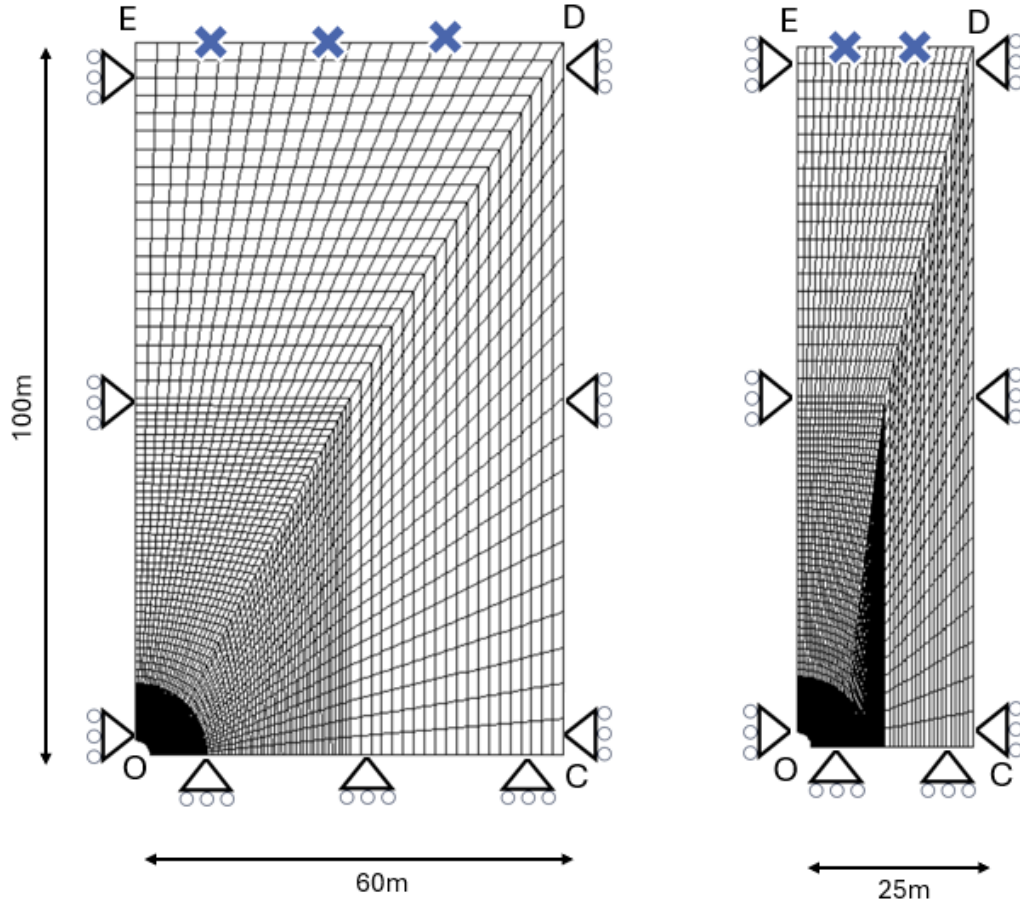


Figure 4.8: New lengths and boundary conditions on sides OC and CD, with blue crosses meaning that the water pressure is fixed along these borders.

The storage of nuclear waste implies the construction of several galleries in the same site. They will be spaced depending on the waste they will contain. The distance between two galleries is projected to be equal to 120 meters for category C waste and 50 meters for category B waste, with different spaces related to the heat generated by these categories of waste. A new model corresponding to these lengths can therefore be established. Additionally, there is now a symmetry with the adjacent gallery with respect to side CD, meaning that it is possible to consider that the horizontal displacement along that side can be fixed while the water pressure that was previously blocked on that border can now be set to evolve freely. All other boundary conditions along the lining or the intrados of Boom clay are identical to the reference configuration. The new dimensions and boundary conditions of these two new models are displayed in Figure 4.8.

4.3 Effect of gravity

In order to establish the previous model, gravity was neglected and the domain was meshed over only a quarter of its real size. A new model can be created to take gravity into account, which means that it is now necessary to represent half of the real gallery.

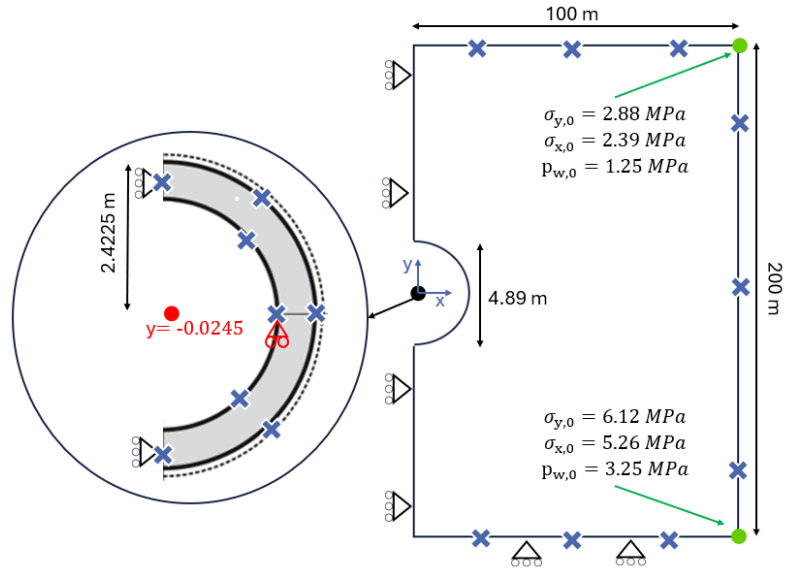


Figure 4.9: New geometry and boundary conditions.

The model is now a 200x100m rectangle made of Boom clay and is represented in Figure 4.9. Because the configuration is symmetrical, it is not necessary to represent the entirety of the problem. For this reason, only half of the plane is constructed and the bottom and left borders are respectively fixed along y and x.

The domain is meshed as displayed in Figure 4.10, three zones can be observed:

- A first rectangle of 20x10 meters, where the mesh is the finest to allow a better understanding and representation of the impact of the excavation in the zone just around the gallery. This zone includes the concrete lining with a width of 40 cm.
- A second rectangle of 100x50 meters. Although this area is further away from the excavation, it will be demonstrated that the initial state of the rock is still influenced by the excavation of the tunnel, albeit in a limited way.
- A last rectangle of 200x100 meters, where the mesh is coarser as it is located very far from the excavation and requires less precision when looking at the results.

Five degrees of freedom can be represented: x and y displacement, water pressure, gas pressure and temperature. The origin of the coordinate system is placed at the black dot in Figure 4.9. At this point, the real depth of this point is equal to 225 meters. As illustrated in Figure 4.9, the stresses and water pressures are now affected

by their y coordinate. At the top of the domain, $\sigma_v = 2.88 \text{ MPa}$, $\sigma_h = 2.39 \text{ MPa}$ and $p_w = 1.25 \text{ MPa}$. Then, they follow a linear increase with depths, until they reach $\sigma_v = 6.12 \text{ MPa}$, $\sigma_h = 5.26 \text{ MPa}$ and $p_w = 3.25 \text{ MPa}$ at $y = -100 \text{ m}$.

The left and bottom borders are still blocked respectively in the y and x directions and water pressure is still fixed on the top and right borders. Similarly to the previous model, temperature is set at $T = 289.65 \text{ K}$ and gas pressure at $p_g = 10^5 \text{ Pa}$ for all points and at all times. Supports in the x direction are added to concrete as well as fixations on water pressure. The borders of the domain are also blocked at the degree of freedom corresponding to water pressure.

During the excavation, a linear decrease of the stress and water pressure on the intrados of Boom clay are applied similarly to what has been done previously before being moved to the inside of the lining during the drainage phase and being deleted during the stabilization phase.

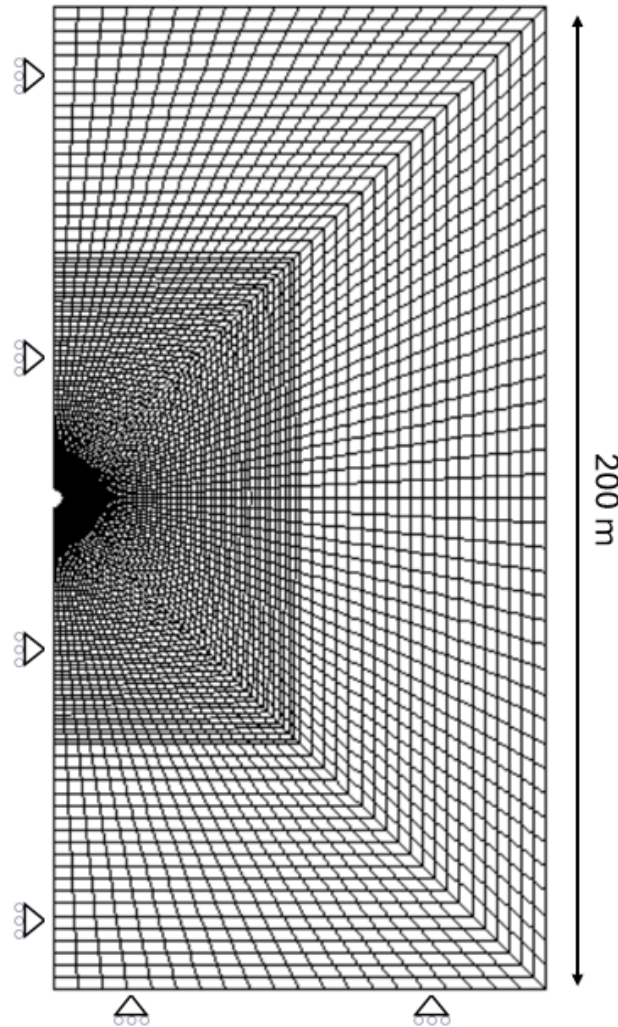


Figure 4.10: View of the mesh

The effect of gravity will be analysed in two steps. First, gravity is not added but the mesh is doubled. In this configuration, the outer radius of the lining is equal to 2.4 meters and is centered at $y = 0$. In this configuration, a support (illustrated in red in Figure 4.9) is added in the middle of the lining to prevent it from displacing

as it has no self weight. Then, when gravity is added, the radius and the centre of the lining are changed in order to have a slight interpenetration between the rock and the lining due to modelling reasons. The new centre is written in red in Figure 4.9 and is located at $y = -0.0245$. In order to keep a reasonable initial gap between the lining and the rock, the outer radius of the lining is also modified to $R = 2.4225$ m.

4.4 Conclusion

The established simulation methodology allows for a systematic analysis of the influence of the main parameters on the pressures exerted on the coating. It constitutes the reference framework for the interpretation of the numerical results.

Chapter 5

2D finite elements analysis

In this chapter, the results obtained from the models defined previously are analysed. The reference configuration is first examined with progressive degrees of complexity, before moving to a variation of the parameters.

5.1 Reference configuration

In this section, a mechanical behaviour is first supposed. Then, anisotropy and hydraulic behaviour are added before examining the stresses appearing in the lining.

5.1.1 Mechanical behaviour

Before moving to a more complete representation of the problem, a configuration without hydraulic behaviour is studied by blocking the degree of freedom associated to water pressure at every node. The stresses considered are therefore not total but effective stresses.

Because there is no hydro-mechanical behaviour, the only phase that can be simulated is the excavation phase, which lasts for one day.

5.1.1.1 Isotropic state

First, it is possible to compare the numerical results obtained for a simpler version of the problem in *LAGAMINE* with theoretical results obtained with the convergence confinement method. Because the theoretical approach does not allow for anisotropy, the initial stress state is set at $\sigma_0 = 2.25 \text{ MPa}$ with $K_0 = 1$, meaning that the stress is the same vertically and horizontally (as $\sigma_h = K_0 \times \sigma_v$). Moreover, in order to conform to the convergence confinement approach, the Coulomb angle is set to be constant at $\phi_i = \phi_f = 18^\circ$.

Because the gallery is circular, it is interesting to consider not the vertical and

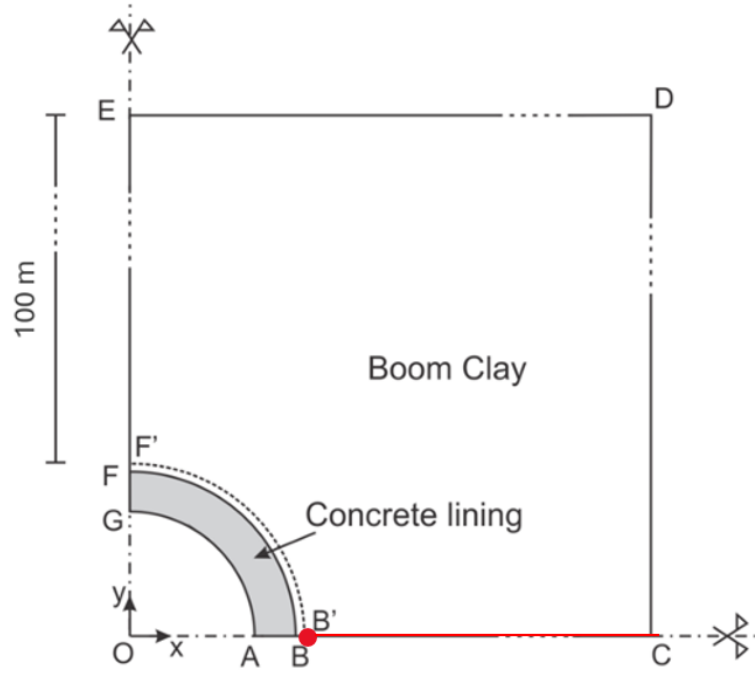


Figure 5.1: Representation of the horizontal profile and the first point of Boom clay.

horizontal stresses σ_v and σ_h but rather the radial and orthoradial stresses σ_r and σ_θ . In order for it to be more practical, it is possible to study only the horizontal profile (represented by a red line in Figure 5.1) where $\sigma_v = \sigma_\theta$ and $\sigma_h = \sigma_r$. The first point of this profile (placed at abscissa $x = 2.445 \text{ m}$) will also be studied and is marked by a red dot in Figure 5.1.

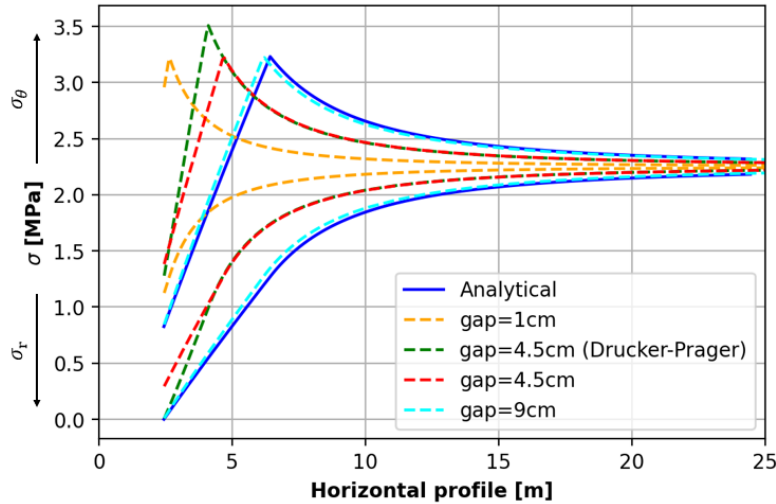


Figure 5.2: Stresses along the horizontal profile, analytical and numerical comparison. For a same colour, the top curve corresponds to σ_θ and the bottom one to σ_r .

Figure 5.2 shows the comparison between the theoretical curve in continuous line and several numerical dotted lines for different gaps and plasticity criteria. The effect of the plasticity criteria is greatly illustrated with the green curve. When, instead of Mohr-Coulomb, a Drucker-Prager criterion is used, the stress is allowed to reach higher value, thus distancing it from the theoretical values. This is explained

by Section 3.2.1.1 and Figure 3.5 where it can be seen that the Mohr-Coulomb criterion is inside the Drucker-Prager criterion.

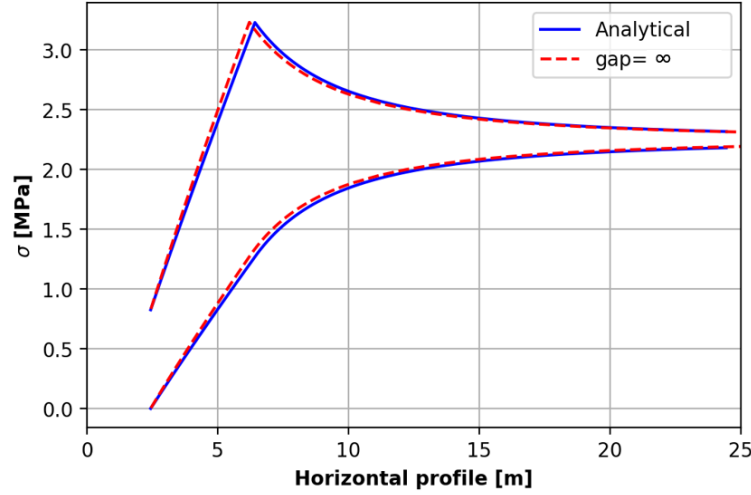


Figure 5.3: Comparison between the analytical curve and a case without contact

Most of the curves show different gap sizes (1 cm, 4.5 cm and 9 cm). The lower the gap, the further the curve is from the theoretical curve. With a gap of 9 cm, the results obtained through *LAGAMINE* and the convergence confinement method are almost identical, comforting the validity of the simulation with a finite element code. The reason for which bigger gaps are closer to the theoretical curves is that, in the analytical approach, it is not possible to obtain a contact between the lining and the curve, it is only possible to look at the intersection of the lining and rock curves but each has no influence on the other. For this reason, Figure 5.3 illustrates a numerical case where the gap has been set to 2.345 m (equivalent to an infinite distance), in order to make sure that there is no contact. In that configuration, it can be seen that both the theoretical and numerical curves superpose almost perfectly.

It is also worth observing the first point on the profile for the σ_r line in Figure 5.2. The lower the gap, the higher the value of the stress for that point. For the convergence confinement σ_r line, that first point is at $\sigma_r = 0 \text{ MPa}$. This is due to the fact that, for a bigger gap, the contact between the concrete lining and the rock happens later, leading to lower stresses when the equilibrium is found but also to higher displacements. This can be seen in Figure 5.4 where the final displacement that is reached over time increases with the size of the gap. Despite the big difference in size between the 9-cm gap and the 2.345-m gap, their curves are almost identical. This is because, in the case with a gap of 9 cm, the contact happens just minutes before the end of the simulation, which means that the size of gap from which there is no contact is very close to 9 cm. However, if there is no contact, it does not matter if the lining is far or near from the rock as there is no time left to continue increasing the displacement. For the distance left to be relevant, it would be necessary to increase the excavation time so that the rock has time to reach the lining.

Regardless of the size of the gap, Figure 5.2 displays that both the radial and orthoradial stresses converge to their initial value of $\sigma_0 = 2.25 \text{ MPa}$ when going further from the excavation. This simply means that the excavation of the gallery

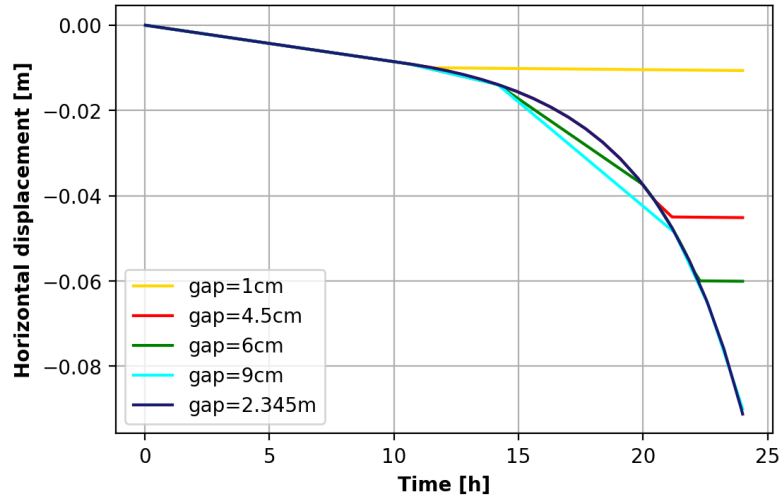


Figure 5.4: Comparison of the displacements over time for different sizes of gap.

creates a "disturbed" zone in the area around it. When located far enough from the gallery (at about 20-25 meters), the stress state in the ground does not vary at all. The comparison between the different sizes of gap reveals that the lower the gap is, the nearer the rock seems to keep its initial stress state.

Figure 5.5 shows a comparison between the finite element code (in red) and the convergence confinement approach (in blue) for the relation between the pressure on the lining (in dotted line) or the stress in the rock (in continuous line) and the associated displacement of the lining or the rock. It confirms what has been said previously: it is possible to find numerical and analytical curves that superpose almost perfectly.

When the rock moves, it eventually comes into contact with the lining, thus applying pressure to it. Taking Figure 5.5a as an example, it can be seen that, for low displacements (up to 4.5 cm, which is the distance between the rock and the concrete lining), the lining curve stays at a pressure $p = 0 \text{ MPa}$ as there is still an empty gap between the lining and the rock. When the rock reaches sufficiently high displacements (4.5 cm, the size of the gap), it comes into contact with the lining and applies pressure to it. This increase in the pressure in the lining continues until the rock curve is reached.

The comparison between Figures 5.5a and 5.5b displays that, for bigger gaps, the contact happens for lower stresses but higher displacements. If the lining is placed so far that there is no contact (as in Figure 5.5c where the lining is at more than 2 meters from the rock), the displacements of the rock reaches much higher values as nothing is there to stop it. In contrast, the stress is equal to zero, as the rock is free to deform and move when subjected to forces.

In Figure 5.5, the main difference is that the numerical curves for the rock and the lining always stop at the same point whereas the theoretical curves simply briefly intersect. This is because it is not possible to consider what happens after the contact in the analytical approach and both curves just keep the same behaviour even after the contact, as the formulas they are based on are independent of each

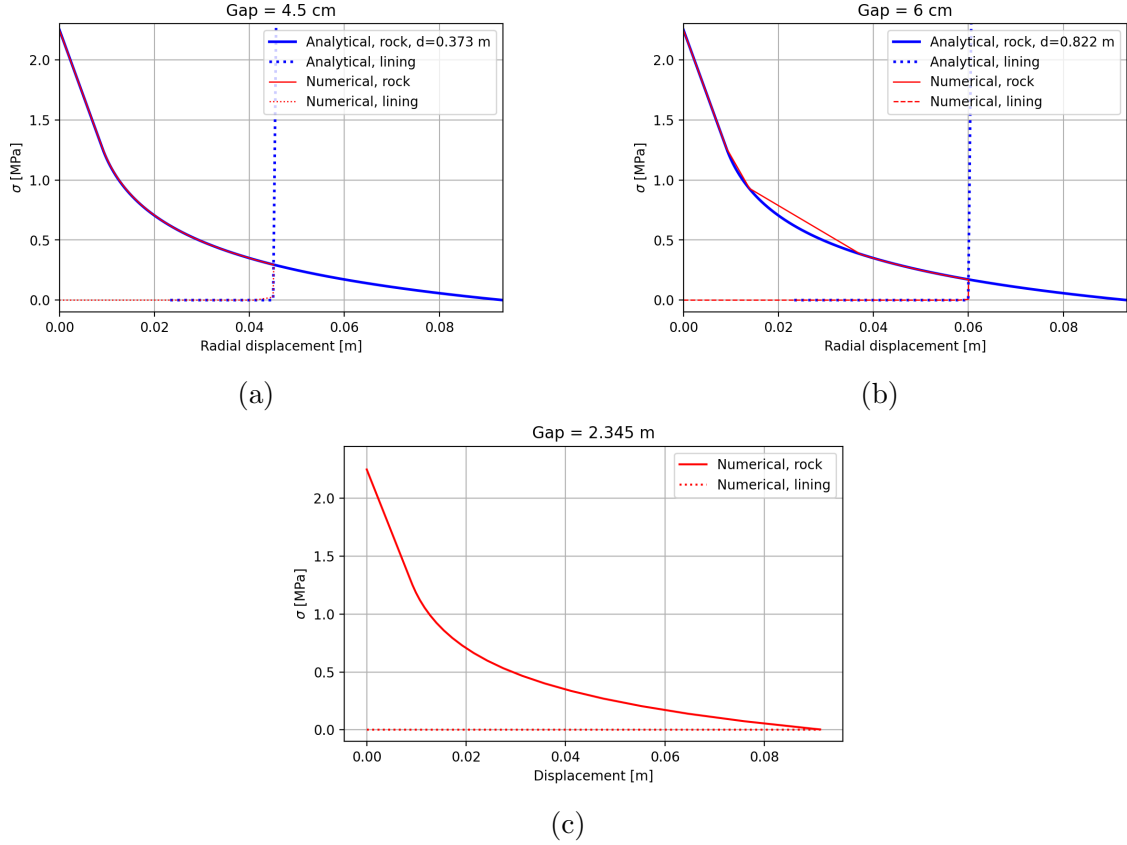


Figure 5.5: Rock and lining curves for different gaps, comparison between numerical and theoretical results.

other (as explained in Section 2.2).

In the numerical simulation, the contact really happens and is characterized by an equilibrium between the pressure on the lining and the pressure in the first point of the horizontal profile, which is the one displayed in the rock curve. The limitation of the current numerical approach is that it is a 2D simulation. This means that it is not possible to take into account the distance d from the excavation front at which the lining is placed. This can be solved by considering a 2D axisymmetric model in the longitudinal direction (i.e. the direction in which the tunnel boring machine is moving) or by creating a full 3D model.

Indeed, in reality, the lining can not be activated as soon as the excavation is over, because the TBM has to advance before the following train can install the concrete elements. However, modifying that distance to the excavation front in the analytical calculations has the effect of translating the lining curve. It is then possible to search for the distance d that enables the analytical and numerical curves to superpose. This distance d is dependent on the size of the gap. This leads to $d = 37,3 \text{ cm}$ when the gap is equal to 4.5 cm or $d = 82.2 \text{ cm}$ for a gap of 6 cm . This shows that it is possible to link the distance to the excavation front to the size of the gap, thus establishing a relation between parameters that are related to different directions as the size of the gap in the transversal plane seems to play the role of the distance to the excavation front in the longitudinal plane.

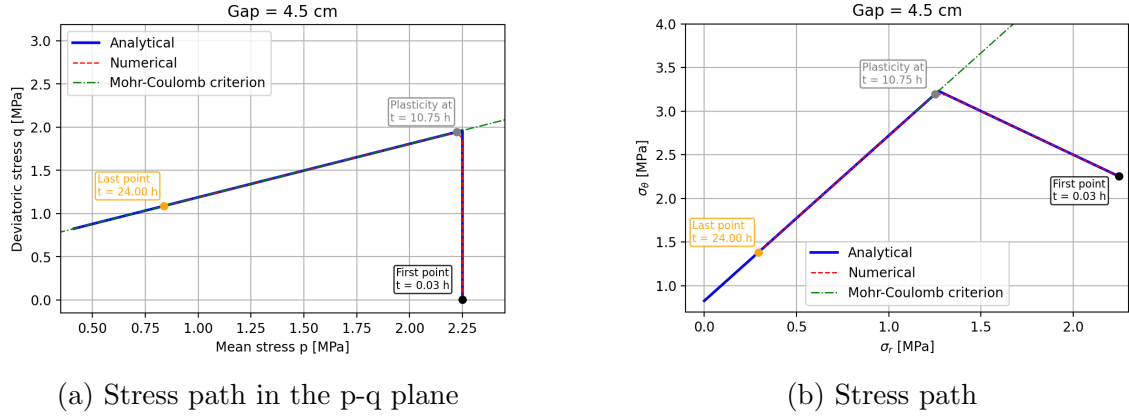


Figure 5.6: Stress paths at the first point of the horizontal profile.

Still at the first point of the horizontal profile, it is possible to look at the path followed in the plane of the radial and orthoradial stresses as well as in the plane of the mean stress $p = \frac{\sigma_r + \sigma_\theta}{2}$ (Figure 5.6a) and deviatoric stress $q = \sigma_\theta - \sigma_r$ (Figure 5.6b). Again, both analytical and numerical results are identical. In Figure 5.6a, the mean stress stays constant until plasticity develops. This can be explained with Figure 5.6b, as it can be seen that σ_r is decreasing as the same rate as σ_θ is increasing. This also explains why the deviatoric stress increases. After 10.75 h, the Mohr-Coulomb criterion is reached and there is development of a plastic zone. Until the end of the excavation phase, the followed path is identical to the plasticity criterion.

5.1.1.2 Anisotropic state

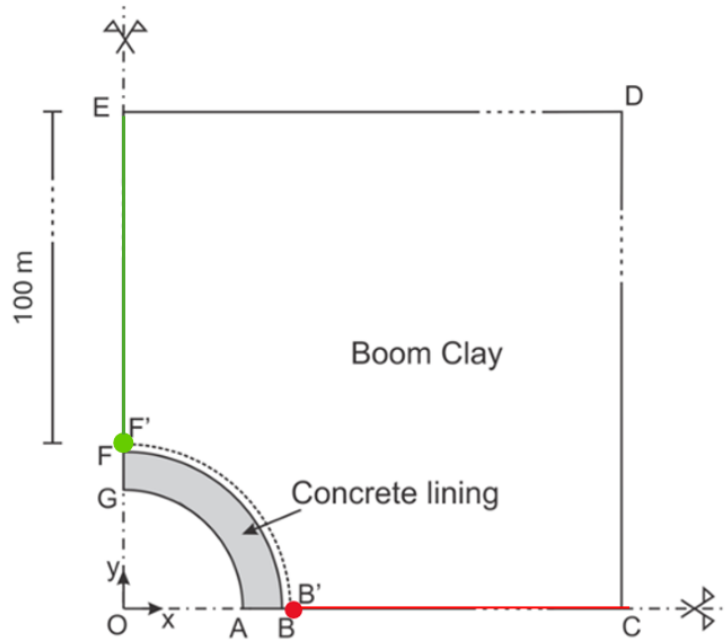


Figure 5.7: Representation of the horizontal and vertical profile as well as the location of the first two nodes of those profiles.

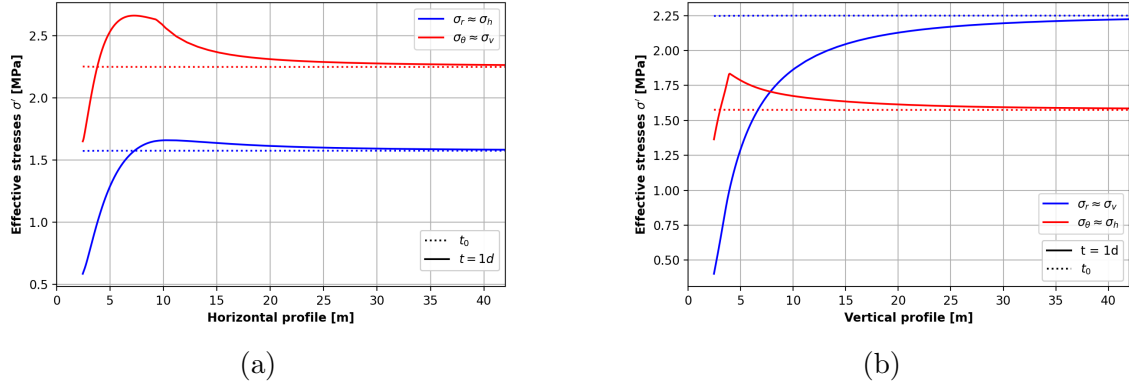


Figure 5.8: Radial stress σ_r and orthoradial stress σ_θ along horizontal and vertical profiles for a gap of 4.5 cm.

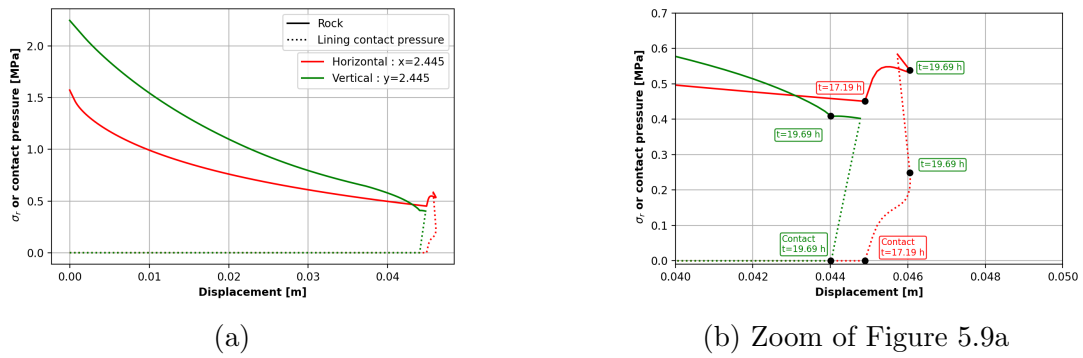


Figure 5.9: Rock and lining curves for the horizontal and vertical profiles with a gap of 4.5 cm.

Now that the mechanical and isotropic case has been compared to the theoretical results, an anisotropic stress state is considered. This is done by choosing $K_0 = 0.7$, which gives the following initial stress state:

$$\sigma_v = 2.25 \text{ MPa} \quad \sigma_h = K_0 \times \sigma_v = 1.575 \text{ MPa}.$$

With the anisotropy, a hardening of the Coulomb angle ϕ is introduced in order to better represent the real behaviour of Boom clay. The Coulomb angle is now characterized by an initial value of 5° and a final value of 18° instead of a constant $\phi = 18^\circ$ value.

Due to the anisotropy, a second profile must be introduced for analysis. This profile, shown in green in Figure 5.7, is chosen to be vertical for the same practical reasons as before: along this profile, the vertical stress (σ_v) equals the radial stress (σ_r), and the horizontal stress (σ_h) equals the orthoradial stress (σ_θ). The stress paths and the characteristic curves will again be evaluated at the first point of each of these profiles, which are marked by a dot in Figure 5.7.

The radial and orthoradial stresses are displayed along the horizontal and vertical profiles in Figure 5.8. In this anisotropic configuration, the stresses located far from the gallery converge towards different values which correspond to the initial stress state. While the vertical stress curve still goes to $\sigma = 2.25 \text{ MPa}$, the horizontal stress now goes to a lower value, $\sigma = 1.575 \text{ MPa}$.

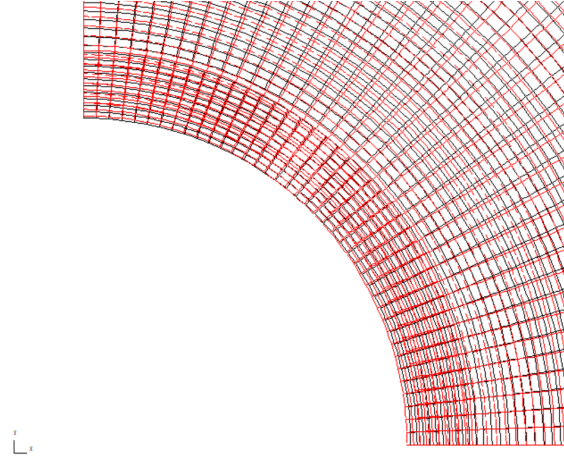


Figure 5.10: The deformed configuration at the end of the excavation phase shows an ovalization of the lining, amplified by a factor of 8.

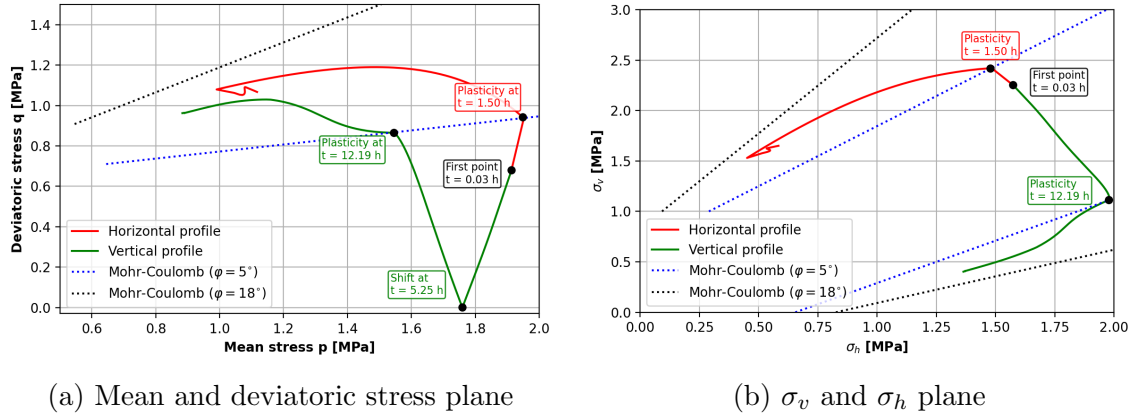


Figure 5.11: Stress path for the first point of the vertical and horizontal profiles.

As the behaviour of the rock is not the same in both directions anymore, there are now two different rock and lining curves to consider: one horizontal and the other vertical.

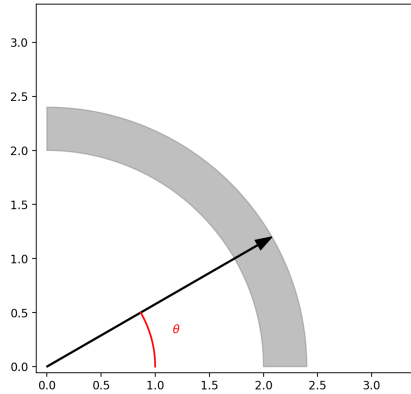
Figure 5.9a illustrates that the final pressure at which the equilibrium is found is higher in the horizontal profile than in the vertical profile (respectively $p = 0.58 \text{ MPa}$ and $p = 0.4 \text{ MPa}$). This is explained by Figure 5.9b which shows that the contact first happens for the horizontal profile at 17.19 h for a displacement equal to the size of the gap, 4.5 cm . The earlier contact leads to a final pressure that is also higher on the horizontal profile. At $t = 19.69 \text{ h}$, when the contact happens on the vertical profile, it can be seen in Figure 5.9b that both the horizontal rock and lining curves have displacements that are decreasing. From the moment of the second contact, the rock and lining that were moving towards the centre of the excavation actually begin to move backwards.

This backwards movement is due to the redistribution of stresses during the excavation and development of higher orthoradial stresses. This causes an ovalization of the liner, which pushes the rock on the horizontal profile. The deformed shape of the liner is displayed in Figure 5.10, which depicts the final geometry of the gallery

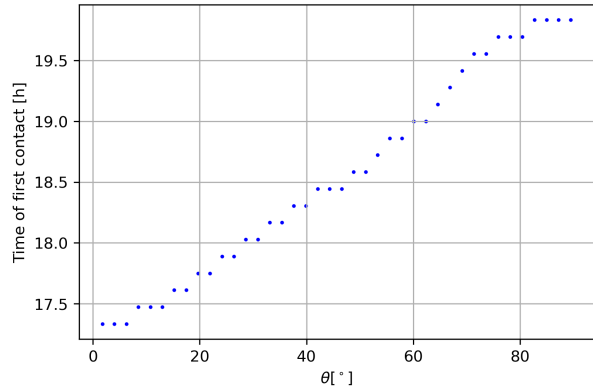
at the end of the excavation phase, after the ovalization has taken place.

As can be seen in Figure 5.11, there are now different paths for the horizontal and vertical profiles due to anisotropy. Because of the newly introduced hardening of the Coulomb angle ϕ , the Mohr-Coulomb criterion is not a straight line anymore but is rather evolving throughout the excavation. For the sake of clarity, only the initial and final values of that criterion are displayed in Figures 5.11a and 5.11b.

Unlike the isotropic case, Figure 5.11a illustrates that the mean stress varies. Indeed, due to anisotropy, the horizontal and vertical stresses still evolve in opposite directions but simply not at the same rate anymore. In the horizontal profile, the increase of the vertical stress ($\sigma_v = \sigma_\theta$) and the decrease of the horizontal stress ($\sigma_h = \sigma_r$) lead to an increase of the deviatoric stress in the p-q plane, which ultimately meet the Mohr-Coulomb criterion. At this moment, it develops plasticity ($\approx 1.5 h$ after the start of the excavation). In the vertical profile, the deviatoric stress evolves until it also enters plasticity after $\approx 12 h$. The deviatoric stress in the vertical profile initially decreases before increasing again. This behaviour is explained in Figure 5.11b, which shows that, at the start of excavation, the principal stress is σ_v (since $\sigma_v > \sigma_h$). However, after approximately 5.25 hours, σ_h surpasses σ_v and becomes the principal stress, leading to the observed shift in the deviatoric stress evolution.



(a) The x-axis of Figure 5.12b corresponds to the angle shown here.



(b) Time of first contact on the lining depending on the angle (0° corresponding to the horizontal profile and 90° to the vertical profile)

Figure 5.12: Polar coordinates and time of first contact.

As said earlier and illustrated more precisely in Figure 5.12, the contact happens progressively from the horizontal to the vertical profile over time. It seems surprising as the vertical stress is greater than the horizontal (due to $K_0 = 0.7$), which would lead to believe that the displacement would be higher in the vertical direction, in turn leading to an earlier contact. This is indeed what happens at first, as can be seen in Figure 5.13 where, up until approximately 12.75 hours, the vertical profile converges faster. However the two curves end up by crossing shortly before the contact. That acceleration in the horizontal profile displacement can be explained by looking back at Figure 5.11 which illustrates the stress paths followed by the two profiles. A great difference is that the time at which plasticity is reached is much

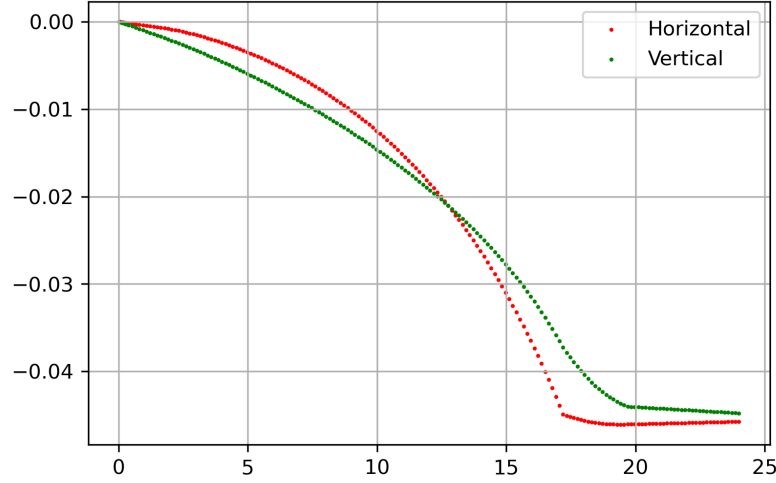


Figure 5.13: Displacement of the first point of the horizontal and vertical profiles over time (< 0 towards the centre of excavation, > 0 towards the outside).

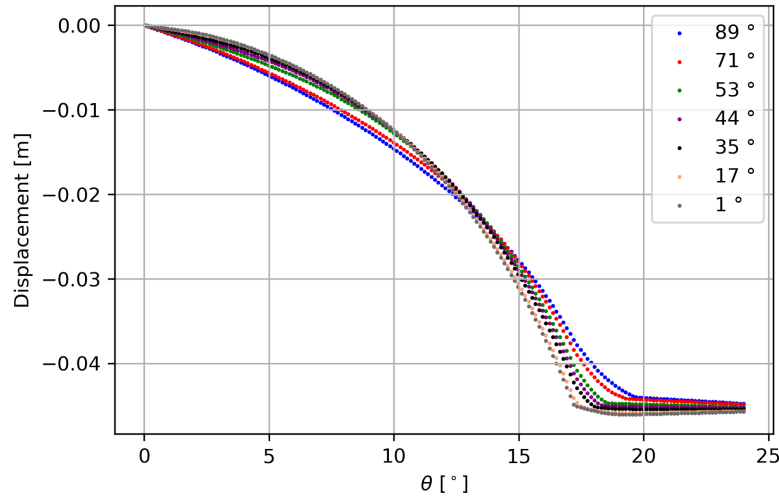


Figure 5.14: Displacement of the first point of the horizontal and vertical profiles over time (< 0 towards the centre of excavation, > 0 towards the outside).

lower horizontally (after only 1.5 h against 12.19 h). However, the development of plasticity comes with increased strains, which explains the earlier contact despite the initially slower convergence and the lower horizontal stress.

Figure 5.14 displays that the change in the convergence speed happens in a similar way for all orientations. The closer to the vertical profile, the faster the convergence before the inversion and the slower the convergence after the inversion.

5.1.2 Hydro-mechanical behaviour

The hydraulic behaviour is now considered by unlocking the degree of freedom associated to water pressure and setting the initial value at $p_w = 2.25 \text{ MPa}$. First, an isotropic case is studied to isolate the effect of water pressure before moving to

the complete model with anisotropy and water pressure, which better represents the real configuration of the problem. For most following figures, the reference case is used, i.e. a gap of 4.5 cm with a hardening Coulomb angle ($\phi_i = 5^\circ$ and $\phi_f = 18^\circ$) and an anisotropy coefficient $K_0 = 0.7$ when anisotropy is introduced.

5.1.2.1 Isotropic state

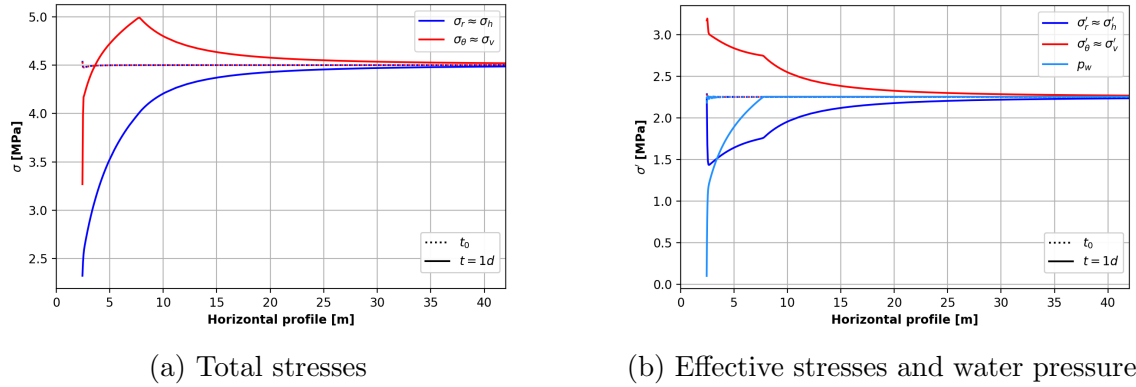


Figure 5.15: Total and effective stress along the horizontal profile for a gap of 4.5 cm.

Figure 5.15a illustrates that, far from the excavation where the ground is still at its initial state, the total stress is equal to 4.5 MPa.

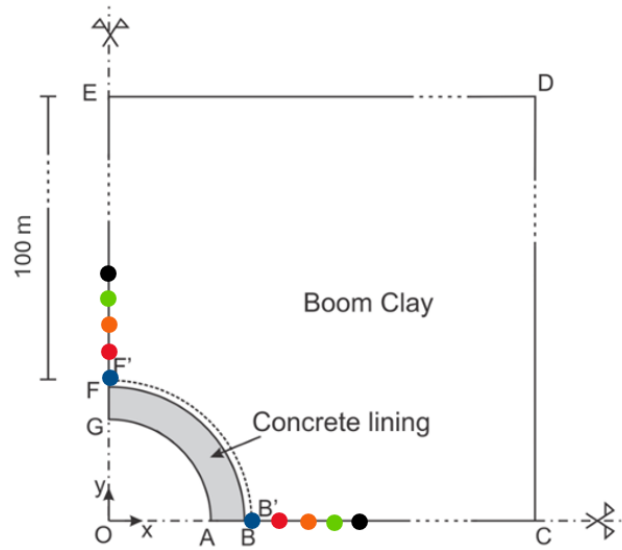


Figure 5.16: View of the different points along the horizontal and vertical profiles.

The evolution of water pressure over time is displayed in Figure 5.17 at five points chosen over the longitudinal profile (Figure 5.16). The first point is located right at the start of the horizontal profile while the last point is at a distance of 8.5 m. It can be seen that, at the first point, the evolution of the pressure is identical to the imposed boundary condition on the radius of excavation and is a linear decrease from the initial pressure of $p_{w,0} = 2.25$ MPa to $p_w = 0.1$ MPa (the value of the atmospheric pressure) over a period of one day. The time at which pressure starts

decreasing is variable, the closer to the excavation, the sooner the pressure starts decreasing. Indeed, because the point is further from the boundary condition, some time is needed for the perturbation to arrive to that point. For closer points, it can also be seen that the pressure is lower than for further points ($p_w \approx 1.5 \text{ MPa}$ at 3.368 m but $p_w = 2.25 \text{ MPa}$ at 8.5 m). As for the stresses, there is a distance from which the initial state is conserved and the rock is not disturbed.

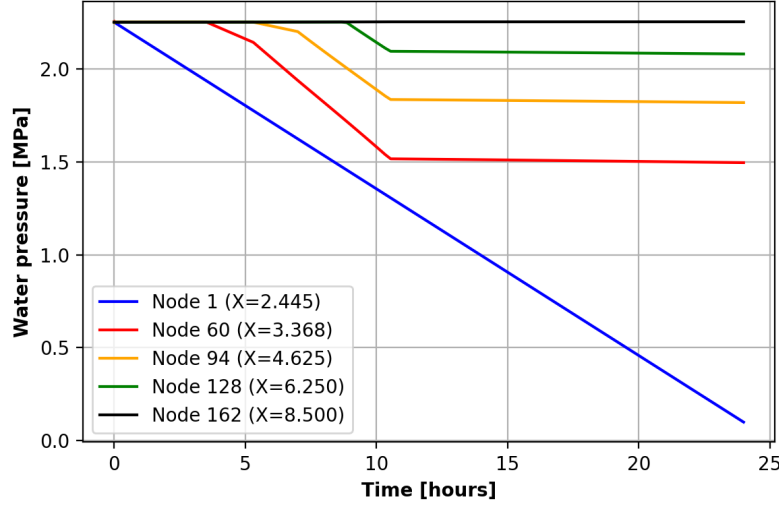


Figure 5.17: Water pressure over time at five locations along the horizontal profile.

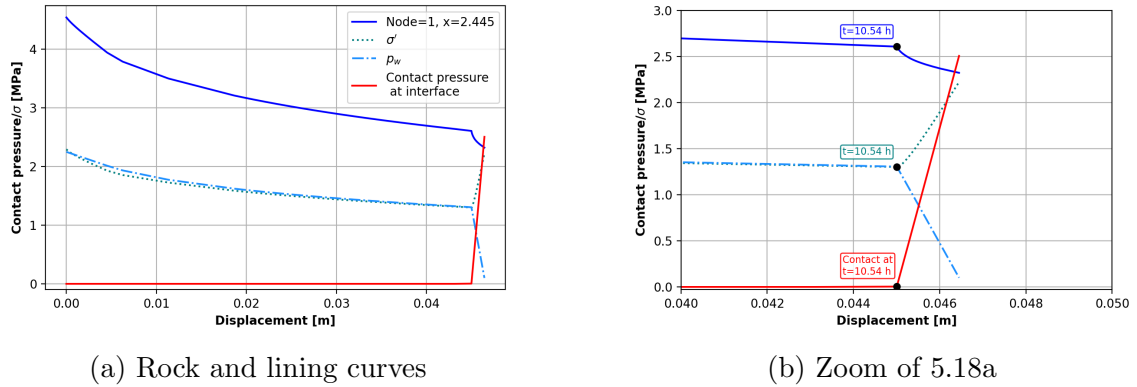


Figure 5.18: Rock and lining curves at the first point of the horizontal profile for a gap of 4.5 cm .

Figure 5.18a shows the rock and lining curves and the distribution of the effective stress and water pressure in the rock curve. Compared to the same configuration but without water pressures, the contact happen much sooner, after only approximately 10 hours instead of 17 hours. Before the contact, the effective stress and the water pressure are near identical but, after the contact, the water pressure drops while the effective stress increases almost as much. The reason is that, after the contact, the evolution of water pressure becomes identical to the boundary condition, which is a linear decrease of $(1 - \zeta_w)$. This is confirmed by Figure 5.19 which displays the rock characteristic curve for a point located 50 centimetres further, at $x = 3 \text{ m}$. It shows that, further in the profile, the effective stress and the water pressure stay closer.

Regarding the stress path illustrated in Figure 5.20a, the main difference between

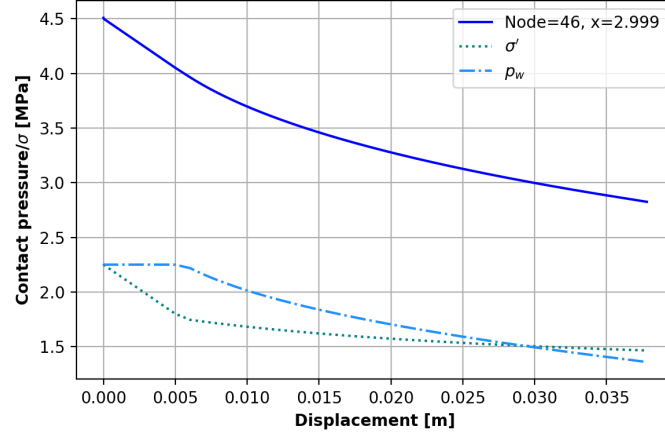
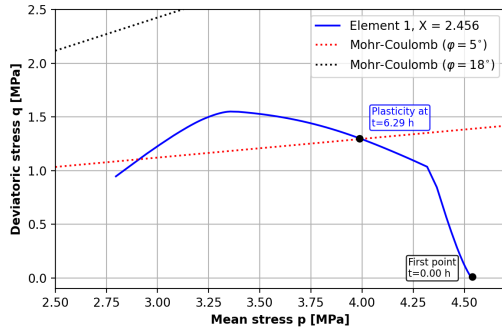
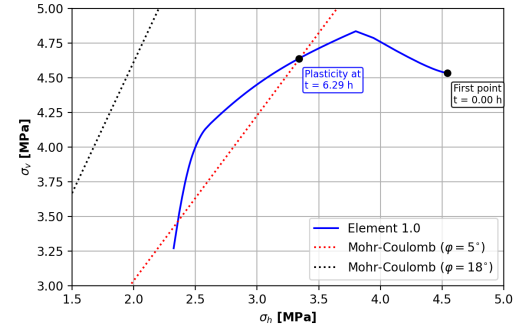


Figure 5.19: Rock curve and its decomposition in water pressure and effective stress at 3 meters in the horizontal profile for a gap of 4.5 cm.



(a) Mean and deviatoric stresses plane.



(b) Horizontal and vertical stresses plane.

Figure 5.20: Stress paths for the first point of the horizontal profile.

the cases with and without water pressures is the time at which plasticity develops. Plasticity appears after respectively 6.3 hours and 5.2 hours. At the end of the excavation, the point leaves plasticity and comes back to the elastic domain, due to a decrease in both the horizontal and vertical stresses.

5.1.2.2 Anisotropic state

As discussed in Chapter 4, two new phases of 22 and 200 years are introduced now that the model considers both anisotropy and water pressures. The distribution of stresses and water pressure along the horizontal profile during excavation illustrated in Figure 5.21 is very similar to what has been presented for other configuration. For points close to the tunnel (up to ≈ 30 m), it can be seen that the value of the stresses and the water pressure continue increasing during drainage phase where they reach their maximum. Figure 5.21b displays that, around ≈ 10 m along the profile, the water overpressure present at the end of the excavation has dissipated at the end of the drainage. After 200 years, the pressure is almost back at its initial value, except near the gallery even if still very close to 2.25 MPa.

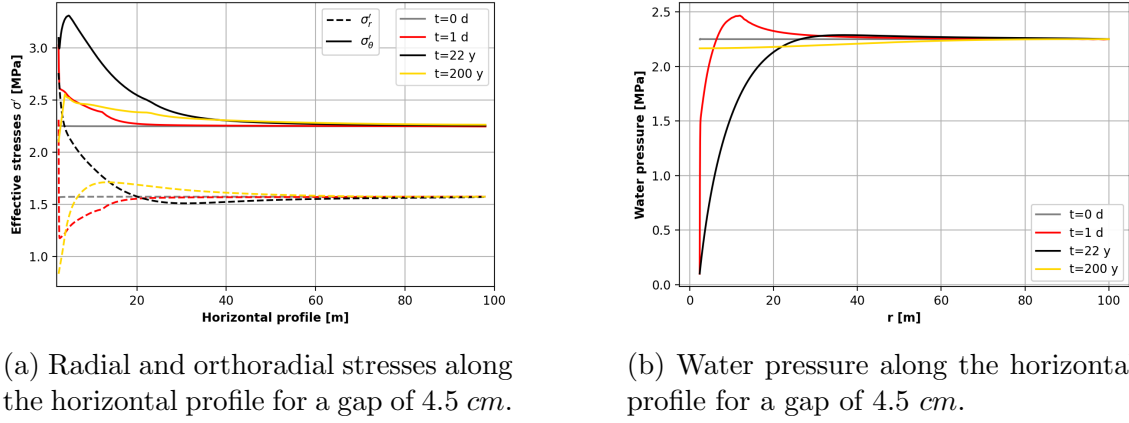


Figure 5.21: Stresses and water pressure along the horizontal profile.

In addition to Figure 5.21b, Figure 5.22 shows the evolution of water pressure over time. The first sudden drop is due to the excavation and the boundary condition which causes a linear decrease from 2.25 to 0.1 MPa. During stabilization phase pressure is kept at 0.1 MPa to match air pressure. When the boundary condition is removed, water pressure is free to go back to its initial state, even if the change is very slow.

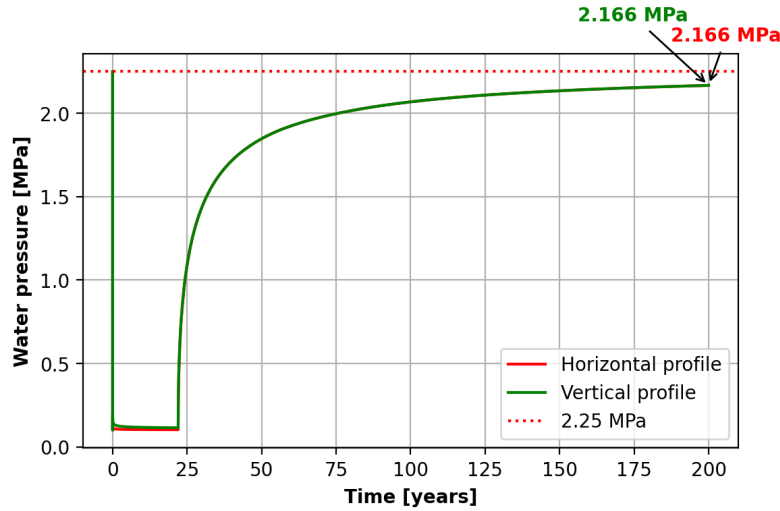


Figure 5.22: Evolution of water pressure over the three phases.

Figure 5.23 illustrates that, after excavation, the stress keeps increasing during drainage and stabilization. Especially, during the phase of drainage, that increase is non negligible as ≈ 0.5 MPa is added to the contact pressure on both profiles. Regarding the displacement it can be seen that the backward displacement, which until then had been rather limited, rises to ≈ 10 mm. During the phase of stabilization, despite its duration of 200 years, the contact pressure only grows by ≈ 0.2 MPa.

Figure 5.24 confirms that the main displacements happen during the excavation when the gap between the lining and the rock need to close before contact. During drainage and stabilization, the additional displacement does not exceed 1 mm for the vertical profile and it is even reduced for the horizontal profile due to the ovalization of the liner.

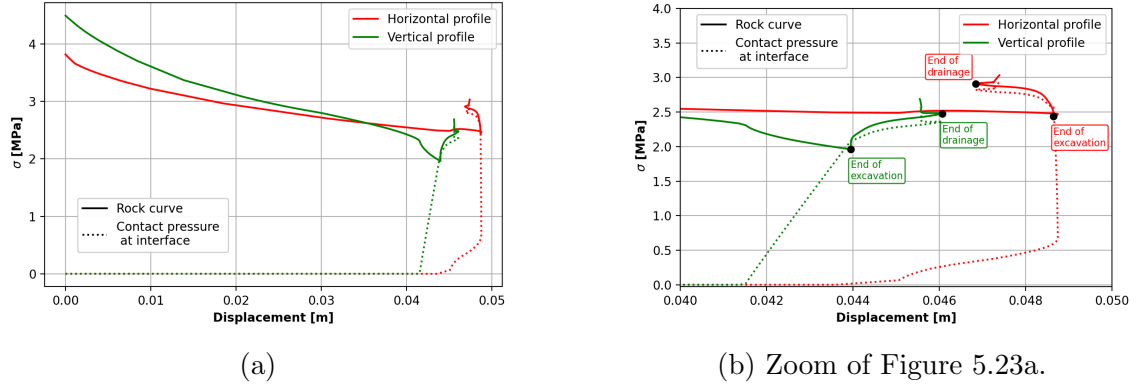


Figure 5.23: Rock and lining curves for the horizontal and vertical profiles for a gap of 4.5 cm.

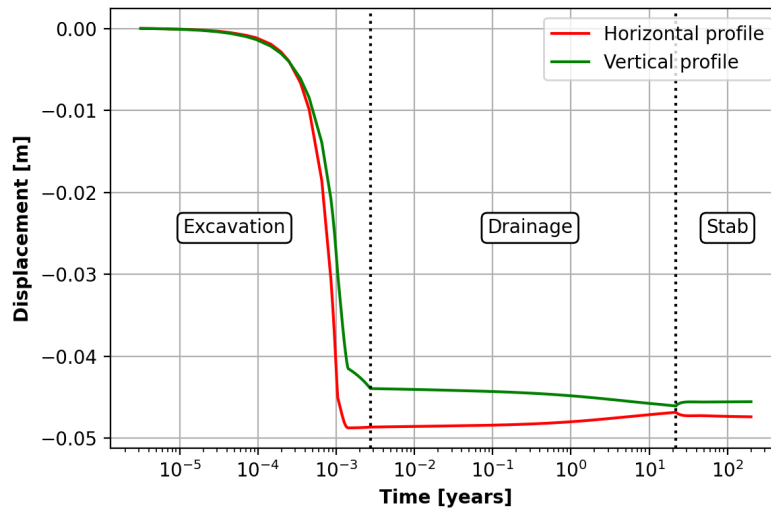
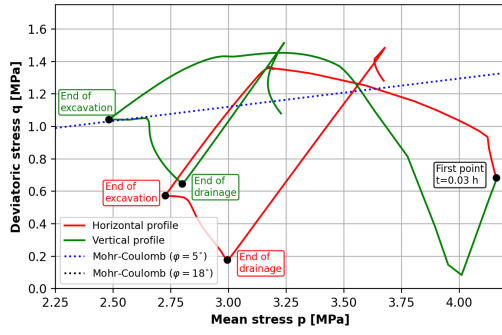


Figure 5.24: Displacement of the first points of the horizontal and vertical profiles over time, for a gap of 4.5 cm.

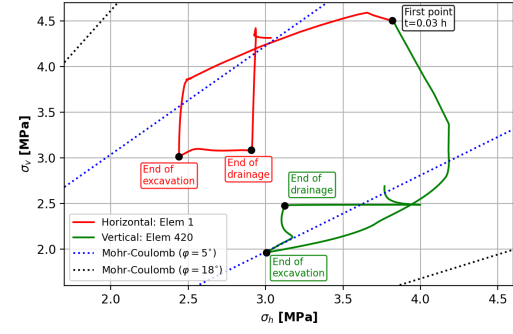
Finally, Figure 5.26 shows the evolution of the contact pressure over the 200 years that have been simulated. At first, before the contact, it logically stays at zero before reaching 2.572 MPa after 24 hours. 200 years later, the contact pressure has only gone up by 14%, at 2.935 MPa despite the much longer time elapsed.

As mentioned previously, for a gap of 4.5 cm, the increase of contact pressure after 20 or 200 years stay quite limited. Figure 5.27 illustrates that the change is very dependent on the size of the gap. When looking at the difference between the end of excavation and the end of stabilization, the rise in contact pressure keeps getting greater with the size of the gap, reaching almost 20% in the horizontal profile and even 40% in the vertical profile. This correlation between the increase in contact pressure only holds when comparing the end of the excavation and stabilization phases as the biggest increase between the end of excavation and drainage phases is reached for an intermediate gap, 2.4 cm, before decreasing for greater gaps.

In addition to the two profiles on which the contact pressure has been shown until now, Figure 5.28 illustrates the contact pressure for different angles θ (with θ



(a) Mean and deviatoric stress plane.



(b) Horizontal and vertical stresses plane.

Figure 5.25: Stress path for the first point of the vertical and horizontal profiles.

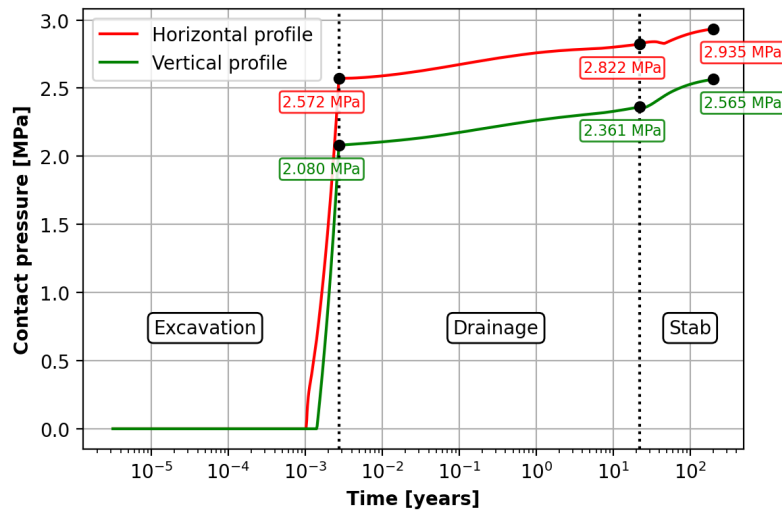


Figure 5.26: Evolution of the contact pressure over time in the horizontal and vertical profiles.

defined in Figure 5.12a) along the lining and for different times during excavation. Figure 5.28 confirms that the contact initiate horizontally and gradually propagates towards the top of the lining. Whatever the time considered, the pressure is always stabilizing at greater values for lower angles while it is stabilizing at lower values for greater angles, with a transition between the two where the pressure is no more stable and is decreasing.

Figure 5.29 displays the same points during the drainage and excavation phases. The trend is similar, with the maximum contact pressure still located at an angle $\theta = 0^\circ$ and increasing with time, although the increase experienced during the last hundred years stays quite limited (from 2.9 MPa to 2.94 MPa).

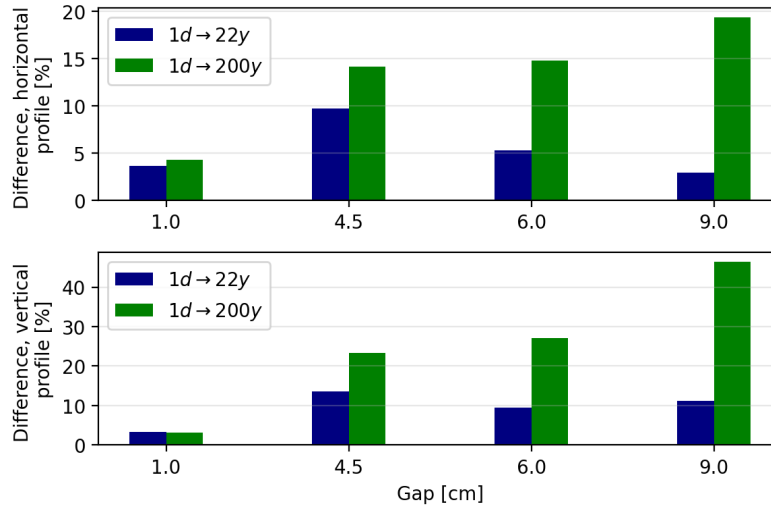


Figure 5.27: Change in contact pressure between the phases depending on the gap.

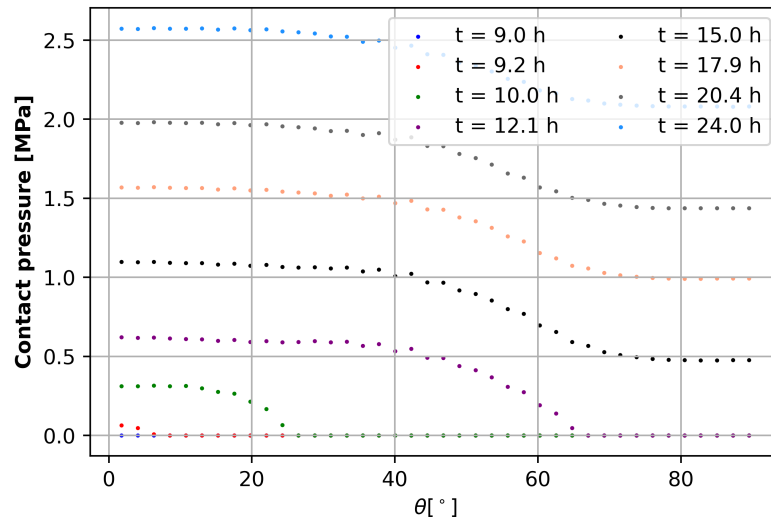


Figure 5.28: Contact pressure along the lining for different times during the excavation.

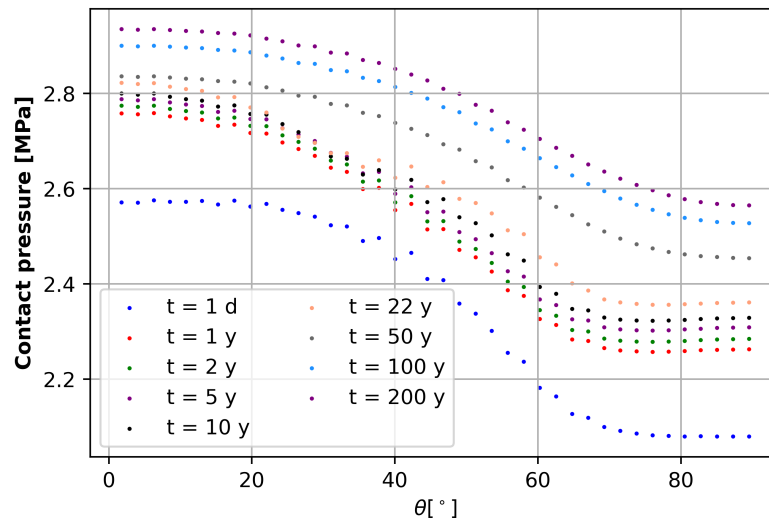


Figure 5.29: Contact pressure along the lining for different times.

5.1.3 Stresses in the lining

Because of the contact with Boom clay, stresses appear in the lining. The stresses returned by the finite element code are expressed in the horizontal and vertical axes. As the lining is circular, it is more interesting to consider the radial and orthoradial stresses σ_r and σ_θ . At a point located at coordinates x and y and subjected to stresses σ_x , σ_y and σ_{xy} , the radial and orthoradial stresses are given by:

$$\alpha = \arctan\left(\frac{y}{x}\right) \quad (5.1)$$

$$\beta = \alpha + \frac{\pi}{2} \quad (5.2)$$

$$\sigma_r = \sigma_x \cos^2(\alpha) + \sigma_y \sin^2(\alpha) + \sigma_{xy} \sin(2\alpha) \quad (5.3)$$

$$\sigma_\theta = \sigma_x \cos^2(\beta) + \sigma_y \sin^2(\beta) + \sigma_{xy} \sin(2\beta) \quad (5.4)$$

In the following, positive and negative values of the stresses respectively correspond to tension and compression.

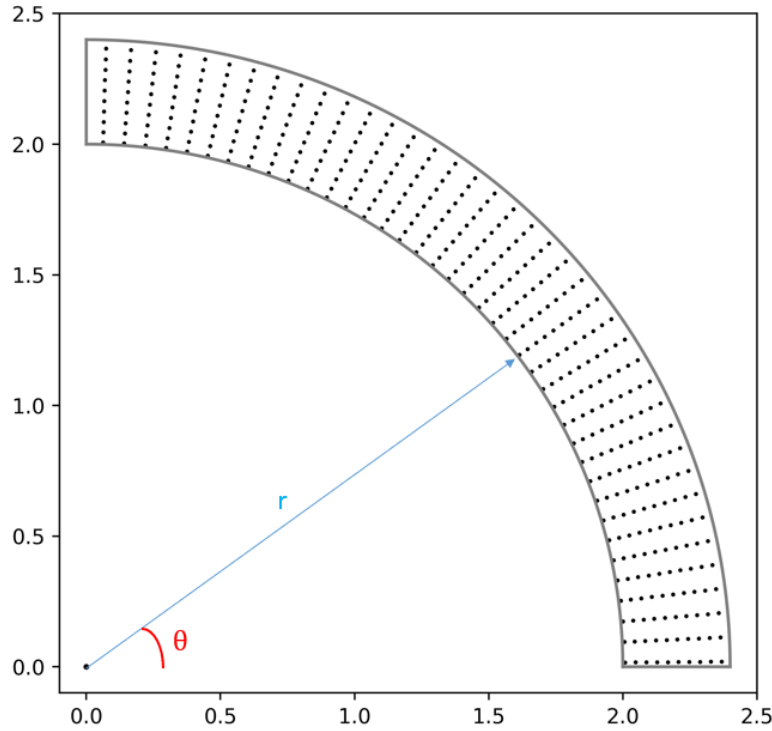


Figure 5.30: Layout of the available data points for stresses and displacements.

Figure 5.30 displays the points that can be identified according to their polar coordinates with angle θ and distance r . The configuration of this section is the reference one, with a gap of 4.5 cm, an anisotropy coefficient of 0.7 and initial water pressures of 2.25 MPa. The value of the orthoradial stress at the points located on the outside and inside of the lining are respectively illustrated in Figures 5.31a and 5.31b. It can be seen that, as could be expected, there are no stresses in the lining up to $t = 7.9$ hours (blue points) as the contact did not happen yet. When comparing the intrados and the extrados of the lining, the stresses seem to be simply inverted.

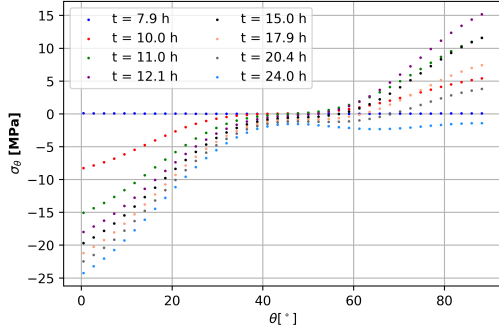
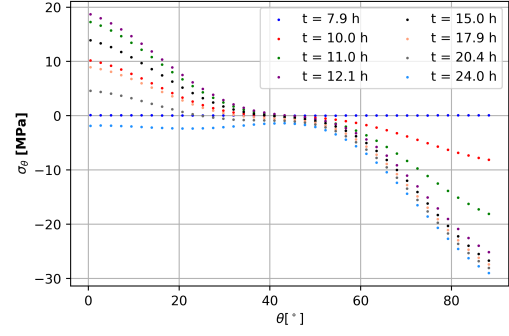
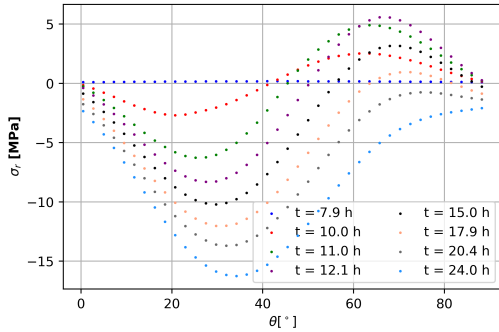
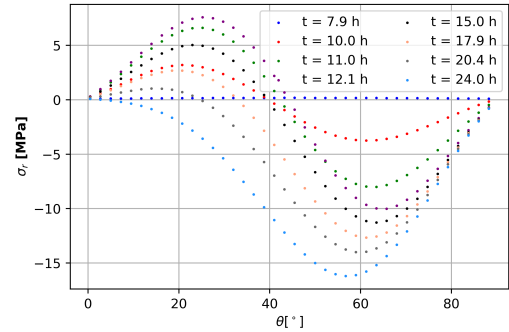
(a) $r = 2.37 \text{ m}$.(b) $r = 2 \text{ m}$.Figure 5.31: σ_θ depending on the angle θ for different times during the excavation.

Figure 5.31a (corresponding to the extrados of the lining) illustrates that the bottom of the lining ($\theta = 0^\circ$) always stays in compression during the excavation while the top ($\theta = 90^\circ$) undergoes tension at first before switching to compression. From the first contact, which happens just before $t = 9$ hours, the orthoradial stress keeps increasing until the end of the excavation at the bottom of the lining, reaching about $\sigma_\theta = -25 \text{ MPa}$. At the top of the lining, the stress reaches its maximum in tension at approximately $t = 12$ hours and $\sigma_\theta = 15 \text{ MPa}$ before decreasing and switching to compression. Around the middle of the lining ($\theta = 45^\circ$), the concrete is mostly in compression but stays at quite low values.

The situation is inverted when considering the intrados of the lining (Figure 5.31b). All that happens at $\theta = 0^\circ$ for $r = 2.37 \text{ m}$ happens at $\theta = 90^\circ$ for $r = 2 \text{ m}$ and vice versa.

(a) $r = 2.37 \text{ m}$.(b) $r = 2 \text{ m}$.Figure 5.32: σ_r depending on the angle θ for different times during the excavation.

Unlike the orthoradial stress, Figure 5.32 shows that the radial stress is practically null at the extremities of the lining and maximum around $\theta = 35^\circ$ and $\theta = 65^\circ$. Similarly to the orthoradial stress, in the extrados of the lining, the radial stress is always in compression and keeps increasing until the end of the excavation at the bottom. At the top, once again, the maximum in tension is reached at approximately $t = 12$ hours before a decrease in the stress brings the concrete in compression. As for the orthoradial stress, the values of the stresses are inverted between the intrados and extrados of the lining.

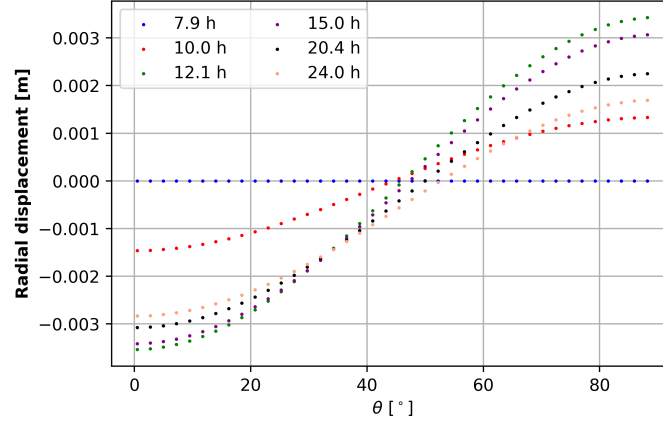
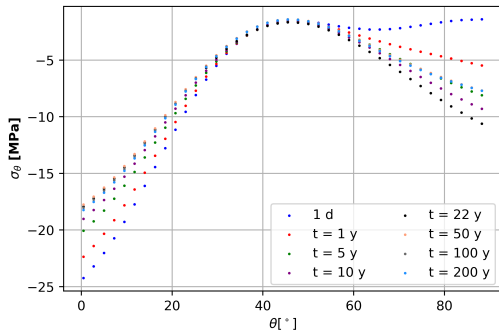
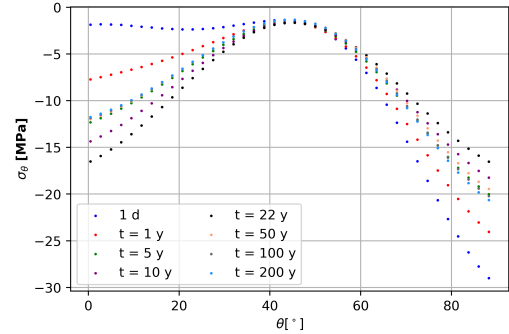


Figure 5.33: Displacements of the lining depending on the angle θ for different times during the excavation (< 0 if towards the centre of excavation, > 0 if away), $r = 2.37$ m.

The displacement of the lining is displayed in Figure 5.33. The difference in the direction of the displacement depending on the angle θ is due to the ovalization of the lining that has been explained previously. It can be seen that the displacement is the highest at $t = 12$ hours, before it decreases until the end of the excavation.



(a) $r = 2.37$ m.



(b) $r = 2$ m.

Figure 5.34: σ_θ depending on the angle θ for different times.

Figure 5.34 shows the evolution of the orthoradial stress in the lining during the drainage and excavation phases. In the extrados of the lining, for angles close to $\theta = 0^\circ$, compression diminishes, approximately going from -24 MPa to 18 MPa. At the top lining, compression reaches its maximum at the end of the drainage phase, after 22 years, before decreasing to ≈ 7 MPa at the end of the stabilization phase. In the intrados of the lining, everything is inverted.

During the excavation, there were two local maxima (around 35° and 65°) for the radial stress in the lining. Figure 5.35 illustrates that they tend to merge into only one as the drainage and stabilization phases progress. That new maximum is lower than the two previous ones as it is approximately equal to $\sigma_r = -13$ MPa in the extrados and $\sigma_r = -12.5$ MPa in the intrados. It can be seen in Figure 5.35b that rather limited tension appears in the intrados at the extremities of the lining for times following $t = 50$ years.

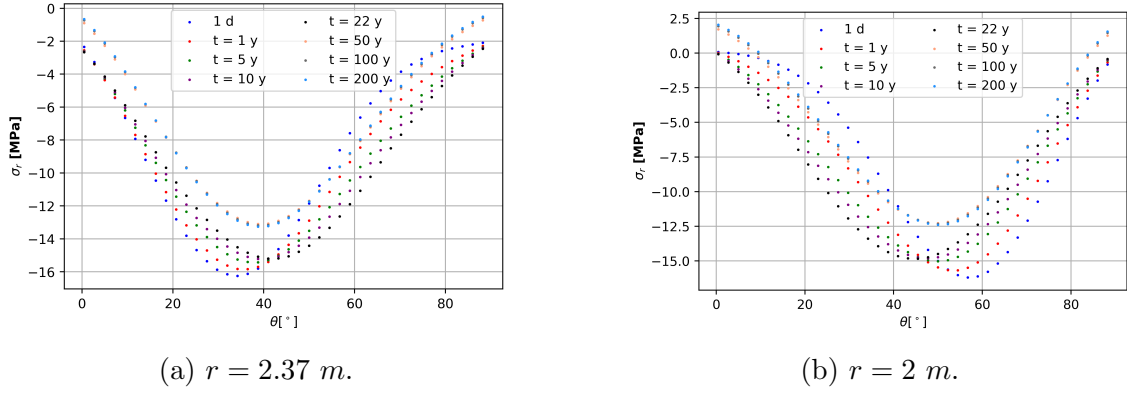


Figure 5.35: σ_r depending on the angle θ for different times during the excavation.

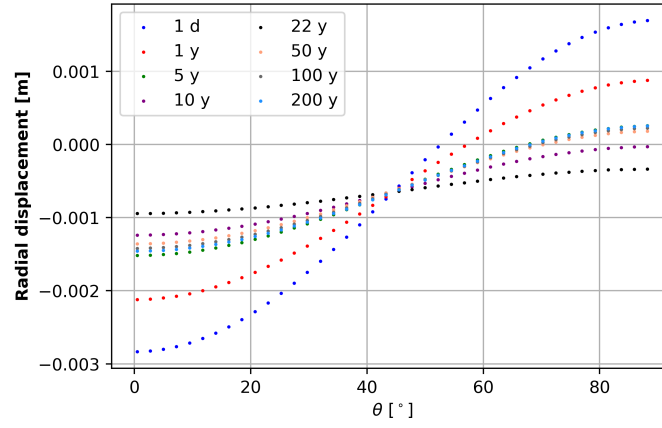


Figure 5.36: Displacements of the lining depending on the angle θ for different times (< 0 if towards the centre of excavation, > 0 if away), $r = 2.37 \text{ m}$.

In the same way that the stresses diminish after the end of the excavation, Figure 5.36 displays that the displacement also attenuates, even if the ovalization of the liner can still be noticed.

5.2 Optimisation-oriented parametric study

In this section, various parameters of the reference configuration are modified to study their effect on the lining and Boom clay. Parameters tested here include the size of the gap, the initial stress, the coefficient of anisotropy, the deconfinement rate and speed and the width of the model.

These parameters can be arranged into three categories:

- Influence of the method of excavation (variation of the size of the gap and deconfinement rate and speed).
- Influence of the geological conditions (variation of the coefficient of anisotropy and initial stress).

- Influence of the design choice for the storage of nuclear waste (variation of the model width).

5.2.1 Influence of the method of excavation

5.2.1.1 Distance to the excavation front/size of the gap

While the main objective of previous sections (where hydro-mechanical behaviour and anisotropy were progressively added) was to get a model closer to reality, there are still other parameters that can not be considered with a 2D transversal model such as distance to the excavation front at which the lining is placed. However, it is possible to study various sizes of the gap, which has a similar effect on the curves. As displayed in Figure 5.37, increasing the gap between the lining and Boom clay only translates the characteristic curve of the lining and leaves the rock curve unchanged.

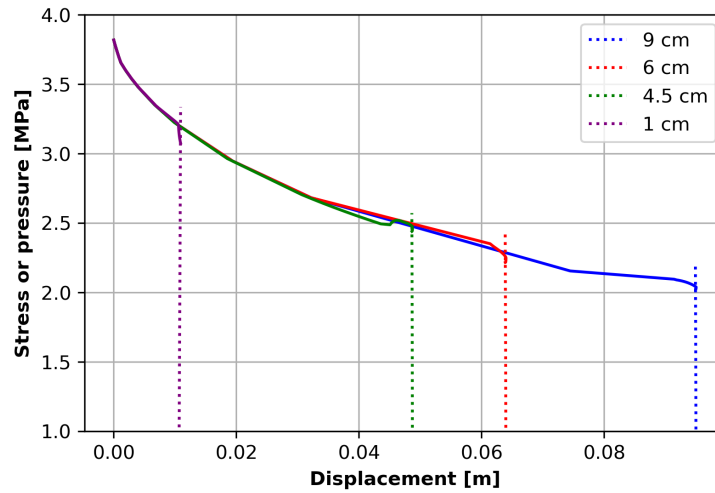


Figure 5.37: Characteristic curves of the rock and lining for different sizes of gaps, hydro-mechanical behaviour with anisotropy.

Figure 5.37 illustrates that higher values of the gap (or, similarly, greater distances of placement of the lining from the excavation front) lead to higher displacement of Boom clay before the contact happens and thus cause lower contact pressures when the equilibrium is reached.

Figure 5.38 shows a comparison of the contact pressure at the end of excavation for the different configurations of the reference configuration that have been tested (with and without anisotropy, with and without water pressures). All contact pressures are given in total stresses, taking water pressure into account. It can be seen that the introduction of anisotropy causes the contact pressure to increase, even if by a small amount. That difference between the isotropic and anisotropic configurations is almost non existent when there are no water pressures. The source of the big increase in pressure between the simplest configuration ($K_0 = 1$ and $p_{w,0} = 0 \text{ MPa}$) and the more complex one ($K_0 = 0.7$ and $p_{w,0} = 2.25 \text{ MPa}$) is the addition of water pressure. Table 5.1 shows the value of the parameters used to interpolate a polynomial of third degree $a x^3 + b x^2 + c x + d$

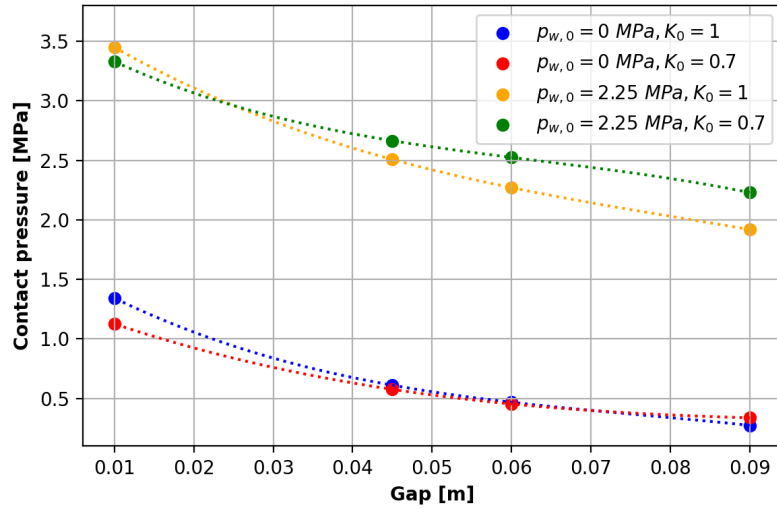


Figure 5.38: Comparison of the contact pressure at the end of excavation for the different configurations

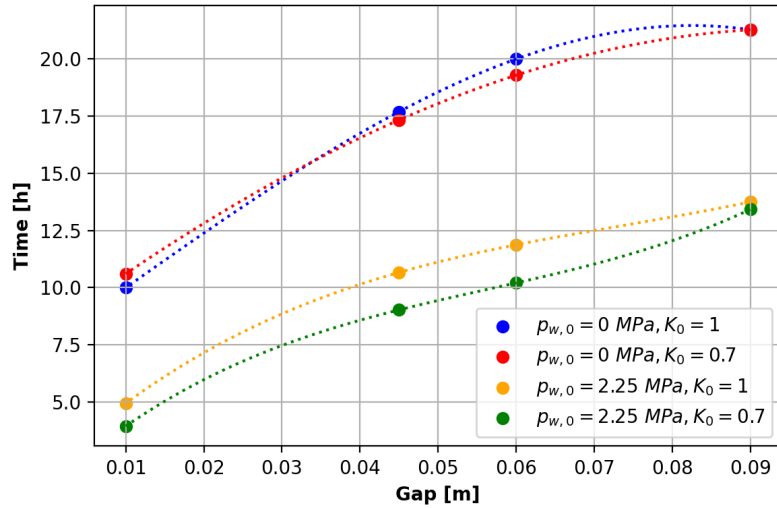


Figure 5.39: Comparison of the time of first contact for the different configurations

As displayed in Figure 5.39, the time of first contact between the rock and lining behaves the opposite way: for smaller gaps, the distance that has to be travelled by the rock is lower, which means that, for all configurations, the contact happens later for bigger gaps. When the hydraulic behaviour is added, the first contact time is much lower as it happens between approximately 5 and 9 hours earlier.

		a	b	c	d
$p_{w,0} = 0 \text{ MPa}$	$K_0 = 1$	-1982.736	454.898	-40.802	1.707
	$K_0 = 0.7$	-680.975	228.639	-26.523	1.369
$p_{w,0} = 2.25 \text{ MPa}$	$K_0 = 1$	-1651.608	412.486	-45.328	3.862
	$K_0 = 0.7$	-2542.262	484.911	-39.104	3.674

Table 5.1: Third degree polynomials used for the interpolation of the contact pressure of the different configurations

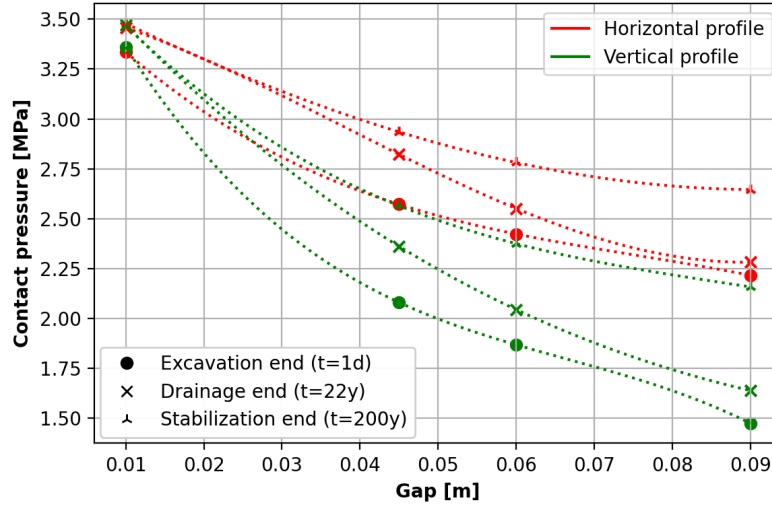


Figure 5.40: Contact pressure at the end of each phase for different gaps with $p_{w,0} = 2.25 \text{ MPa}$ and $K_0 = 0.7$.

		a	b	c	d
Horizontal profile	Excavation	-2137.113	483.853	-42.936	3.719
	Drainage	2521.213	-288.985	-8.745	3.571
	Stabilization	263.070	75.687	-20.376	3.675
Vertical profile	Excavation	-5313.703	1059.566	-81.196	4.073
	Drainage	-533.818	272.96	-45.367	3.899
	Stabilization	-1752.250	463.161	-46.740	3.890

Table 5.2: Third degree polynomials used for the interpolation of the contact pressure at the three phases.

Finally, Figure 5.40 shows the contact pressure obtained at the end of all phases for the hydro-mechanical and anisotropic configuration ($p_{w,0} = 2.25 \text{ MPa}$ and $K_0 = 0.7$). As already explained in Figure 5.27, smaller gaps induce less variation between the different phases while there is a difference of almost 0.5 MPa between the end of the excavation and stabilization when the gap has a size of 9 cm. All parameters used in the fitting of the curves are shown in Table 5.2.

It can be seen in Figure 5.41 that higher orthoradial stresses are reached in the concrete lining for wider gaps. The radial stress is behaving in the opposite way, with higher stresses for smaller gaps.

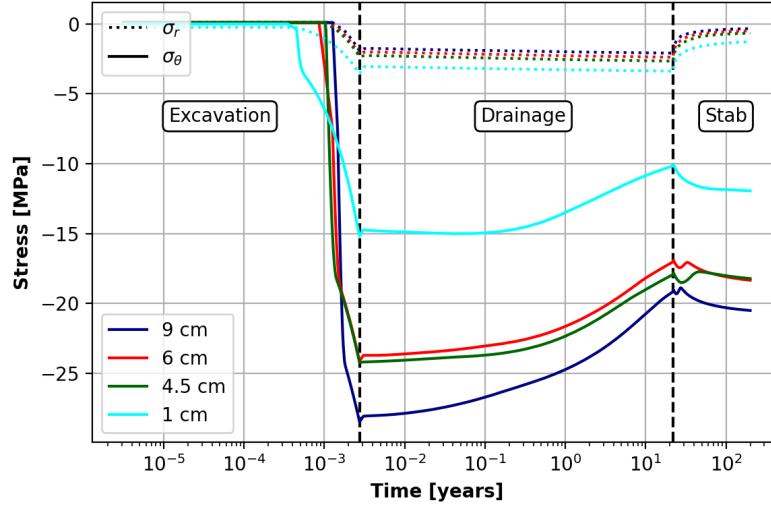


Figure 5.41: Radial and orthoradial stresses depending on the size of the gap, for a point located at the bottom right of the lining.

5.2.1.2 Variation of the deconfinement speed and profile

As illustrated in Figure 5.42a, the reference deconfinement profile is linear. In this section, a trilinear (Figure 5.42b) and a curved (Figure 5.42c) profile are tested. In addition, for both new profiles, it is considered the water pressure decreases suddenly as soon as the excavation has made enough progress and the front has reached the studied section.

Figure 5.43 displays that, while the profile does not change at the end of the excavation, the red and green curves are the most affected (respectively corresponding to five and ten hours after then start of the excavation) by the change in the deconfinement rate. Indeed, the trilinear profile affects the five-hour curves by keeping them closer to the initial state near the gallery (meaning that σ_r is greater and σ_θ lower). The curved profile has an effect on both curves related to five and ten hours with, again, values of the stresses that are closer to the initial state near the tunnel.

As can be seen in Figure 5.44, the consequences of the change in the deconfinement profile are the same in the vertical profile as in the horizontal profile. Again, between the reference and trilinear profile, the five-hour curves experience lower values of the orthoradial stress but higher values of the radial stress at proximity of the gallery. Between the trilinear and curved profiles, the ten-hour curves are also closer to the initial state.

The conclusion is exactly the same regarding water pressures in the horizontal (Figure 5.45) and vertical (Figure 5.46) profiles: the trilinear profile lowers the value of the pressure near the gallery after five hours, the curved profile lowers the value of the pressure near the gallery, and the pressures corresponding to greater times are left unchanged no matter the profile.

Figure 5.47 shows the evolution of the contact pressure for different points in the lining over time and for the three profiles. A first difference is the shapes of

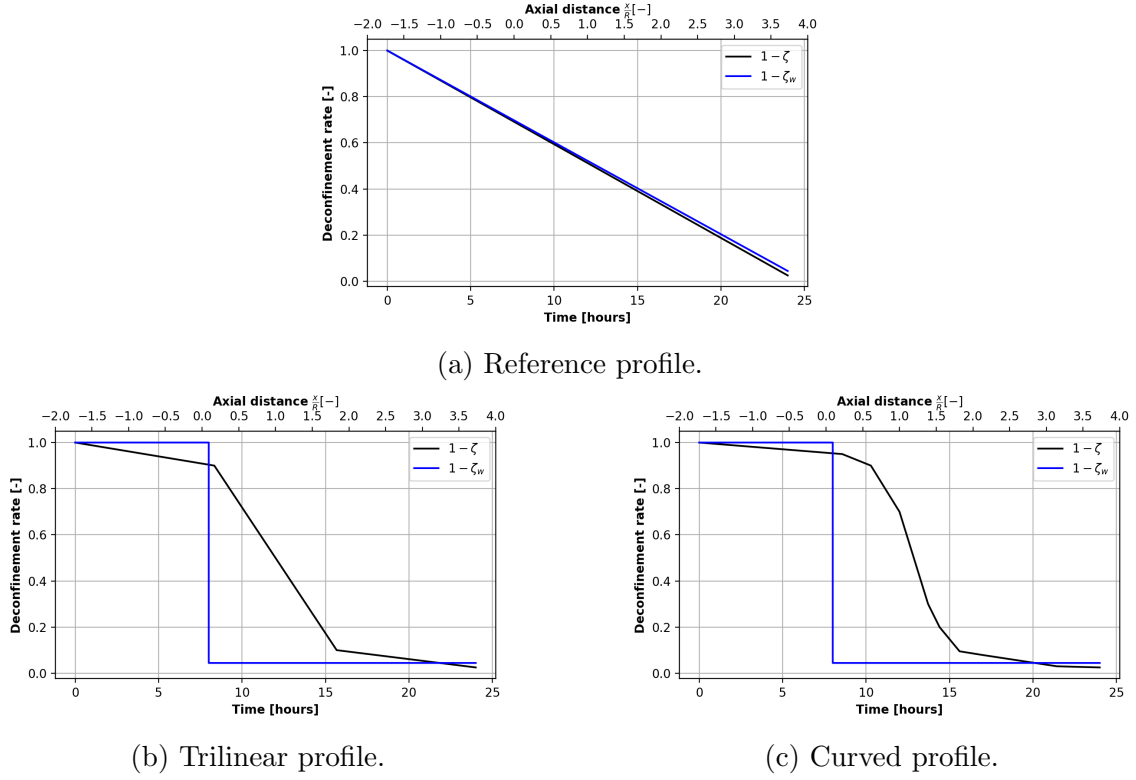
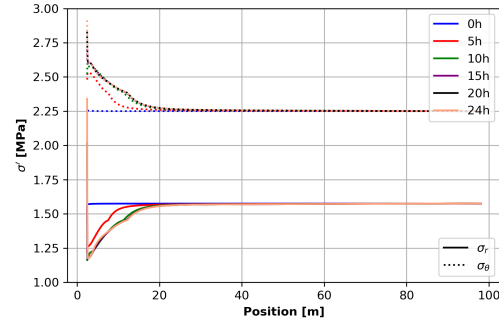
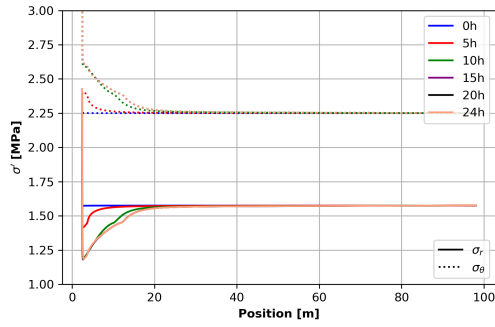


Figure 5.42: Deconfinement profiles.

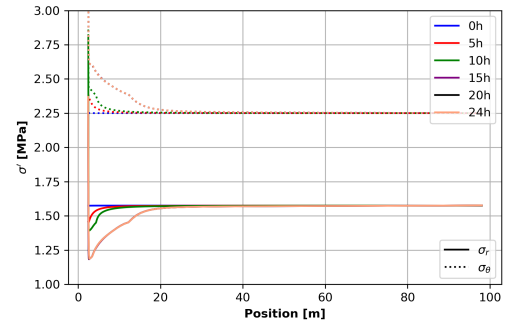
the curves that seem to follow the shape of the deconfinement profile. While the development of the pressure is rather linear for the linear deconfinement rate, it is quite trilinear or curved for the two other profiles. Another change is the point in the lining at which the contact pressure is at its maximum. The value of that maximum does not really change but, in the linear deconfinement, the maximum is found at $\theta = 2^\circ$ (corresponding to the horizontal profile) while, as the profile gets more curved, the maximum is found at an angle $\theta = 27^\circ$. This means that modifying the deconfinement profile moves the point at which the maximum pressure is located, without necessarily affecting its value by much.



(a) Reference.

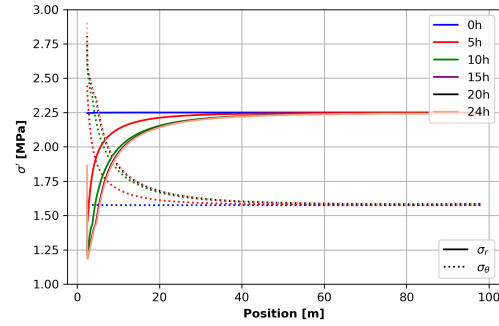


(b) Trilinear.

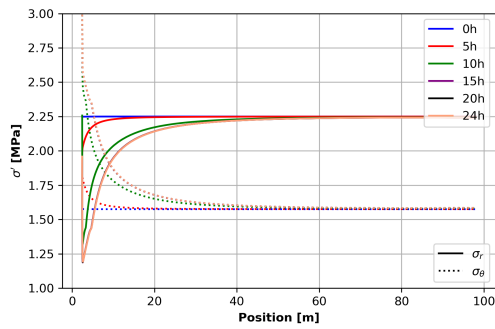


(c) Curved.

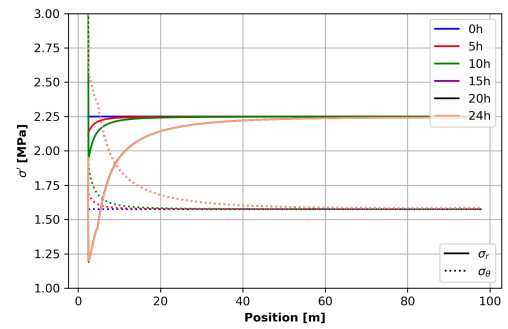
Figure 5.43: Radial and orthoradial effective stresses along the horizontal profile for the three deconfinement rates.



(a) Reference.

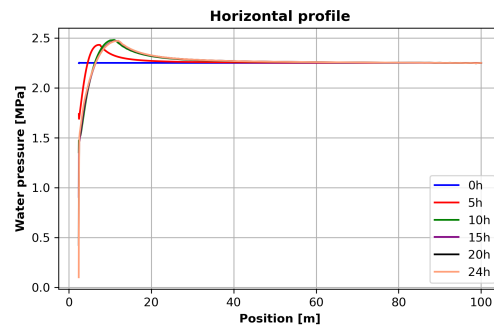


(b) Trilinear.

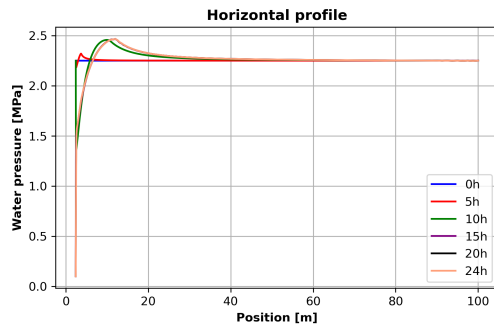


(c) Curved.

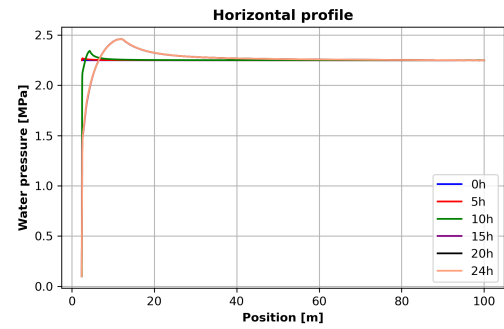
Figure 5.44: Radial and orthoradial effective stresses along the vertical profile for the three deconfinement rates.



(a) Reference.

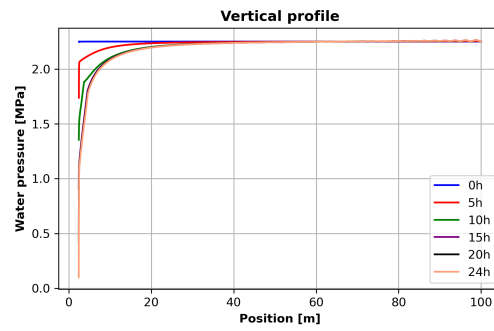


(b) Trilinear.

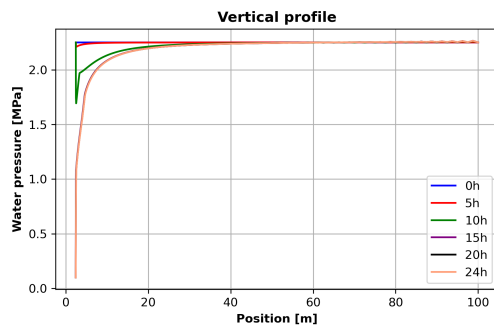


(c) Curved.

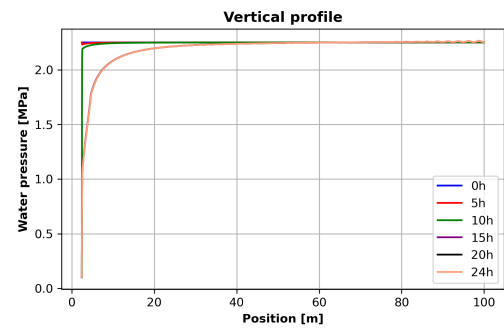
Figure 5.45: Water pressure along the horizontal profile.



(a) Reference.

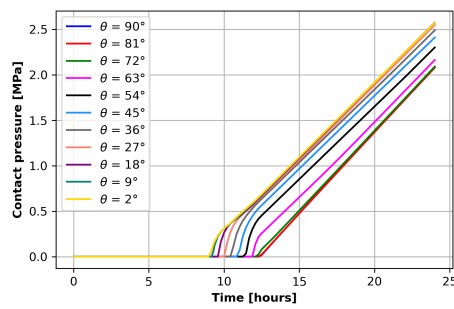


(b) Trilinear.

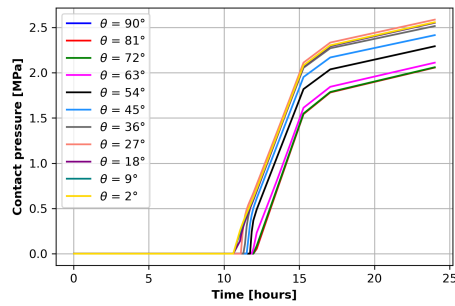


(c) Curved.

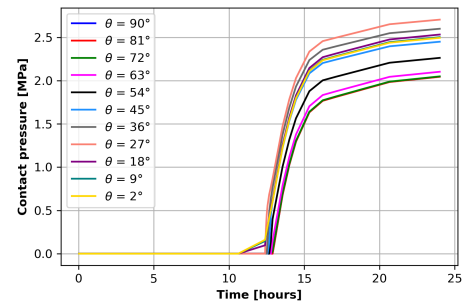
Figure 5.46: Water pressure along the vertical profile.



(a) Reference.



(b) Trilinear.



(c) Curved.

Figure 5.47: Contact pressure over time at different angles in the lining.

5.2.2 Influence of the geological conditions

5.2.2.1 Variation of the initial stress

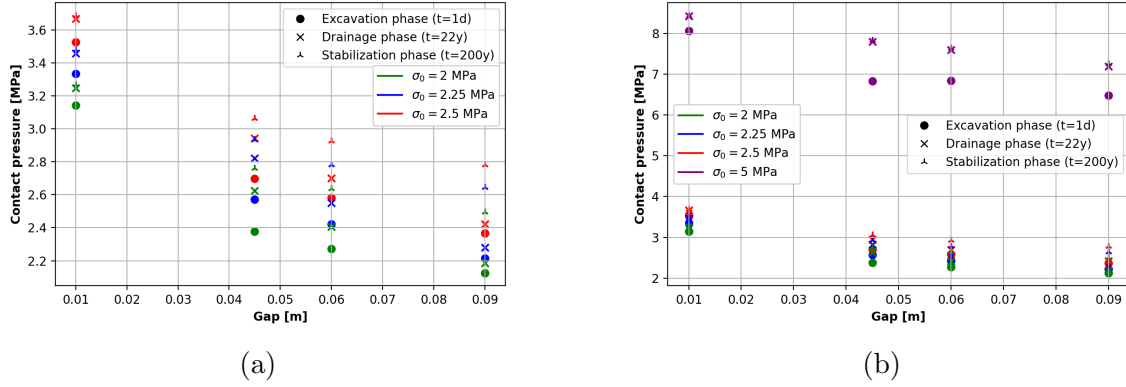


Figure 5.48: Contact pressure for different depths.

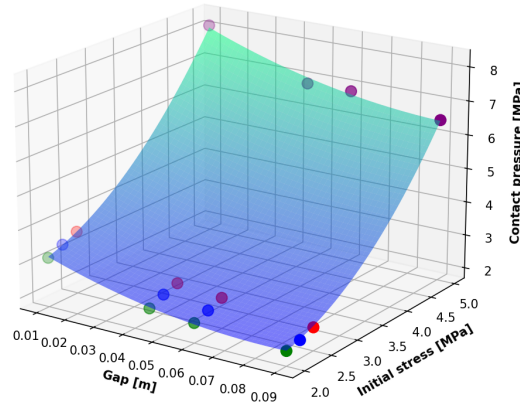


Figure 5.49: Contact pressure as a function of the gap and the initial stress.

As geological repositories are generally located at very high depths, it is interesting to study the impact of a variation in the depth at which the facility is built by modifying the initial value of the horizontal and vertical stresses. In the reference configuration, the depth is considered to be 225 m and is associated to an initial stress $\sigma_0 = 2.25$ MPa. From now on, all cases will consider anisotropy as well as the hydro-mechanical behaviour of Boom clay as this model better represents reality. Figure 5.48a displays that the shape of the graph stays very similar for all depths that are close to $\sigma_0 = 2.25$ MPa. When a fourth depth is introduced in Figure 5.48b, the general trend remains that the larger the gap, the higher the contact pressure. The only two points that are exceptions to that rule are the ones for gaps of 4.5 and 6 centimetres which have similar pressures at the end of excavation. This can be explained by the fact that there is only a difference of 1.5 cm, which is very close, between them while the initial stress and water pressure have been doubled. It is possible to deduce a relation between the initial stress σ_0 , the size of the gap g and the contact pressure p at the end of each phases from these points. The general formulation of that relation is given in Equation 5.5 and all parameters are given in Table 5.3. That function is shown in a 3D view in Figure 5.49.

$$p = a \times g^2 + b \times \sigma_0^2 + c \times g + d \times \sigma_0 + e \times g \times \sigma_0 + f \quad (5.5)$$

	a	b	c	d	e	f	r^2
Excavation	193.40044	0.35709	-28.23505	-0.87005	-2.062232	3.78776	0.9990
Drainage	92.03929	0.42853	-23.02054	-1.27231	-0.32136	4.35006	0.9997
Stabilization	102.08245	0.40523	-17.10771	-1.09186	-1.56780	4.04859	0.9998

Table 5.3: Value of the parameters of Equation 5.5 for the different phases.

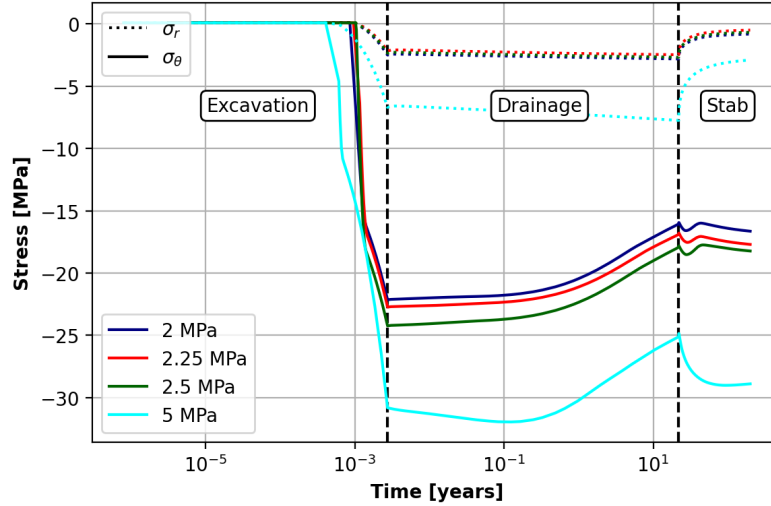


Figure 5.50: Radial and orthoradial stresses depending on the depth.

Figure 5.50 shows that increasing the depth has logically the effect of increasing the stresses inside the lining, reaching up to 32 MPa in compression when the initial stress is equal to 5 MPa.

5.2.2.2 Variation of anisotropy

At great depths, the value of the coefficient of anisotropy $K_0 = \frac{\sigma_h}{\sigma_v}$ is not well established. It is therefore possible to compare different values ranging from $K_0 = 0.5$ (high anisotropy) to $K_0 = 1$ (isotropic) to quantify their impact on the contact pressure. Previously, when K_0 was equal to 0.7, the contact pressure was higher in the horizontal profile.

Figure 5.51 shows the contact pressures obtained for different gaps and values of K_0 , at the end of the excavation and in a simple configuration where $p_{w,0} = 0$ MPa. It can be seen that, for coefficients between 0.7 and 1, the contact pressure in the vertical plane is still lower than in the horizontal plane (except for $K_0 = 1$ where it is identical). However, when the anisotropy is high ($K_0 = 0.5$, which means that $\sigma_v = 2 \times \sigma_h$), the pressure in the vertical profile greatly increases and is between 2 and 10 times higher than the pressure in the horizontal profile depending on the gap. The contact pressures observed in the configuration where $K_0 = 0.5$ seem quite surprising as the values they reach are fairly high and the decreasing trend with the gap that can be observed for other coefficients disappear.

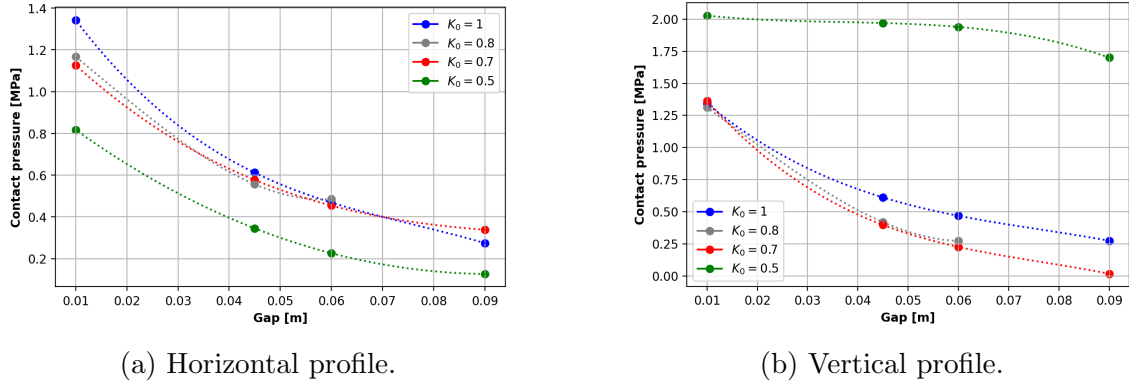


Figure 5.51: Evolution of the contact pressure for different gaps and anisotropy coefficients, without water pressures.

This behaviour can be explained by Figures 5.52a and 5.52b that display the evolution of the contact pressure over time and the stress path in the $\sigma_h - \sigma_v$ plane. The much higher final values of the contact pressure in the vertical profile are due to the contact happening much sooner than habitually, after barely more than 10 hours as can be seen in Figure 5.52a. In other configurations, it was observed that a faster entry into plasticity caused the first point of the horizontal profile to experience higher deformations, leading in turn the contact happening horizontally first and to the development of higher contact pressures. However, as displayed in Figure 5.52b, the coefficient of anisotropy is so low that the initial point is shifted to the left (as σ_h is lower) and starts in the plasticity zone. That plastic state is associated to higher deformations, which cause the points of the vertical profiles to displace much more rapidly and to the contact happening sooner and on the vertical profile.

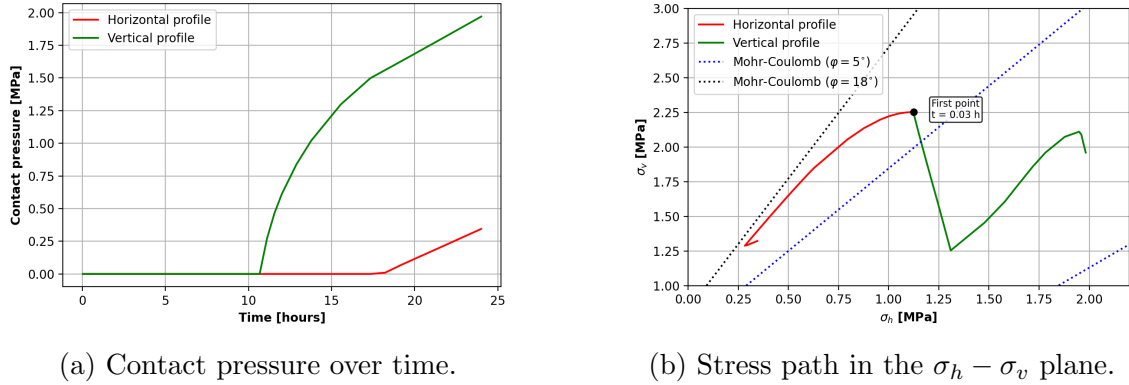


Figure 5.52: Contact pressure and stress path in the horizontal and vertical profiles.

Moreover, for coefficients of anisotropy closer to 1, the introduction of anisotropy does not seem to cause significant changes in the contact pressure values, either in the horizontal or vertical profiles. For the configurations in which K_0 is equal to 0.7, 0.8 or 1, the shape and magnitude of the curves are almost identical, with only a slight decrease in pressure as anisotropy increases. The main shift occurs for $K_0 = 0.5$, where, in the horizontal profile, the shape remains unchanged but the values are lower, while in the vertical profile, the decrease in pressure with increasing gaps is less pronounced, as the trend appears to become more linear.

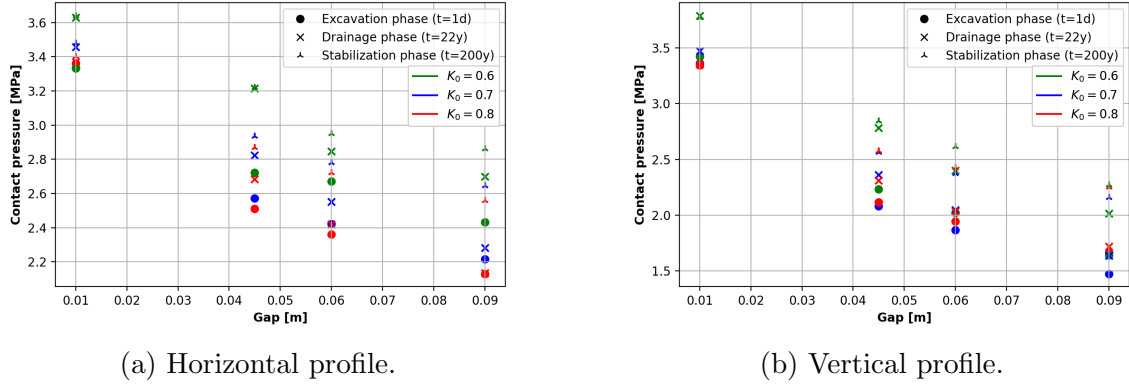


Figure 5.53: Evolution of the contact pressure for different gaps and anisotropy coefficients, configuration with hydro-mechanical behaviour.

Figure 5.53 illustrates the evolution of contact pressures at the end of the different phases for three coefficients of anisotropy in a configuration with hydro-mechanical behaviour. It can be seen that, the shape of the curves stays very similar for all coefficients (0.6, 0.7 and 0.8) and all phases. For all coefficients of anisotropy tested here, the greatest increase in pressure between the phases happens between excavation and drainage. For larger gaps, that difference between phases is more marked than for smaller gaps. Furthermore, for $K_0 = 0.6$, the shape of the curves is identical to other values of the coefficients of anisotropy. This means that the shift in behaviour highlighted in Figure 5.51 did not happen yet.

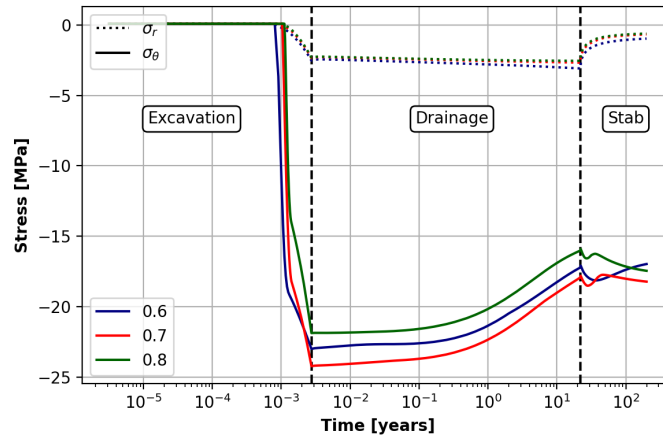


Figure 5.54: Radial and orthoradial stresses depending on the anisotropy, horizontal profile.

Figure 5.54 shows the evolution of the stresses in the lining at the bottom right point. The highest stress is reached at the end of the excavation for a coefficient $K_0 = 0.7$.

5.2.3 Variation of boundary conditions and model length

This section aims to study the presence of other galleries at proximity of the current one. Indeed, as mentioned in Chapter 4, other facilities can be located at just 50 m

(for category B waste) or 120 m (for category C waste). As a reminder, the presence of another gallery on the right side of the domain means water pressure is therefore not fixed anymore at the border while horizontal displacement is blocked.

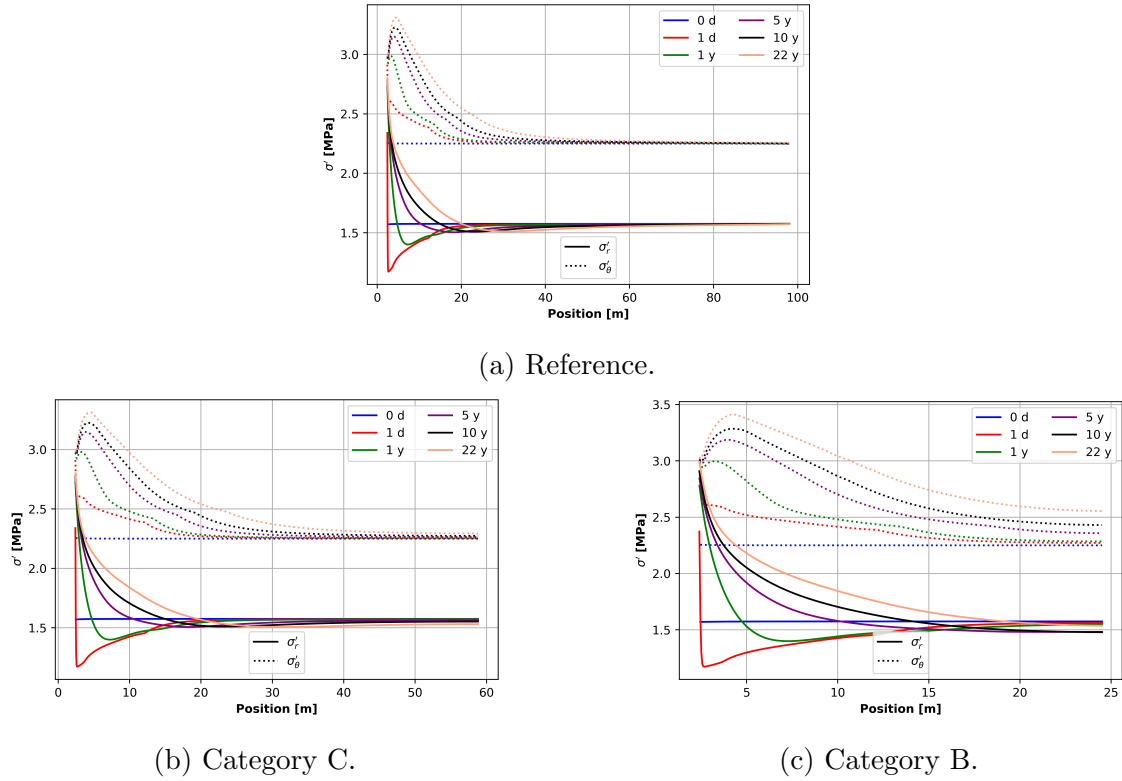


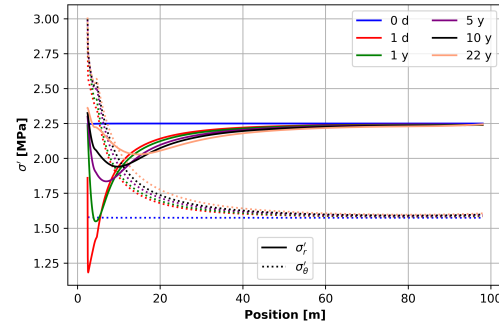
Figure 5.55: Radial and orthoradial stresses on the horizontal profile.

Figure 5.55 illustrates the evolution of the radial and orthoradial stresses over the horizontal profile. It can be seen that the first 25 meters of Figures 5.55a, 5.55b and 5.55c are nearly identical. This means that the construction of several galleries, even with little space between each other, does not disturb the stress state around each gallery and that there seems to be no interaction between them.

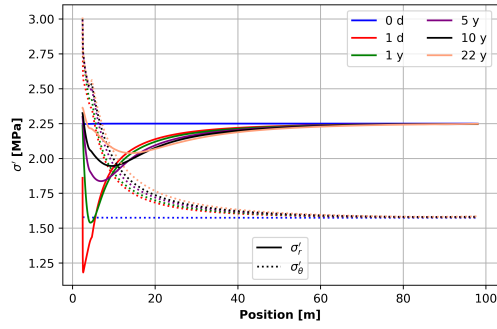
The evolution of the radial and orthoradial stresses along the vertical profile is displayed in Figure 5.56. As could be expected as the height of the domain does not change in that direction, the profiles are identical for all configurations.

Figures 5.57 and 5.58 which respectively show the evolution of water pressure over the horizontal and vertical profiles once again lead to the conclusion that the consideration of other nearby galleries does not affect the profiles.

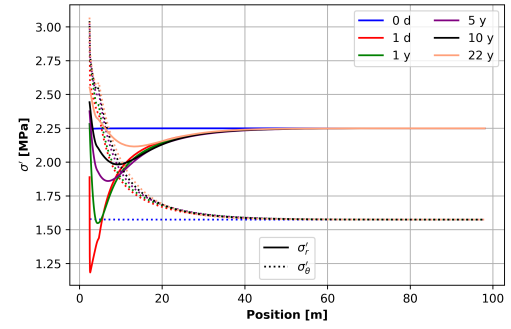
Figure 5.59 illustrates the evolution of the contact pressure on the lining over time for different angles θ . While the angles at which the highest pressures are found are left unchanged, there is a slight increase in the maximum contact pressure that is reached at the final time: the pressure goes from 2.822 MPa when the width is equal to 100 meters to 3.012 MPa when the width is equal to 25 meters.



(a) Reference.

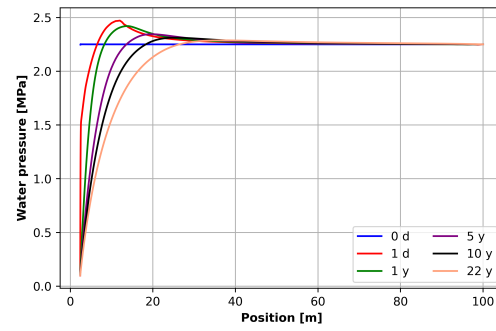


(b) Category C.

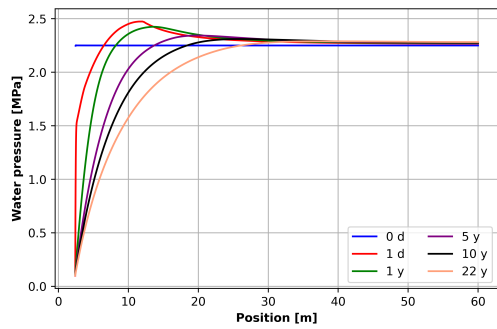


(c) Category B.

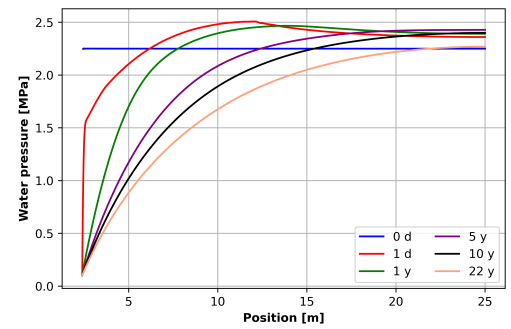
Figure 5.56: Radial and orthoradial stresses on the vertical profile.



(a) Reference.

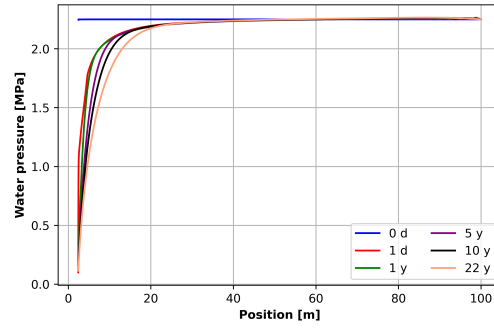


(b) Category C.

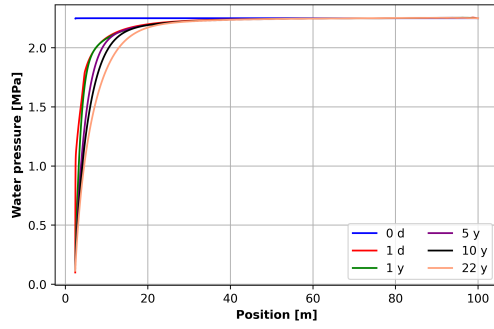


(c) Category B.

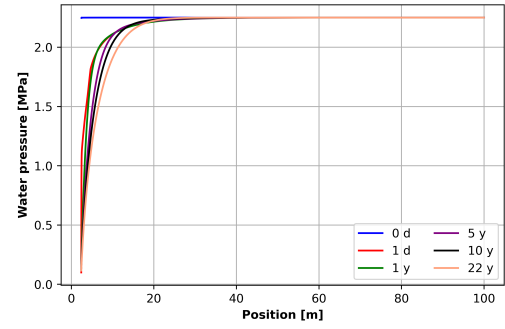
Figure 5.57: Water pressure on the horizontal profile.



(a) Reference.

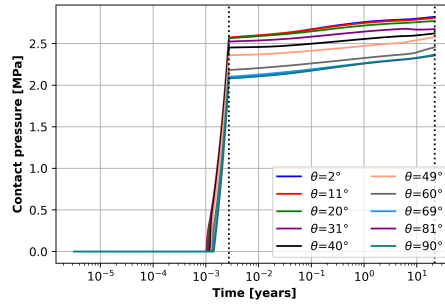


(b) Category C.

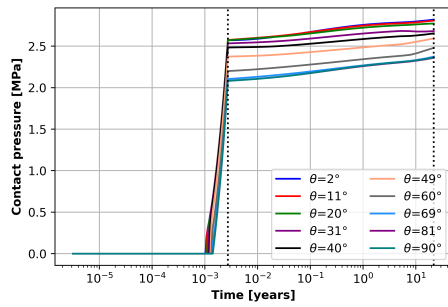


(c) Category B.

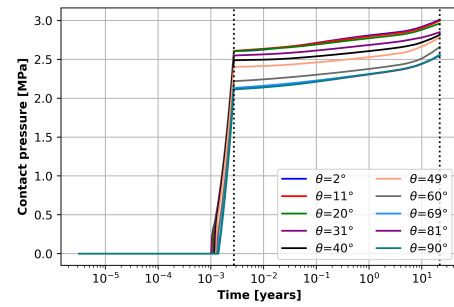
Figure 5.58: Water pressure on the vertical profile.



(a) Reference.



(b) Category C.



(c) Category B.

Figure 5.59: Evolution of contact pressure over time for different angles.

5.3 Effect of gravity

As mentioned in Chapter 4, half of the real size of the gallery is now represented in *LAGAMINE* in order to consider the effect of gravity.

5.3.1 Comparison without gravity

This section aims to study the changes brought by the new dimensions of the domain, without yet introducing gravity to the calculations.

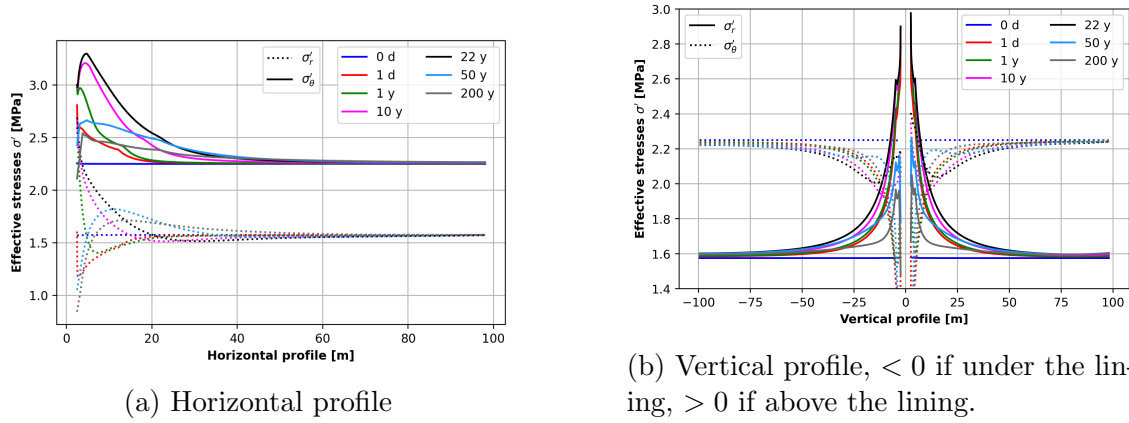


Figure 5.60: Radial and orthoradial stresses along the horizontal and vertical profiles.

The radial and orthoradial stress profiles are presented in Figure 5.60 for the horizontal and vertical profiles. The stresses in the horizontal profile (Figure 5.60a) are similar to the profiles that were obtained with the previous mesh. The maximum orthoradial stress is still reached at the end of the drainage phase before it decreases during the next 180 years. The rock further than approximately 60 meters remain undisturbed by the excavation, as the stress state stays identical to its initial value.

Figure 5.60b illustrates the stresses along the vertical profile. As the mesh has been extended under the gallery, there are now negative values in the profile that correspond to the part of the profile under the gallery. It can be seen that both vertical profiles are almost symmetrical, although the stresses in the profile above the gallery reach values that are slightly lower. Again, it can be observed that, once the drainage is over and the water pressures are left free to evolve, both the radial and orthoradial stresses slowly move back to their initial value.

Figure 5.61 displays the stress paths followed in the horizontal and vertical stresses $\sigma_h - \sigma_v$ plane (Figure 5.61a) and in the mean and deviatoric stresses $p - q$ plane (Figure 5.61b). Although the global shape of the paths is different compared to the previous configuration, especially in the plastic zone, the time at which the rock enters that plastic zone is identical to what has been found earlier. The horizontal profile reaches plasticity first at $t \approx 5.7 h$ and the vertical profile a little bit later at $t \approx 8.4 h$. Once again, plasticity is associated to higher deformations and thus to a contact that should happen first in a zone near the middle of the lining.

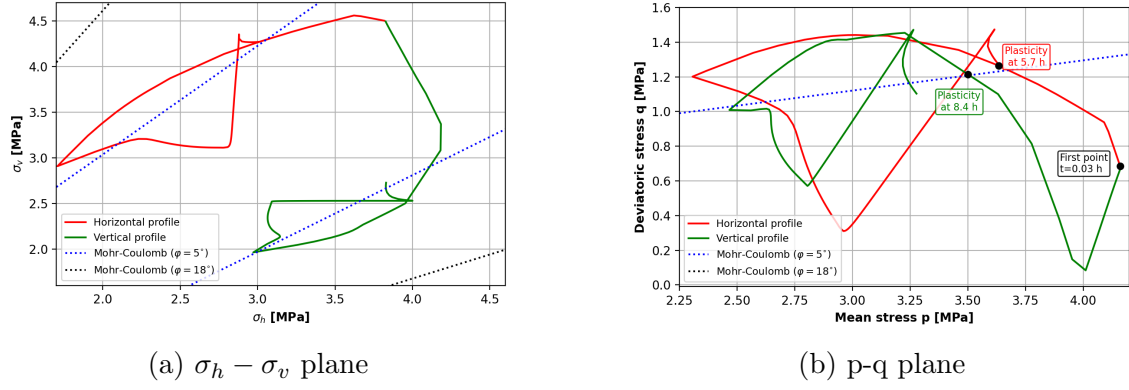


Figure 5.61: Stress paths.

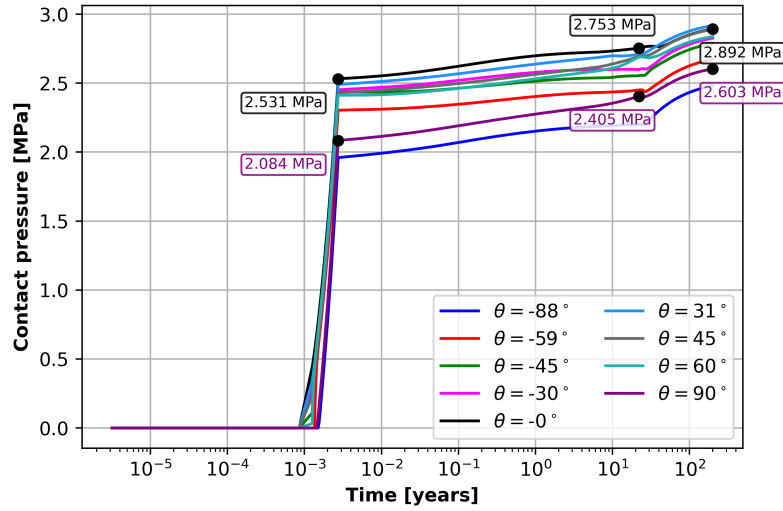


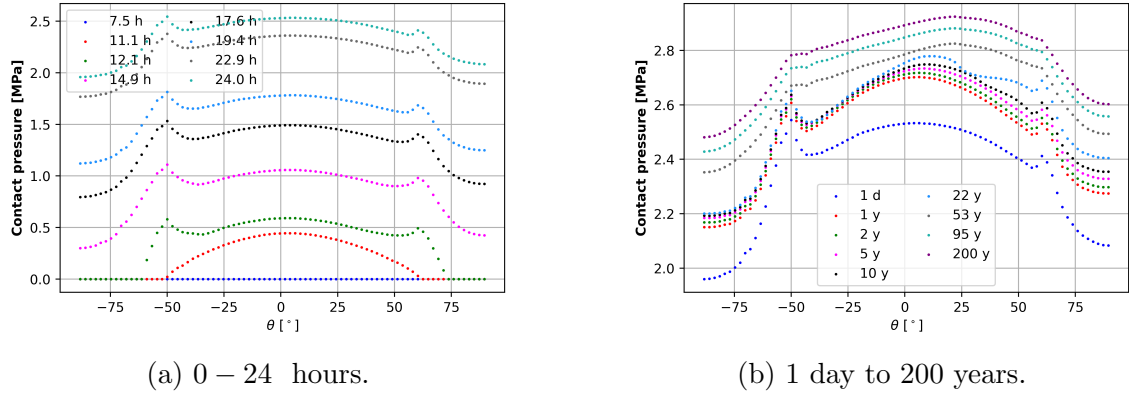
Figure 5.62: Contact pressure over time for different angles inside the lining.

Figure 5.62 shows the development of the contact pressure for different angles θ in the lining. After the sharp increase of the pressure during the excavation, it keeps slowly rising during the drainage and stabilization phases. The pressure at the end of the three phases are highlighted for the horizontal ($\theta = 0^\circ$) and vertical ($\theta = 90^\circ$) profiles. Table 5.4 compares the pressures reached when the mesh was only a quarter of the real configurations. It can be seen that, for all phases, the pressures are now higher, even if they are still close to their previous values.

Configuration	Excavation	Drainage	Stabilization
One half of the mesh	2.531	2.753	2.892
One fourth of the mesh	2.526	2.548	2.892

Table 5.4: Contact pressure (in MPa) at the end of the three phases for the horizontal profile.

Figure 5.63a shows that the contact is initiated a little before 10 hours and in a zone around $\theta = 0^\circ$, rather close to the middle of the lining. Quite rapidly after the contact, the single peak in pressure is divided into two local maxima (around 50° and -50°). As predicted by Figure 5.62, Figure 5.63b displays that the pressure keeps rising during all phases, with the maxima progressively disappearing

Figure 5.63: Distribution of the contact pressure depending on the angle θ .

once the drainage is over and leading to an homogenization of the pressure for $-50^\circ < \theta < 50^\circ$.

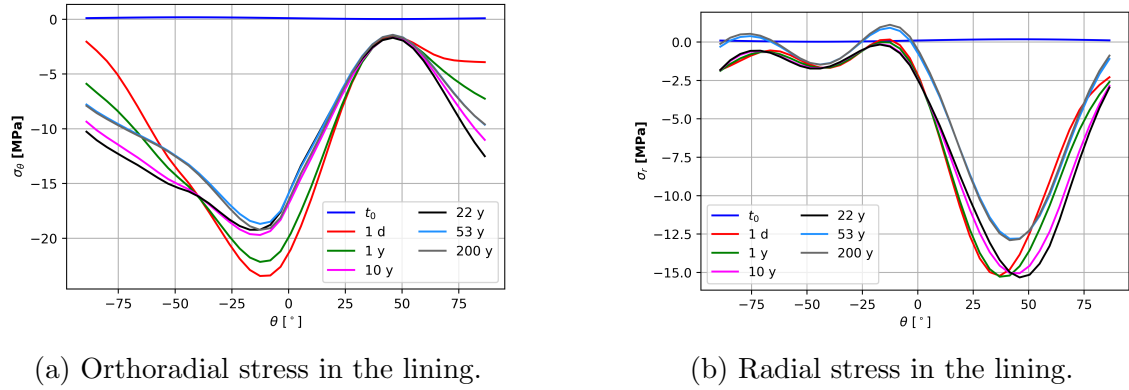
Figure 5.64: Radial and orthoradial stresses in the lining along θ .

Figure 5.64a shows that the greatest orthoradial stress σ_θ is reached around $\theta = -10^\circ$ and after one day. After the excavation, the stress decreases but always stays in compression. The radial stress σ_r displayed in Figure 5.64b is in compression during most of the simulation, although slight zones of tension emerge after 50 years. The maximum radial stress is now rather high as almost 17.5 MPa is reached around $\theta = 30^\circ$, which is a noticeable difference as the radial previously stayed quite limited.

Figure 5.65 illustrates the displacement of the points located on the outer edge of the lining. The displacement seems to be symmetrical. As it was previously the case, the ovalization of the lining can still be seen, with the extremities moving away from the centre of excavation while the points between -50° and 50° are converging towards the centre of excavation.

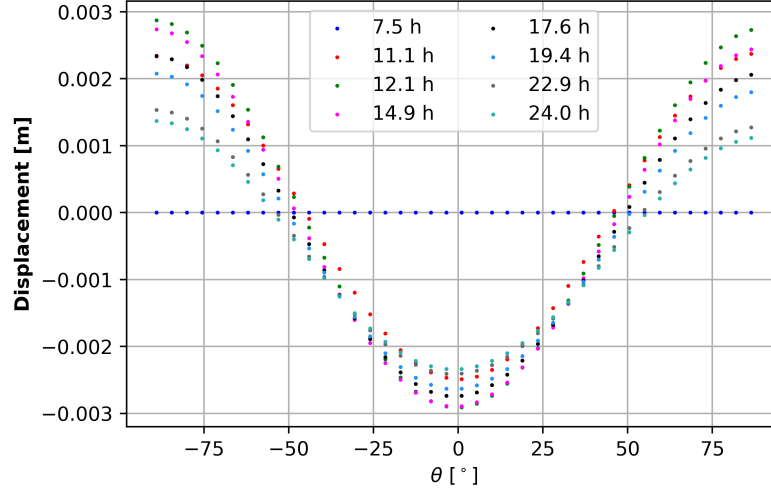


Figure 5.65: Displacement of the lining (< 0 towards the centre, > 0 away from the centre).

5.3.2 Configuration with gravity

Gravity can now be considered. As a reminder, the lining now has an outer radius of 2.4225 m and there is a slight interpenetration between the bottom of the lining and the rock.

The evolution of the orthoradial and radial stresses is shown in Figures 5.66a and 5.66b along the horizontal and vertical profiles. Although the general shape of the profiles does not change, it seems that the distance from the lining at which the rock remains undisturbed is lower with gravity, at approximately $x \approx 30 - 40$ m against $x \approx 50 - 60$ m previously. In Figure 5.66b, as the depth decreases from $y = -100$ m to $y = 100$ m, it can be seen that the curves are inclined. Points on the left correspond to higher depths which in turn lead to higher stresses.

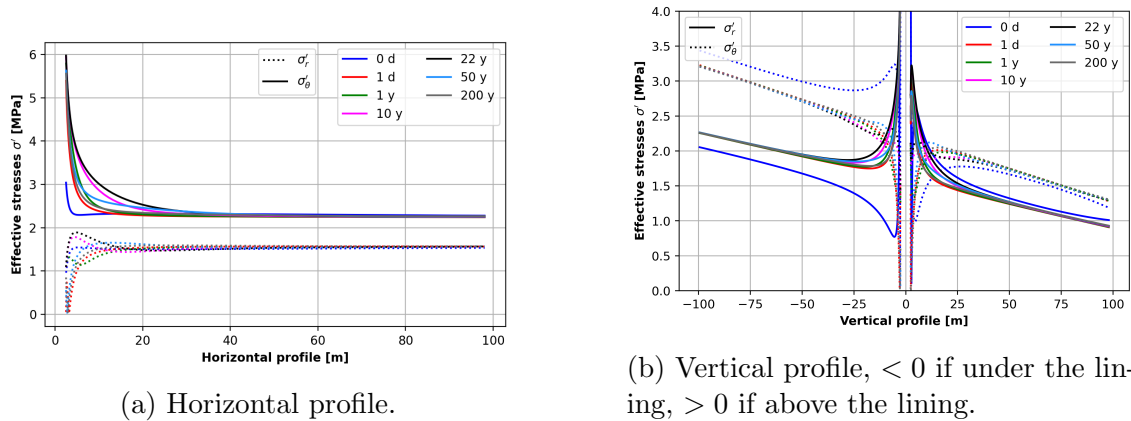


Figure 5.66: Radial and orthoradial stresses in the horizontal and vertical profiles.

Figure 5.67 illustrates the evolution of the contact pressure on the lining over time. Up until approximately 10 h, all pressures remain null except for a small zone around $\theta = -75^\circ$. These pressures are caused by the initial interpenetration between

the lining and the rock. Later, at approximately $t = 12.7$ h, the first contact between the rock and the lining is initiated around $\theta = -65^\circ$. In the configuration without gravity, the contact appeared in the middle of the lining. Adding gravity thus has the effect of shifting the first contact (other than the initial interpenetration) towards the bottom of the lining.

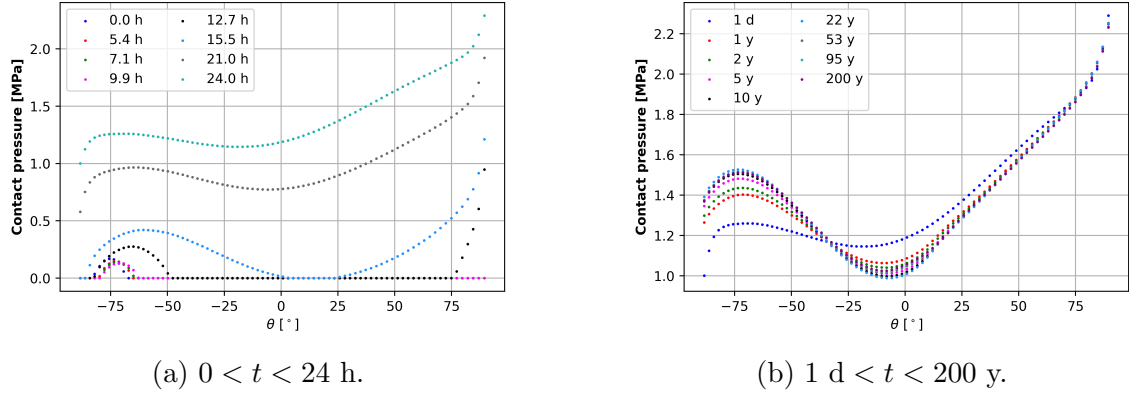


Figure 5.67: Contact pressure depending on the angle θ during the excavation, drainage and stabilization phases.

Later, after $t = 15.5$ h, a second contact appears at the top of the lining and quickly reaches higher values of pressure. Over longer periods of time, Figure 5.67b displays that the pressure increases in the bottom of the lining while the maximum pressure is located at the top of the lining and stabilizes during the following phases.

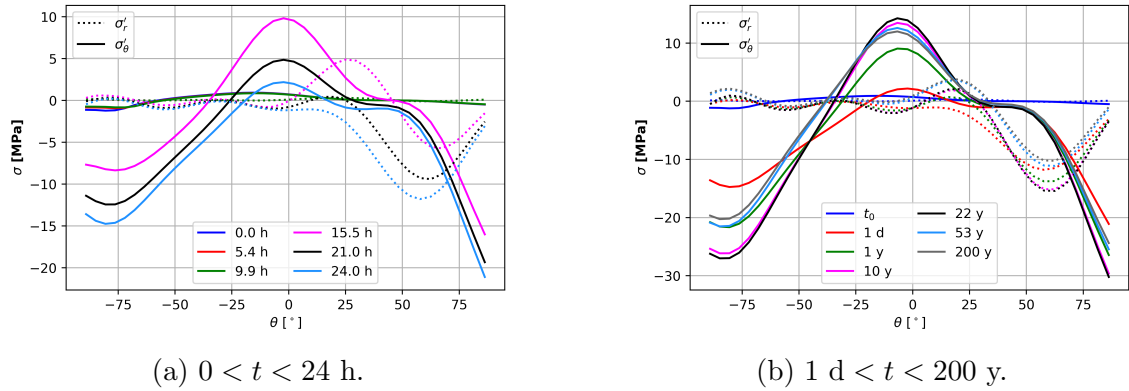


Figure 5.68: Radial and orthoradial stress in the lining during the excavation, drainage and stabilization phases.

As displayed in Figure 5.68a, during the excavation, the maximum value of the stresses is reached after 24 h in compression and after 15.5 h in tension. In the previous configuration without gravity, the stresses were lower and there was almost no tension in the lining. The radial stresses σ_r are again much lower than the orthoradial stresses. As can be seen in Figure 5.68b, once the excavation is over, the maximum values of the stresses are reached at the end of the drainage phase and then decrease in the next 170 years.

As shown in Figure 5.69, the stresses closer to the intrados of the lining often have a sign opposed to the stresses located on the extrados. At both the intrados and

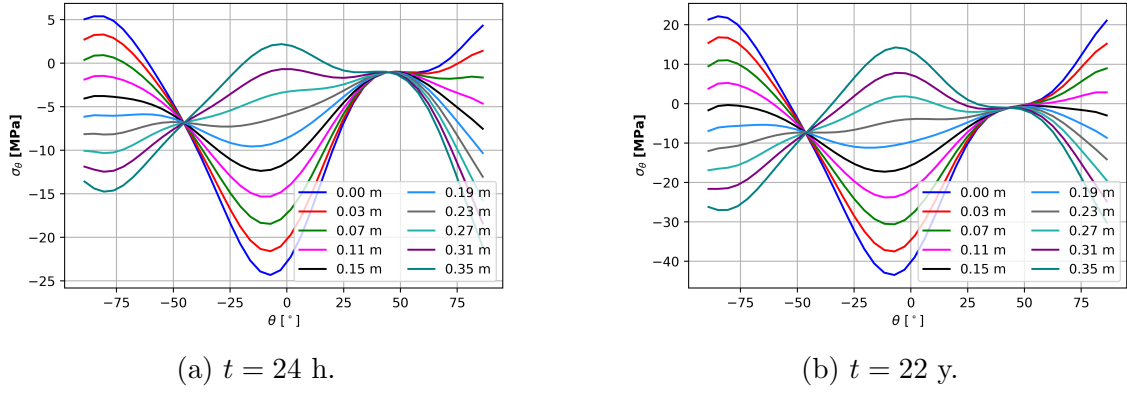
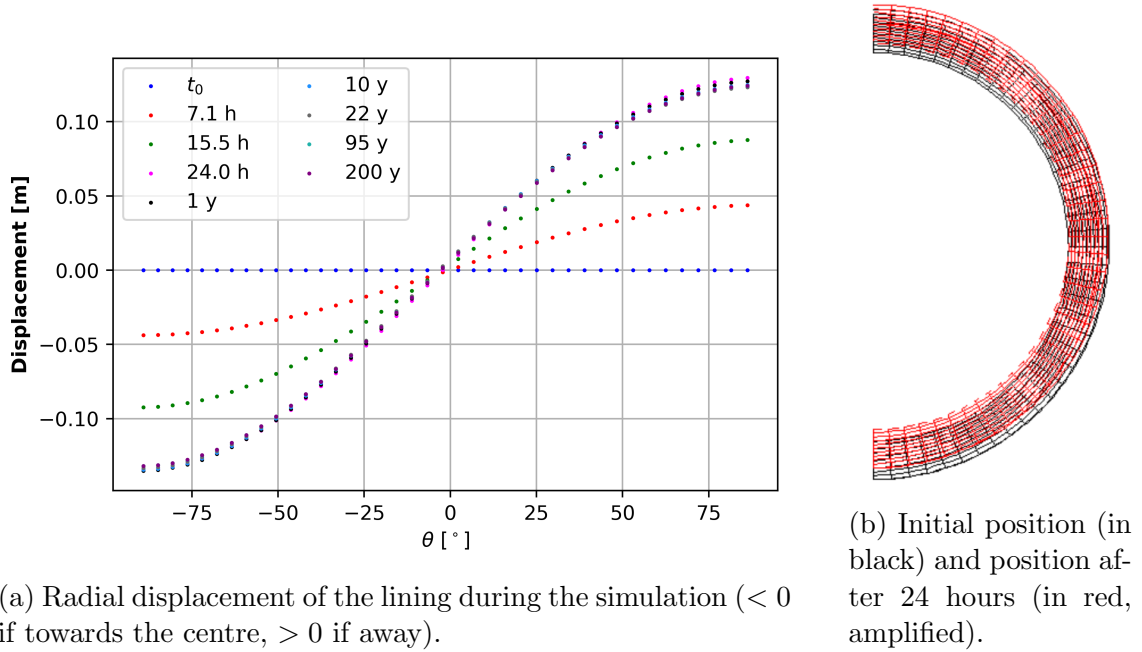


Figure 5.69: Orthoradial stress in the lining depending on the width.

extrados, the comparison between the end of the excavation and drainage confirms that the stresses are increasing both in compression and in tension.



(a) Radial displacement of the lining during the simulation (< 0 if towards the centre, > 0 if away).

(b) Initial position (in black) and position after 24 hours (in red, amplified).

Figure 5.70: Displacement of the lining.

Finally, Figure 5.70 illustrates the displacement of the lining. The ovalization can not be seen anymore as it seems that the lining is subjected to an upwards translation.

5.4 Conclusion

The results show that larger gap sizes delay the onset of contact and reduce the equilibrium contact pressure, while higher initial stresses and lower values of K_0 increase the anisotropy of the loading around the lining. The introduction of hydro-mechanical coupling reveals a significant time-dependent component to the lining loads, driven by the progressive dissipation of excess pore pressures in the Boom

clay over timescales of decades. These findings provide quantitative relationships between key design and geological parameters and the resulting lining loads, which can serve as practical guidelines for the optimisation of gallery linings in deep geological repositories.

The observed trends allow for useful recommendations for optimizing the design of coatings in the next Chapter as well as a description of the perspectives and limitations of this work.

Chapter 6

Conclusion

This work investigated the mechanical behaviour of concrete linings in deep geological repositories for radioactive waste, with a focus on the Boom clay context representative of the HADES underground research laboratory. Using the finite element code *LAGAMINE*, a progressive modelling strategy was adopted, starting from a simple isotropic, purely mechanical configuration and gradually introducing stress anisotropy and hydro-mechanical coupling. A parametric study was then conducted to evaluate the influence of key design parameters such as the gap size between the lining and the surrounding rock, the initial stress state, the coefficient of earth pressure at rest K_0 and the deconfinement profile on the contact pressure acting on the lining.

The numerical results were first validated against analytical solutions derived from the convergence-confinement method, demonstrating that both approaches converge to nearly identical results under appropriate conditions. This validation established confidence in the finite element model as a reliable tool for more complex configurations. The parametric study then revealed that the gap size plays a central role in governing the magnitude and timing of the contact pressure: a larger gap delays contact and reduces the pressure ultimately experienced by the lining, while a smaller gap leads to earlier contact and higher stresses. Similarly, the initial stress state and the degree of anisotropy significantly affect the distribution of contact pressure around the lining, with pronounced differences between the horizontal and vertical profiles when K_0 departs from unity. The hydro-mechanical coupling introduces a time-dependent behaviour driven by pore pressure dissipation, which is responsible for the long-term increase of effective stresses and contact pressure after the drainage phase. These findings offer quantitative relationships between the governing parameters and the resulting lining loads, which can serve as practical guidelines for optimising the design of concrete linings in deep geological repositories.

6.1 Limitations

Despite the insights gained, several important limitations must be acknowledged. First, the constitutive law adopted for the concrete lining is purely linear elastic throughout all simulations. While this assumption is standard in many preliminary studies and allows for tractable analytical comparison, it is physically unrealistic: concrete is a quasi-brittle material that exhibits non-linear behaviour, cracking, and stiffness degradation under sustained or high loading. The use of a more advanced constitutive model such as a damage mechanics or plasticity-based framework would be necessary to capture the true structural response of the lining under the high and evolving pressures characteristic of a deep repository environment.

Second, and perhaps more critically, creep is not taken into account in any of the simulations. This is a significant shortcoming given that several simulations extend over time horizons of up to 200 years. Over such periods, the concrete lining undergoes substantial time-dependent deformation. Concrete creep leads to a progressive redistribution of stresses and a reduction in stiffness, while the viscous behaviour of clay is well-documented and plays a central role in the long-term consolidation and deformation around excavations. Neglecting these phenomena means that the stress and displacement states computed at the end of long simulations are unlikely to be representative of reality. The mechanical state of the lining after two centuries of service would be governed to a large extent by creep effects that the current model cannot capture.

Third, the model does not account for the progressive material degradation of the concrete lining that inevitably occurs over multi-century time horizons. Beyond creep, concrete is subject to a range of physico-chemical deterioration mechanisms that alter its mechanical properties over time. The concrete lining may be exposed to chemically aggressive groundwater, elevated temperatures or irradiation, all of which can accelerate microstructural degradation through mechanisms such as alkali-silica reaction (ASR). The cumulative effect of these phenomena is a time-dependent reduction in the elastic modulus and load-bearing capacity of the lining that is not reflected in the present simulations. Incorporating such degradation laws into the constitutive behaviour of the concrete, for instance through a time-varying stiffness or a coupled chemo-mechanical framework, would be essential for a more physically faithful assessment of the long-term structural integrity of the lining.

6.2 Perspectives

Several directions could be pursued to extend and improve this work. The most natural continuation would be the development of a full three-dimensional finite element model. The current 2D plane strain approach, while well-suited for a first parametric study, is unable to account for the three-dimensional geometry of the excavation front and the progressive installation of the lining along the tunnel axis. A 3D model or, as an intermediate step, a 2D axisymmetric model in the longitudinal direction, would allow an explicit representation of the distance between the

excavation front and the point of lining activation, which has been shown in this work to be an important parameter linked to the gap size. Such a model would also enable a more realistic simulation of the construction sequence and the spatial variability of the loading.

Beyond the geometry, the constitutive modelling of the lining should be enriched. Adopting a non-linear or damage-based law for concrete would substantially improve the physical representativeness of the simulations, especially for the long time scales involved. Finally, the inclusion of thermal effects, relevant for repositories storing heat-generating waste, and a more complete treatment of the chemical interactions between the concrete and the clay environment would bring the model closer to the full thermo-hydro-mechanical-chemical (THMC) complexity of a real deep geological repository.

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