

Modelling of sealing structures for radioactive waste repositories in clay formations (en ce compris une introduction à la méthodologie de la recherche)

Auteur : Hayart, Dorian

Promoteur(s) : Collin, Frédéric

Faculté : Faculté des Sciences appliquées

Diplôme : Master en ingénieur civil des constructions, à finalité spécialisée en "civil engineering"

Année académique : 2025-2026

URI/URL : <http://hdl.handle.net/2268.2/26122>

Avertissement à l'attention des usagers :

Tous les documents placés en accès ouvert sur le site le site MatheO sont protégés par le droit d'auteur. Conformément aux principes énoncés par la "Budapest Open Access Initiative"(BOAI, 2002), l'utilisateur du site peut lire, télécharger, copier, transmettre, imprimer, chercher ou faire un lien vers le texte intégral de ces documents, les disséquer pour les indexer, s'en servir de données pour un logiciel, ou s'en servir à toute autre fin légale (ou prévue par la réglementation relative au droit d'auteur). Toute utilisation du document à des fins commerciales est strictement interdite.

Par ailleurs, l'utilisateur s'engage à respecter les droits moraux de l'auteur, principalement le droit à l'intégrité de l'oeuvre et le droit de paternité et ce dans toute utilisation que l'utilisateur entreprend. Ainsi, à titre d'exemple, lorsqu'il reproduira un document par extrait ou dans son intégralité, l'utilisateur citera de manière complète les sources telles que mentionnées ci-dessus. Toute utilisation non explicitement autorisée ci-avant (telle que par exemple, la modification du document ou son résumé) nécessite l'autorisation préalable et expresse des auteurs ou de leurs ayants droit.

Modelling of sealing structures for radioactive waste repositories in clay formations

Master's thesis presented in order to obtain the degree of Master of Science in Civil
Engineering by Hayart Dorian

AUTHOR:

HAYART DORIAN

THESIS SUPERVISOR :

COLLIN FRÉDÉRIC

JURY :

COLLIN FRÉDÉRIC

CORMAN GILLES

FRANÇOIS BERTRAND

DE LA VAISSIERE REMI

GERARD PIERRE

Année académique : 2025-2026

Acknowledgements

First of all, I would like to thank my supervisor, Mr. Collin, for the time he dedicated to helping me complete this work. Whether through our various meetings, his feedback after the reading of my work, or simply by answering my questions, his guidance was of great help.

Next, I would like to express my gratitude to Gilles Corman for his daily help and for answering my numerous questions throughout this semester. His reading of my work, his advice on how to structure my work, and, of course, his crucial assistance with Lagamine greatly contributed to the quality of this work.

I would also like to thank the members of the jury for the time they will dedicate to reading and evaluating my work.

Furthermore, I am deeply grateful to my family for their unwavering support from the very beginning.

Finally, I want to thank the friends with whom I shared these past five years, making them so much more enjoyable.

Abstract

To meet the climate goals established in the Paris Agreement, decarbonisation of global energy production is necessary. To do so, nuclear energy has a key role to play. However, the nuclear industry faces several challenges. Among the latter, the production of high level radioactive waste represents a threat to the biosphere; their safe management is thus of great importance. Currently, deep geological disposal systems are widely recognised as the safest long-term option. It consists of the burial of the waste in a stable geological formation, as well as the implementation of an engineered barrier system. Nevertheless, the latter is subject to complex coupled hydro-mechanical phenomena, particularly due to the presence of bentonite, a clay exhibiting a significant swelling capacity upon hydration. Understanding these phenomena is thus essential to ensure the safety of this solution.

Accordingly, in this work, the finite element code **LAGAMINE** is employed to numerically simulate the lifetime of the sealing structure. A progressive modelling strategy is adopted, starting with small scale benchmark exercises before a gradual evolution of the complexity of the models, up to the representation of the entire sealing structure. The emphasis is first placed on the behaviour of bentonite and on a sensitivity analysis of its mechanical parameters. Then, its behaviour under a situation closer to reality is studied through a larger scale 2D axisymmetric model, with a first quantification of several variables, such as the swelling pressure or the displacement of the bentonite sample. Lastly, the behaviour of the entire sealing structure is studied, still through a 2D axisymmetric model. In this last model, different phases of the lifetime of the gallery are modelled, from the excavation works to the end of the saturation of the structure.

In terms of results, this work first showed that the employed numerical model is capable of reproducing the experimental data corresponding to laboratory tests of a bentonite sample. Then, it showed that the first type of bentonite used in this work develops too much swelling pressure, making it necessary to switch to a different mixture of bentonite, composed of 20% of powder and 80% of pellet. Subsequently, it demonstrated that this new mixture effectively reduces the swelling pressure, thereby satisfying the requirements of the repository site. Finally, the full-scale model yielded results that are consistent with those presented before, confirming that the requirements are also satisfied for the entire sealing structure.

Résumé

Pour atteindre les objectifs climatiques établis par l'Accord de Paris, la décarbonisation de la production mondiale d'énergie est nécessaire. Pour ce faire, l'énergie nucléaire a un rôle clé à jouer. Cependant, l'industrie nucléaire est confrontée à de nombreux défis. Parmi ces derniers, la production de déchets radioactifs de haute activité représente une menace pour la biosphère; leur gestion sûre revêt donc une importance capitale. Actuellement, les systèmes de stockage géologique profond sont largement reconnus comme l'option la plus sûre à long terme. Ils consistent en l'enfouissement des déchets dans une formation géologique stable, ainsi qu'en la mise en place d'un système de barrières ouvragées. Néanmoins, ce dernier est soumis à des phénomènes hydro-mécaniques couplés complexes, en particulier en raison de la présence de bentonite, une argile présentant une capacité de gonflement importante lors de son hydratation. La compréhension de ces phénomènes est donc essentielle pour garantir la sûreté de cette solution.

Par conséquent, le code éléments finis **LAGAMINE** est utilisé dans ce travail pour simuler numériquement la durée de vie de la structure de scellement. Une stratégie de modélisation progressive est adoptée, partant d'exercices de benchmark à petite échelle avant une évolution graduelle de la complexité des modèles, pour aboutir à la représentation de la structure de scellement complète. L'accent est d'abord mis sur le comportement de la bentonite et sur une analyse de sensibilité de ses paramètres mécaniques. Ensuite, son comportement dans une situation plus proche de la réalité est étudié à travers un modèle 2D axisymétrique à plus grande échelle, avec une première quantification de plusieurs variables, telles que la pression de gonflement ou le déplacement de l'échantillon de bentonite. En dernier lieu, le comportement de la structure de scellement complète est étudié, toujours au moyen d'un modèle 2D axisymétrique. Dans ce dernier modèle, différentes phases de la durée de vie de la galerie sont modélisées, depuis les travaux d'excavation jusqu'à la fin de la saturation de la structure.

En termes de résultats, ce travail a d'abord montré que le modèle numérique employé est capable de reproduire les données expérimentales correspondant aux essais en laboratoire d'un échantillon de bentonite. Il a ensuite montré que le premier type de bentonite utilisé dans ce travail développe une pression de gonflement trop importante, rendant nécessaire le passage à un mélange différent, composé de 20% de poudre et de 80% de pellets. Il a par la suite été démontré que ce nouveau mélange réduit efficacement la pression de gonflement, satisfaisant ainsi les exigences du site de stockage. Enfin, le modèle complet a fourni des résultats cohérents avec ceux présentés précédemment, confirmant que les exigences sont également satisfaites pour la structure de scellement dans son ensemble.

Contents

1	Introduction	2
1.1	Context	2
1.2	Belgian concept	5
1.3	French concept	7
1.4	Sealing material	8
1.5	Objectives and content of this work	9
2	Literature review	10
2.1	Natural barrier material	10
2.1.1	Callovo-Oxfordian argillite	10
2.1.1.1	Geological context	10
2.1.1.2	Mechanical parameters	11
2.1.1.3	Hydraulic parameters	11
2.1.1.4	Excavation Damaged Zone	11
2.2	Engineered barrier materials	13
2.2.1	Bentonite based seal	13
2.2.1.1	Generalities	13
2.2.1.2	Composition of bentonite	15
2.2.1.3	Behaviour of bentonite	15
2.2.2	COx-Sand backfill	20
2.2.3	Concrete	20
2.3	Numerical models	21
2.3.1	Balance equations	22
2.3.1.1	Solid mass balance equation	22
2.3.1.2	Fluids mass balance equation	22
2.3.1.3	Momentum balance equation	22
2.3.2	Constitutive equations of the natural barrier material and the concrete	23
2.3.2.1	Water retention model	23
2.3.2.2	Mechanical model	23
2.3.2.3	Hydraulic model	24
2.3.3	Constitutive equations of the engineered barrier materials	25
2.3.3.1	Water retention model	25
2.3.3.2	Mechanical model	27
2.3.3.3	Hydraulic model	31
2.3.4	Interface	31
2.3.4.1	Mechanical model	32
2.3.4.2	Hydraulic model	32
2.4	Existing numerical works	32
2.5	Preliminary conclusion	34

3	Benchmark exercises	36
3.1	Introduction	36
3.2	Numerical models and their objective	36
3.3	Calibration	38
3.4	Reference configuration	41
3.5	Technological gaps	43
3.6	Sensitivity analysis	46
3.6.1	Influence of the mechanical parameters on the stress path	46
3.6.2	Influence of the mechanical parameters on the mean total stress	49
3.7	Preliminary conclusion	51
4	Test cases for the engineered barrier material	52
4.1	Introduction	52
4.2	Numerical models	52
4.3	Test case 1	55
4.3.1	Distributed water inflow	55
4.3.1.1	Results for the new mixture	58
4.3.2	Localised water inflow	61
4.3.3	Preliminary conclusions	62
4.4	Test case 2	63
4.4.1	Distributed water inflow	63
4.4.2	Localised water inflow	67
4.4.3	Preliminary conclusion	69
5	Full test case	72
5.1	Introduction	72
5.2	Numerical model	72
5.3	Results	75
5.3.1	Excavation phase	75
5.3.2	Drainage phase	77
5.3.3	Saturation phase	78
5.4	Preliminary conclusion	83
6	Conclusion	84
6.1	Limitations and perspectives	85
A	Graphs for Test case 2	92

List of Figures

1.1	Total energy and electricity demand (IEA 2025)	2
1.2	Global energy demand and source of electricity generation (Energy Institute 2025)	3
1.3	Conceptual illustration of the waste classification scheme (IAEA 2009)	4
1.4	Conceptual scheme of a deep geological repository for nuclear waste (Gens et al. 2009)	5
1.5	Engineered barrier systems components of actual or planned geological disposal/storage facilities (Ojovan et al. 2022)	5
1.6	Supercontainer for vitrified high-level waste (left) and uranium-oxide irradiated fuel (right) (ONDRAF 2013)	6
1.7	Schematic cross-sections of the supercontainer design for vitrified high-level waste (left) and uranium-oxide irradiated fuel (right) (ONDRAF 2013)	6
1.8	Conceptual schemes of the French concept (M. Alonso et al. 2024, Sellin et al. 2025)	7
1.9	Lifetime phases of the gallery (M. Alonso et al. 2024)	8
1.10	Possible forms of bentonite	9
2.1	Geological zone of the URL (ANDRA 2005)	10
2.2	Experimental data for the retention curve of Callovo-Oxfordian argillite (Charlier et al. 2013)	12
2.3	Experimental data for the water and gas permeability evolution with the degree of saturation for Callovo Oxfordian argillite (Charlier et al. 2013)	12
2.4	Evolution of permeability with the distance from gallery wall (Pardoën et al. 2016)	13
2.5	Structure of montmorillonite (María Victoria Villar 2000)	14
2.6	Schematic diagram of the organisation of a clay material (María Victoria Villar 2000)	15
2.7	Pore size distribution of bentonite/sand mixture compacted to different dry density with a water content of about 11.0% (Wang et al. 2013)	15
2.8	Grain size distribution MX-80 bentonite and sand (Wang et al. 2013)	16
2.9	Effect of the void ratio on the water retention behaviour of MX-80 bentonite (Seiphoori et al. 2014)	17
2.10	Water retention behaviour of highly compacted MX-80 granular bentonite during cyclic variation of the total suction (Seiphoori et al. 2014)	17
2.11	Water retention curves of bentonite–sand mixture and pure bentonite (Wang et al. 2013)	18
2.12	Evolution of hydraulic conductivity with the effective montmorillonite dry density (EMDD) (Dixon et al. 2023)	18
2.13	Variation in intrinsic permeability with porosity from saturated hydraulic conductivity tests (María Victoria Villar 2000)	19
2.14	Evolution of the swelling pressure with dry density (Dieudonné 2016)	19
2.15	Influence of initial water content on the swelling pressure (M. V. Villar et al. 2008)	19

2.16	Evolution of the yield stress with bentonite void ratio for two mixtures (Wang et al. 2013)	20
2.17	Backfill configurations	20
2.18	Compression curve for saturated condition (adapted from E. E. Alonso et al. 1990)	28
2.19	Compression curves for saturated and unsaturated states (E. E. Alonso et al. 1990)	28
2.20	Yield curves of the Barcelona Basic Model for isotropic stress states (Dieudonné 2016)	29
2.21	Representation of the Modified Cam-Clay model and the Barcelona Basic Model in the (p-q) plane (Dieudonné 2016)	30
2.22	Model geometry used by M. Alonso et al. 2024	33
2.23	Evolution of the mean swelling pressure (M. Alonso et al. 2024)	33
2.24	Evolution of the longitudinal displacement (M. Alonso et al. 2024)	33
2.25	Detailed geometry of the model used in Toprak et al. 2024	33
2.26	Evolution of swelling pressure, liquid pressure, and vertical displacements (Toprak et al. 2024)	34
2.27	Results and performance targets (Toprak et al. 2024)	34
3.1	Geometry of the model for calibration (Toprak et al. 2025)	37
3.2	Representation of the reference configuration (Toprak et al. 2025)	37
3.3	Representation of the model for technological gaps	37
3.4	Calibration - Water retention curve	38
3.5	Calibration - Evolution of permeability with dry density	39
3.6	Calibration - Evolution of swelling pressure with dry density	39
3.7	Calibration - Evolution of mean net stress with suction	39
3.8	Reference configuration - Evolution of suction	41
3.9	Reference configuration - Evolution of the degree of saturation	41
3.10	Reference configuration - Mean net stress evolution	41
3.11	Reference configuration - (a) Evolution of porosity and (b) Evolution of permeability	42
3.12	Reference configuration - Evolution of dry density	42
3.13	Reference configuration - Evolution of suction with the mean effective stress	43
3.14	Technological gaps - Gap aperture	43
3.15	Technological gaps - (a) Evolution of the normal contact pressure and (b) Evolution of the tangential contact pressure	44
3.16	Evolution of the radial stress depending on the x-axis	44
3.17	Technological gaps - Evolution of porosity depending on the x-axis	44
3.18	Technological gaps - (a) Evolution of porosity and (b) Evolution of permeability	45
3.19	Technological gaps - Evolution of dry density	45
3.20	Technological gaps - Evolution of suction	46
3.21	Technological gaps - Evolution of the degree of saturation	46
3.22	Technological gaps - Evolution of the mean total stress	46
3.23	Technological gaps - Stress path	46
3.24	Influence of κ on the stress path	47
3.25	Influence of κ_s on the stress path	47
3.26	Influence of p_0^* on the stress path	48
3.27	Influence of α_p on the stress path	48
3.28	Influence of $\lambda(0)$ on the reference configuration	49
3.29	Influence of κ on the mean total stress	49
3.30	Influence of κ_s on the evolution of the mean total stress	50
3.31	Influence of p_0^* on the evolution of the mean total stress	50
3.32	Influence of α_p on the evolution of the mean total stress	50

3.33	Influence of $\lambda(0)$ on the mean total stress	51
4.1	First test cases of ANCHORS project (Talandier et al. 2025)	53
4.2	Schematic representation of the models for the two first test cases (not at scale)	53
4.3	Second test cases of ANCHORS project (Talandier et al. 2025)	54
4.4	Measurement points for test case 1	56
4.5	Case 1a - Temporal evolution of suction	56
4.6	Case 1a - Temporal evolution of the degree of saturation	56
4.7	Case 1a - Temporal evolution of porosity	57
4.8	Case 1a - Vertical displacements	57
4.9	Case 1a - Temporal evolution of stresses	57
4.10	Powder-Pellet - Temporal evolution of suction	58
4.11	Powder-Pellet - Temporal evolution of the degree of saturation	58
4.12	Powder-Pellet - Evolution of porosity	60
4.13	Powder-Pellet - Evolution of permeability	60
4.14	Powder-Pellet - Vertical displacements	60
4.15	Powder-Pellet - Temporal evolution of stresses	61
4.16	Case 1b - Water pressure at four chosen times	61
4.17	Case 1b - Evolution of suction	62
4.18	Case 1b - Time evolution of the degree of saturation	62
4.19	Case 1b - Vertical displacement	62
4.20	Case 1b - Evolution of axial and radial stresses	63
4.21	Test case 2 - Measurement points	64
4.22	Case 2a - Water pressure	64
4.23	Case 2a - Evolution of the degree of saturation in the two samples	65
4.24	Case 2a - Evolution of suction in the two samples	65
4.25	Case 2a - Evolution of permeability with time in the two cores	66
4.26	Case 2a - Evolution of axial and radial stresses in bentonite	66
4.27	Case 2a - Evolution of axial and radial stresses in backfill	66
4.28	Case 2a - Evolution of the vertical displacement	67
4.29	Case 2b - Water pressure at four chosen times	67
4.30	Case 2b - Evolution of suction with time	68
4.31	Case 2b - Evolution of the degree of saturation with time	68
4.32	Case 2b - Evolution of axial stresses with time	69
4.33	Case 2b - Evolution of radial stresses with time	69
4.34	Case 2b - Evolution of vertical displacement with time	69
5.1	Schematic representation of the studied structure (not at scale)	72
5.2	Model for the third test case from ANCHORS project (Talandier et al. 2025) (not at scale)	73
5.3	Phases undergone by the gallery from Corman et al. 2025b	74
5.4	Measurement points for the full test case	75
5.5	Water pressure at 50 days	75
5.6	Full test case - Temporal evolution of the water pressure	76
5.7	Full test case - Evolution of the mean total stress	76
5.8	Full test case - Water pressure in the sample during phase 2	77
5.9	Full test case - Temporal evolution of the water pressure during phase 2	78
5.10	Full test case - Temporal evolution of the mean total stress during phase 2	79
5.11	Full test case - Water pressure in the sample during phase 3	80
5.12	Full test case - Temporal evolution of the water pressure during phase 3	81
5.13	Full test case - Temporal evolution of the degree of saturation during phase 3	81

5.14	Full test case - Temporal evolution of the mean total stress during phase 3 . . .	82
5.15	Full test case - Horizontal displacement in bentonite	83
5.16	Full test case - Axial and radial total stresses in the bentonite sample during phase 3	83
A.1	Case 2a - Bentonite - First 20 days of evolution of saturation	92
A.2	Case 2b - Backfill - First 40 days of the evolution of the degree of saturation . .	92

List of Tables

2.1	Mechanical parameters of Callovo-Oxfordian argillite (Charlier et al. 2013, Armand et al. 2014), Zhang et al. 2019, Conil et al. 2018	11
2.2	Hydraulic parameters for the Callovo-Oxfordian argillite (Charlier et al. 2013) .	12
2.3	Properties of MX-80 bentonite (M. V. Villar 2007,Madsen 1998, Tang et al. 2008,Wang et al. 2013,Gatabin et al. 2016)	16
2.4	Mechanical and hydraulic parameters of backfills (Corman et al. 2025a)	21
2.5	Mechanical and hydraulic parameters of concrete (Corman et al. 2025a, Charlier et al. 2013)	21
2.6	Calibrated parameters of the microstructure evolution model (Dieudonné 2016) .	27
3.1	Mohr-Coulomb model parameters for the contact element (Toprak et al. 2025) .	38
3.2	Calibrated HM parameters of MX-80 bentonite (Toprak et al. 2025)	40
4.1	Backfill parameters (Talandier et al. 2025)	55
4.2	Parameters of the 20% powder 80% pellet mixture	59
5.1	Hydraulic and mechanical parameters of the host rock, EDZ, and concrete lining	73

Chapter 1

Introduction

1.1 Context

Worldwide, the consumption of energy keeps increasing. Between 2020 and 2050, an increase of 30 % in energy consumption is expected, with electricity production expected to double (IAEA 2021). This trend is shown in Figure 1.1, where two scenarios are studied: the Stated Policies Scenario (STEPS), which is based on the already adopted policies and regulations, and the Current Policies Scenario (CPS), which takes into account the policies and regulations that have been tabled but not yet adopted. In the STEPS, an 8% increase in energy demand is predicted between 2025 and 2035, while the CPS predicts an increase of 14% (IEA 2025).

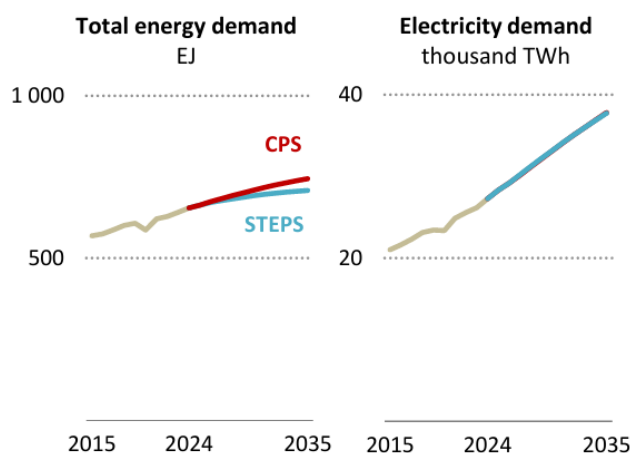


Figure 1.1: Total energy and electricity demand (IEA 2025)

Moreover, looking at the source of this energy, most of it is not renewable. In 2024, oil and coal were still the most used sources of energy, representing, respectively, 30% and 27% of the total energy supply, while renewable energy only represents 13%. This large share of non-renewable energy contributes to a continued deterioration of the climate. For instance, 2024 was the hottest year ever recorded, with an increase in temperature of more than 1.5 °C compared to pre-industrial levels (IEA 2025).

To slow down these threats to living conditions, the world has set climate goals. Indeed, it was decided in the Paris Agreement to limit the global temperature increase by 1.5 to 2 °C by 2050. For instance, missing the 2 °C target could trigger feedback loops with profound implications for human societies: rising sea levels could displace millions of people, many more would face increasingly frequent and intense heatwaves, major biodiversity related damages would occur, up to half of the world population could be exposed to extreme heat during summer, world

food supplies could be put under serious strain, and many more far-reaching impacts (IAEA 2020). However, the world is currently far from on track to meet the goals set out in the Paris Agreement (IAEA 2020), meaning that further actions must be taken to reach these objectives. Among these actions, one can cite the decarbonisation of global energy production and use by 2050. In particular, this decarbonisation means a transition from fossil fuels toward renewable sources (Calvin et al. 2023). For example, in 2024, more than 680 GW of renewable energy capacity was added. This transition also involves an increase in the share of electricity in global energy consumption, from around one fifth nowadays to around one third by 2035 (IEA 2025).

Nevertheless, renewable energies also present some disadvantages, such as the intermittency of the capacity. This is why another source of energy is needed in order to facilitate the integration of renewable energies. In most scenarios, this facilitation is fulfilled through an increase in nuclear capacity (Corman 2023). Indeed, nuclear energy allows for enhanced grid flexibility, resilience, and addresses the issue of intermittency. In practice, nuclear capacity is expected to expand by 70% by 2035 in a scenario where the world succeeds in reaching Net Zero Emissions (NZE) by 2050 (IEA 2025).

Nuclear energy relies on the release of energy that occurs when the nucleus of an atom splits into smaller nuclei – a process known as nuclear fission. In most power plants, uranium-235 is used as fuel. When hit by a neutron, its nucleus splits into two smaller nuclei, releasing two or three neutrons. These newly released neutrons then strike other uranium-235 atoms, causing them to split as well. This process creates a chain reaction where each fission event releases energy in the form of heat and radiation. This heat is used to convert water into steam, which drives turbines, that are connected to an electric generator, thereby producing low-carbon electricity (Galindo 2025). Worldwide, in 2024, nuclear power accounted for about 5% of the global energy demand and 9% of global electricity generation (Energy Institute 2025), as shown in Figure 1.2. In Belgium and France, nuclear energy respectively represented about 15% (SPF Economie 2026, Energy Institute 2025) and 46% (Energy Institute 2025) of the total energy supply in 2024.

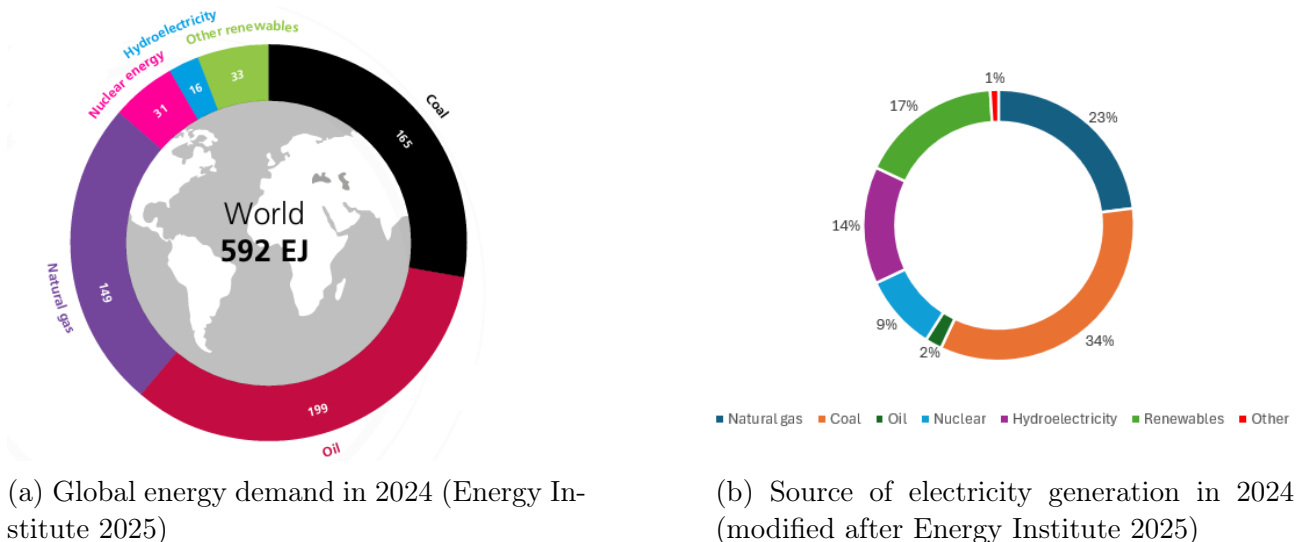


Figure 1.2: Global energy demand and source of electricity generation (Energy Institute 2025)

Nonetheless, nuclear energy is also subject to criticism. Obviously, accidents occurred, responsible for the release of radioactivity in the environment (Lelieveld et al. 2012). Another critical issue with nuclear energy is the production of radioactive waste. This waste has to be stored, because it contains long-lived radionuclides whose decay is particularly slow, so it can remain

in the environment for up to one hundred thousand years, posing a threat to the biosphere (Corman 2023). Of course, radioactive waste can be produced in numerous ways other than nuclear electricity production: by military activities, in hospitals, agriculture, and many other fields (IAEA n.d.). In order to better understand the potential harm to humans, the International Atomic Energy Agency (IAEA) has classified waste into different categories, depending on their activity content and half-life, as shown in Figure 1.3

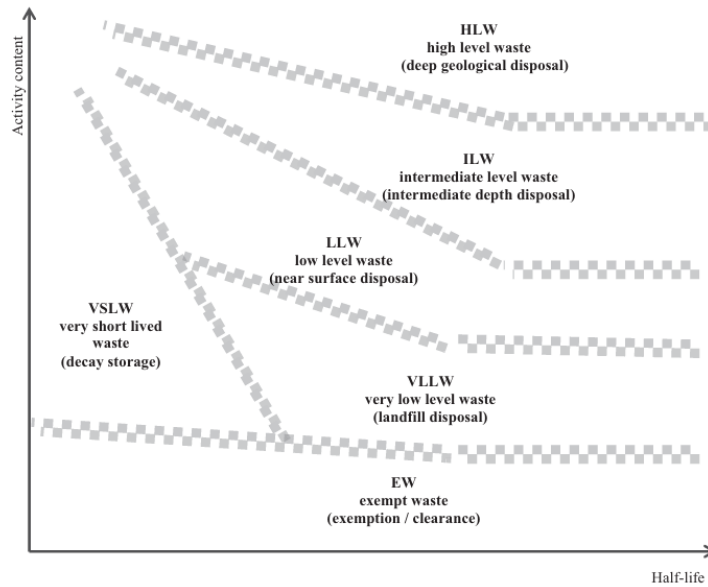


Figure 1.3: Conceptual illustration of the waste classification scheme (IAEA 2009)

These waste must be disposed of by a system that isolates them and ensures that no significant concentration of radioactive substances can reach the environment (OECD NEA 1995). In the case of high level waste, several disposal options have been studied. In particular, as stated by the council directive 2011/70/EURATOM of 19 July 2011 (EU 2011), "deep geological disposal represents the safest and most sustainable option as the end point of the management of high-level waste and spent fuel considered as waste".

Deep geological disposal systems consist in burying waste within a stable geological formation, at a depth of several hundred metres. Two types are possible: deep boreholes drilled from the surface or geological repositories, the latter being the most likely possibility to be used (Vaníček 2003). An example of mined repositories is shown in Figure 1.4. The geological systems have several roles to fulfil, as stated by IAEA (2003), they aim to isolate the waste from the surface, protect the biosphere, isolate the waste from human activities, contain the waste, limit the release, and disperse and dilute the waste.

The deep geological disposal system is based on a multi-barrier approach: a natural barrier system (NBS) and an engineered barrier system (EBS). While the NBS consists of a stable host rock, such as granite, clay, tuff and salt (Li et al. 2025), the EBS can consist of different components, namely the waste form, the container, the backfill, the buffer material and geotechnical sealing structures (Ojovan et al. (2022), Takada et al. (1998)). In the EBS, the first barrier is the waste form, aiming to immobilise and prevent the release of radionuclides. It can take the form of cement, glass, ceramic, etc. A second barrier is the container, also aiming to prevent radionuclide release as well as waste conditioning to allow for safe handling. It is often made of steel or copper. Then, the backfill/buffer material is used to fill the gaps between the container

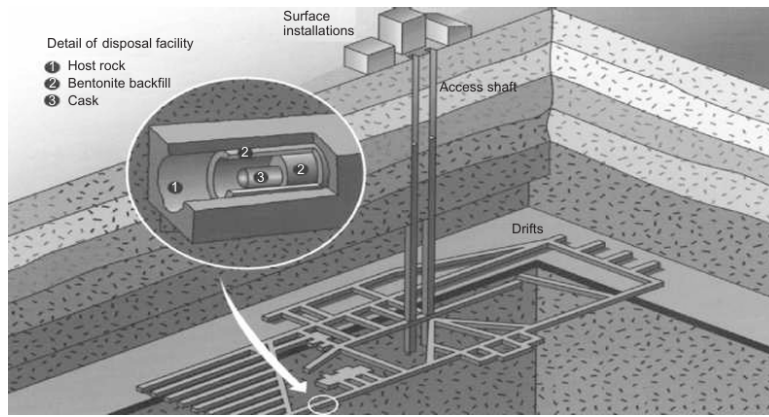


Figure 1.4: Conceptual scheme of a deep geological repository for nuclear waste (Gens et al. 2009)

and the geological formation, with the objective of delaying the release of radionuclides into the environment. The backfill often consists of clay, in particular bentonite, because of its swelling capacity, low permeability, and retention capacities (Dieudonné 2016, Ojovan et al. 2022). On the one hand, the EBS aims to confine the waste for thousands of years. On the other hand, the objective of the NBS is to delay the waste migration once the EBS has deteriorated (Corman 2023).

In several countries, underground research laboratories (URL) have been drilled to study different solutions to dispose of the waste. Ojovan et al. (2022) resumed some of the concepts used, presented in Figure 1.5 In particular, the next sections bring the focus to two similar concepts

Country	Waste	Wasteform	Container	Backfill
Belgium	HLW SNF	Glass SNF	Stainless steel Steel	Clay, Bentonite, Quartz sand, Graphite
Canada	SNF	SNF	Carbon steel, Copper	Bentonite, Sand, Clay, Crashed rock
China	HLW	Glass	Stainless steel	Bentonite
Czech Republic	ILW HLW SNF	Concrete Glass SNF		
Finland	SNF	SNF	Copper, Iron	Bentonite, Crushed host rock
France	ILW HLW SNF	Various Glass SNF	Stainless steel, Concrete Stainless steel, Steel Stainless steel	Concrete lining Bentonite Bentonite, Disposal tube
Germany ¹	LILW	Not considered as EBS	Not considered as EBS	Salt concrete
Japan	HLW	Glass	Carbon steel	Bentonite, Sand
Korea	SNF	SNF	Steel, Copper	Bentonite, Sand
Russia	HLW	Glass	Steel, Stainless steel	Bentonite, Concrete
Spain	SNF	SNF	Carbon steel	Bentonite
Sweden	SNF	SNF	Copper, Iron	Bentonite
Switzerland	HLW	Glass	Steel	Bentonite
UK	LILW	Cement	Steel, Concrete	Cement based
	TRU ² /WIPP ³	Various	Steel	MgO
US	SNF/YMP ⁴	SNF	Stainless steel, Ni-based alloy	-
	HLW/YMP	Glass	Stainless steel	-

¹ Morsleben; ² Transuranic waste; ³ Waste Isolation Pilot Plant; ⁴ Yucca Mountain Plant (Nevada). SNF disposal at the YMP was halted during the Obama administration with Blue Ribbon Commission assigned and tasked with the development of a perspective for the NWM in the US.

Figure 1.5: Engineered barrier systems components of actual or planned geological disposal/storage facilities (Ojovan et al. 2022)

that opted for clay in their repository design: the Belgian and French concepts.

1.2 Belgian concept

In Belgium, ONDRAF/NIRAS is the institution responsible for the safe management of all radioactive waste. They study a poorly indurated clay as the natural barrier. In particular, Boom clay was chosen to be studied due to several of its properties, which are favourable to fulfil the objective of the natural barrier. These properties are explained by ONDRAF (2013):

- Low permeability: limits the impact of the perturbations
- Self-sealing properties: under hydration, Boom clay swells and seals the fractures that can be caused by the excavation works. This sealing happens within weeks.
- Chemical buffering properties: the minerals of the Boom clay act as a buffer against chemical changes.

The host rock is studied at the HADES URL in Mol (Belgium) even if the latter is for demonstration purposes only, and will not be used as the final repository location. Currently, a total volume of $2,979m^3$ is planned for the high level waste, 99.9% of the latter coming from the electro-nuclear industry (ONDRAF 2024).

On the other hand, the chosen engineered barrier system is called a "supercontainer" design. It consists of the following elements: the overpack, the concrete buffer, a potential envelope, the backfill of the galleries, and the concrete lining of the galleries. This supercontainer, shown in Figures 1.6 and 1.7, is explained hereafter.

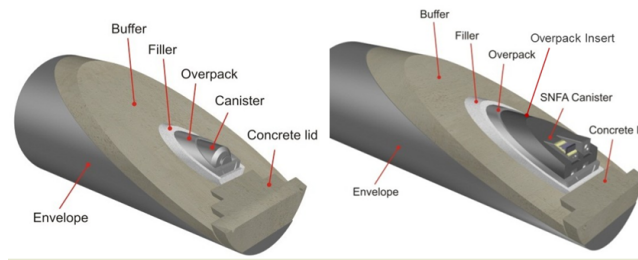


Figure 1.6: Supercontainer for vitrified high-level waste (left) and uranium-oxide irradiated fuel (right) (ONDRAF 2013)

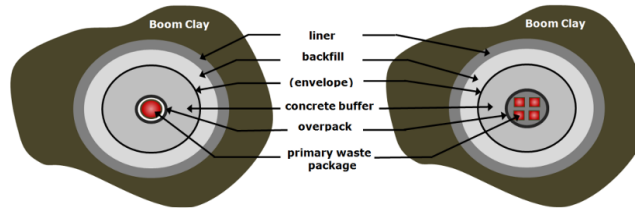


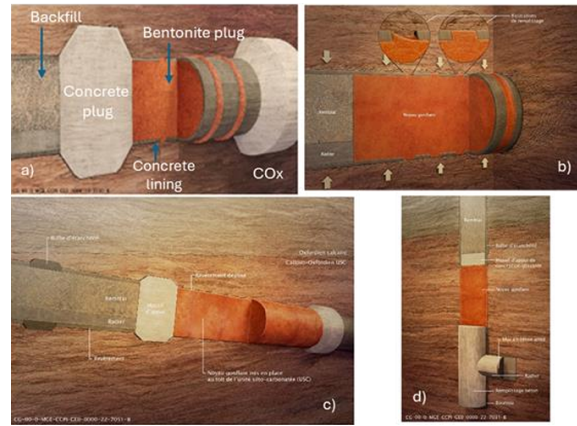
Figure 1.7: Schematic cross-sections of the supercontainer design for vitrified high-level waste (left) and uranium-oxide irradiated fuel (right) (ONDRAF 2013)

First, the waste packages are placed in a carbon steel overpack. The choice of carbon steel is due to the fact that, under the alkaline conditions of the supercontainer, the corrosion will be generalised rather than localised, the latter being less predictable, and because carbon steel can be welded in reasonable thicknesses. Inside the overpack, the additional space is filled with an inert granular material or powder and an inert gas before sealing. Then, the overpack is placed into a buffer that consists of concrete with Portland cement. The purpose of the buffer is to keep a favourable chemical environment for the overpack. Indeed, by maintaining an alkaline environment, corrosion will be prevented due to the formation of a passive layer around the overpack. Next, it is possible to add a stainless steel envelope, which could be used as a mould and as a barrier against aggressive agents. After that, the backfill is used to improve mechanical stability, to help against the formation of an advective transport pathway in the near field, and to reduce the voids in the system. Finally, the gallery will be closed by seals, which can represent an additional physical barrier (ONDRAF 2013).

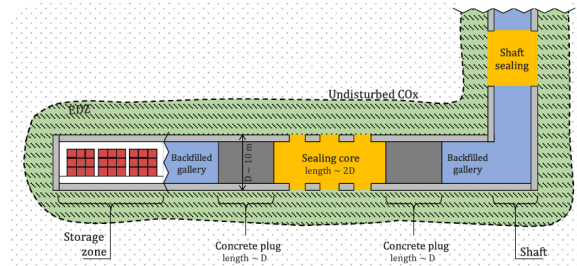
1.3 French concept

In France, the institution responsible for the management of radioactive waste is the ANDRA. In their concept, called the Cigéo project, the natural barrier consists of a Callovo Oxfordian (COx) claystone. As a deep argillaceous formation, it presents several properties that are favourable to the context of radioactive waste storage, as stated by ANDRA 2005: low permeability, mechanical behaviour that limits disturbance in the formation, and stable chemistry of the medium. The CIGEO URL is located in Bure (France) and is where the final geological repository will be located. The ANDRA established several scenarios in order to estimate the volume of high level waste that will be produced by the French nuclear fleet, and they arrived at a total volume between $11,800m^3$ and $20,100m^3$ based on the considered scenario for the 60-year lifetime of their 57 reactors (ANDRA 2026).

On the other hand, the engineered barrier is shown in Figure 1.8. Here, the waste containers are placed in galleries that are backfilled and sealed. The waste packages are placed in non-alloyed



(a) Seal designs – (a) horizontal tunnels core confined between two concrete abutments (b) horizontal tunnels core in contact with backfill, (c) in a ramp, (d) in a shaft (Sellin et al. 2025)



(b) Conceptual scheme of Cigéo sealing concept (M. Alonso et al. 2024)

Figure 1.8: Conceptual schemes of the French concept (M. Alonso et al. 2024, Sellin et al. 2025)

steel casings that are closed with a welded lid. Regarding the sealing, bentonite and concrete plugs are used. Bentonite is used for its low permeability, in order to limit potential flows in the gallery and its ability to swell, which allows it to close the gaps caused by the construction of the gallery and to support the surrounding rock. Indeed, the excavation causes some variations in the stress field around the gallery, in a zone called the excavation-induced damaged zone (EDZ) (M. Alonso et al. 2024). To ensure a good contact between the host rock and the seal, the concrete lining is partially removed around the sealing core, only at localised areas, as can be seen in Figure 1.8b. Then, the concrete plugs confine the bentonite core and add a layer

of physical protection. Next, the backfill aims to mitigate the expansion of the fractured zone once rupture of the lining has occurred. The backfill consists of disaggregated and recompacted excavated COx claystone. Moreover, an objective of the backfill and seal is to allow the dissipation of gas pressure. To do so, a mixture of bentonite and sand is used, enabling a better control of the gas transfer capacity, water permeability, and thermal conductivity (ANDRA 2005, Sellin et al. 2025, Li et al. 2025, M. Villar et al. 2024).

1.4 Sealing material

The main difference between the two concepts originates from the fact that Boom clay presents a more ductile mechanical behaviour than COx claystone. Indeed, the self-sealing properties of Boom clay mean that the fractures will heal over time. Accordingly, in the Belgian concept, the use of bentonite based materials as sealing materials is less beneficial, which partially explains why concrete based materials are preferred. As this work focuses on the modelling of bentonite-based seals, emphasis is placed on the French concept in the following chapters.

In terms of construction, the gallery of the Cigéo project is set to undergo four different phases, as shown in Figure 1.9. First, the gallery is excavated. Once the excavation is completed, the

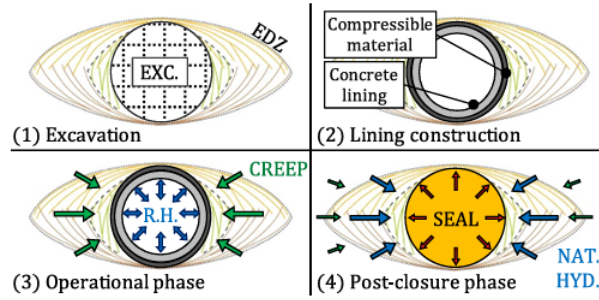


Figure 1.9: Lifetime phases of the gallery (M. Alonso et al. 2024)

lining is constructed. Then, the third phase consists of the operation of the gallery, whose duration is estimated to last about 100 years. Finally, during the post-closure phase, the sealing material is placed and portions of the lining are removed to ensure proper contact between the rock and the seal. After this phase, the sealing material progressively hydrates, which induces its swelling. It will thus apply pressure on the host rock and on the concrete plug, causing compression in the EDZ and in the backfill (M. Alonso et al. 2024). Concerning the form of the bentonite based seal, several options are possible, as illustrated in Figure 1.10. The first and preferred option is a powder-pellets mixture, as it allows for easier placement and reduction of gaps (M. Alonso et al. 2024, Hoffmann et al. 2007). Another possibility is to use bentonite bricks (ANDRA 2005).

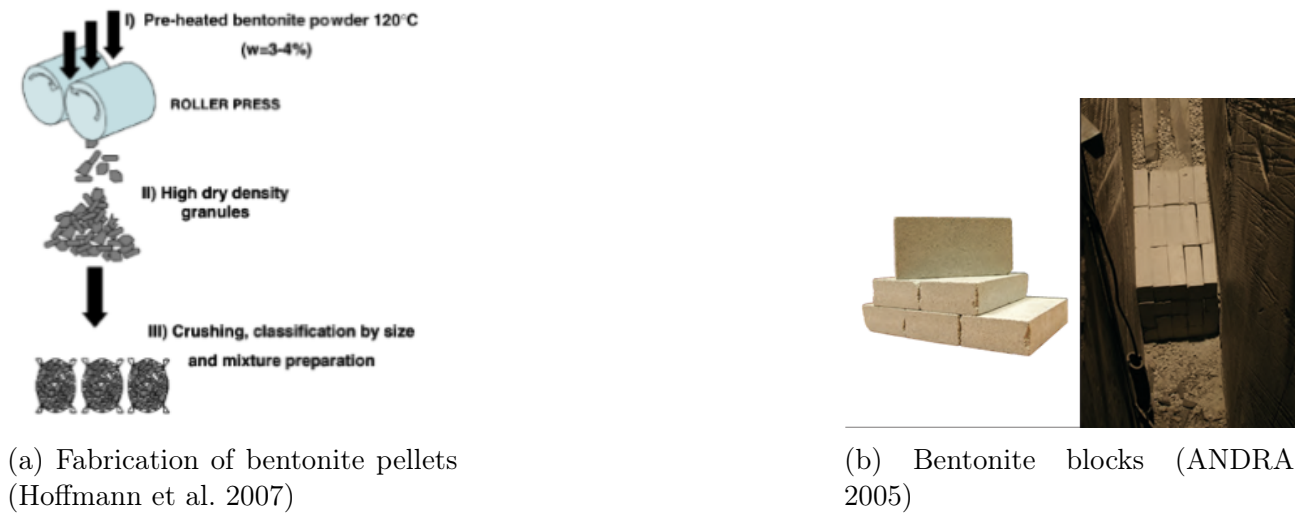


Figure 1.10: Possible forms of bentonite

1.5 Objectives and content of this work

In this work, the objective is to study the behaviour of a sealing structure for high level waste, in the context of the French concept. In particular, the emphasis will be placed on the sealing bentonite based material, due to its complex swelling behaviour upon hydration. Indeed, during its swelling, bentonite undergoes coupled hydro-mechanical phenomena. Hence, the calculations will be performed numerically, using the finite element code **LAGAMINE**, which was developed by the university of Liège.

To do so, Chapter 2 will go through the existing literature in order to better understand the different materials of the geological disposal system. Generalities about the materials, typical values of their parameters, models for the numerical simulation, and existing numerical works will be presented.

Next, Chapter 3 will be devoted to the study of a small scale model, focusing solely on a bentonite sample in order to better understand the mechanical and hydraulic behaviour of this material. A first calibration of parameters for bentonite will be proposed, as well as a sensitivity analysis of the model's mechanical parameters.

Then, Chapter 4 will focus on models whose scale is closer to the actual situation. The behaviour of bentonite will be studied under different configurations: when it is the only material present in the sealing structure, when a second material, namely backfill, is added in the sealing structure in order to bring material heterogeneity, and with different water inlets in order to simulate fractures in the surrounding rock. Other features will be added, such the introduction of friction between the bentonite core and the deposition walls.

Finally, Chapter 5 will study the whole sealing structure, with the addition of the concrete liner, the EDZ, and the surrounding rock. In the latter, the different phases of the lifetime of the gallery will also be modelled.

Chapter 2

Literature review

This chapter consists of a review of the existing literature, in order to present the different materials of both the natural and engineered barrier systems. In particular, general information, typical parameters or behaviour, and appropriate models for the numerical simulation will be described for each of them.

2.1 Natural barrier material

The natural barrier material consists of the host rock.

2.1.1 Callovo-Oxfordian argillite

In the French concept, Callovo-Oxfordian (COx) argillite is studied as the host rock. Research is conducted at the Bure laboratory (France), with the main gallery at a depth of about 490m. In the following sections, the geological context, the mechanical and hydraulic parameters, and the impact of the excavation will be presented.

2.1.1.1 Geological context

The laboratory of Bure is located in the Paris basin, which is filled with Mesozoic and Cenozoic sediments. Figure 2.1 shows the geological zone where the laboratory is located. In particu-

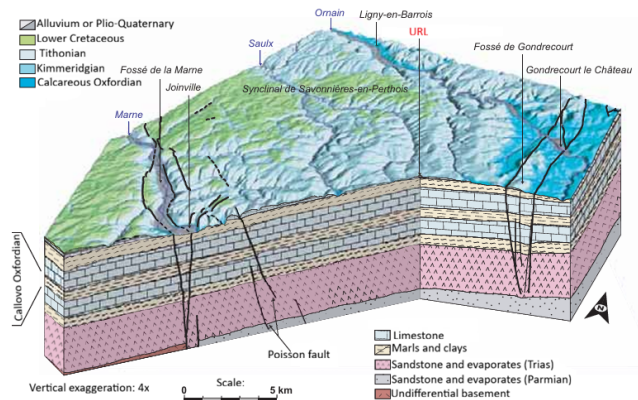


Figure 2.1: Geological zone of the URL (ANDRA 2005)

lar, Callovian (165.3-161.5 Ma) and Oxfordian (161.5-154.8 Ma) stages belong to the Jurassic system, which is part of the Mesozoic (Cohen et al. 2025). The argillite is composed of mixed-layered illite and smectite minerals dominated by illite (45-50%), kaolinite, and minor amounts

of chlorite. Moreover, the carbonate cements, of marine origin, contribute to the low permeability of the rock (Distinguin et al. 2007). The layer of argillite in which the gallery is located has a thickness of about 130m (Gaucher et al. 2004).

2.1.1.2 Mechanical parameters

Typical values for the mechanical parameters of Callovo-Oxfordian argillite are presented in Table 2.1, from different sources.

Table 2.1: Mechanical parameters of Callovo-Oxfordian argillite (Charlier et al. 2013, Armand et al. 2014), Zhang et al. 2019, Conil et al. 2018

<i>Mechanical parameter</i>		<i>Value</i>
Dry density	ρ_d	2.21 – 2.33 g/cm ³
Density	ρ	2.30 – 2.40 g/cm ³
Porosity	ϕ	15 – 18 %
Water content	w	3 – 7.4 %
Young’s modulus	E	4000 – 5600 MPa
Poisson’s coefficient	ν	0.3
Friction angle	φ	20 – 25°
Dilation angle	ψ	0°
Cohesion	c	3 – 7 MPa
Uniaxial compression strength	σ_c	20 – 33.5 MPa
Biot’s coefficient	b	0.6

2.1.1.3 Hydraulic parameters

To define the hydraulic behaviour, a first relationship to determine is the water retention curve, which links the degree of saturation to the suction (also called capillary pressure), defined as $s = u_g - u_w$, where u_g and u_w are the gas and water pressures. In particular, the water retention curve is further detailed in the later sections that describe the models used in this work, while this section only presents typical water retention curves and hydraulic parameters of COx argillite.

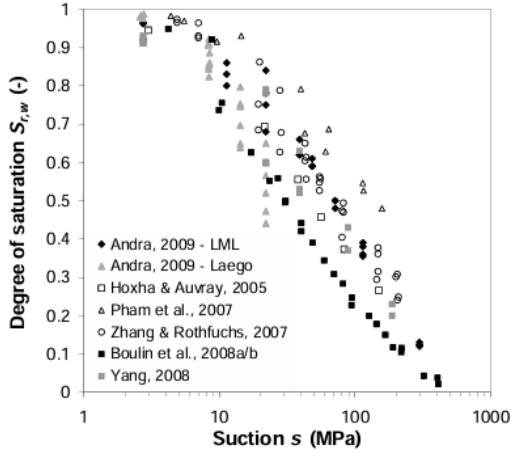
Charlier et al. 2013 summarised some of the experimental data available in Figure 2.2.

Then, the water and gas permeabilities of Callovo-Oxfordian argillite evolve based on the degree of saturation. To characterise these evolutions, Charlier et al. 2013 summarised the data from different sources in Figures 2.3a and 2.3b.

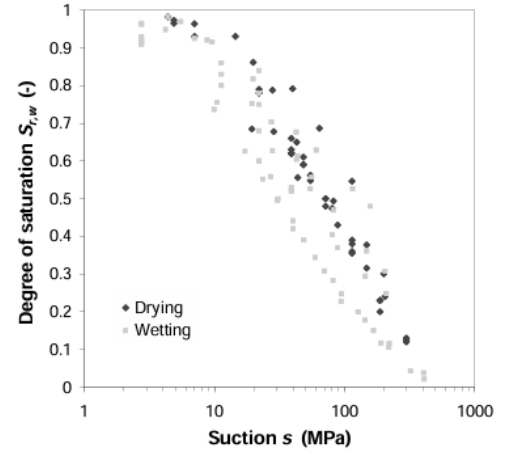
To model COx claystone, typical hydraulic parameters are presented in Table 2.2.

2.1.1.4 Excavation Damaged Zone

During the excavation of the gallery, the local stress field undergoes important changes. The excavation works damage the surrounding rocks, causing irreversible changes to properties such as flow and transport characteristics, while also inducing cracks and potential fractures. This damaged zone, called the Excavation Damaged Zone (EDZ), therefore represents a preferential pathway for fluids (M. Alonso et al. 2024, Corman 2023, Pardoën et al. 2016). For instance, as shown in Figure 2.4, the permeability increases by several orders of magnitude when the

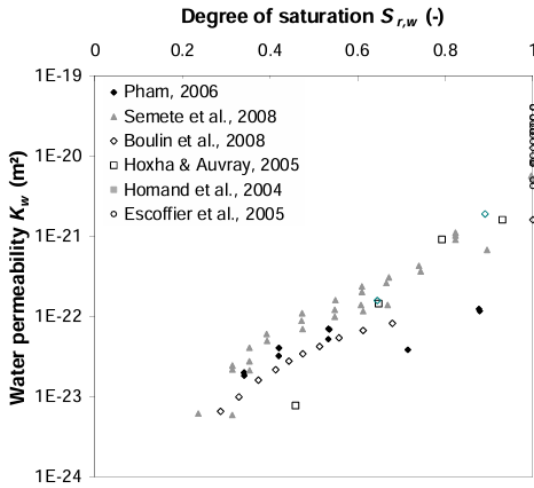


(a) Classified by authors

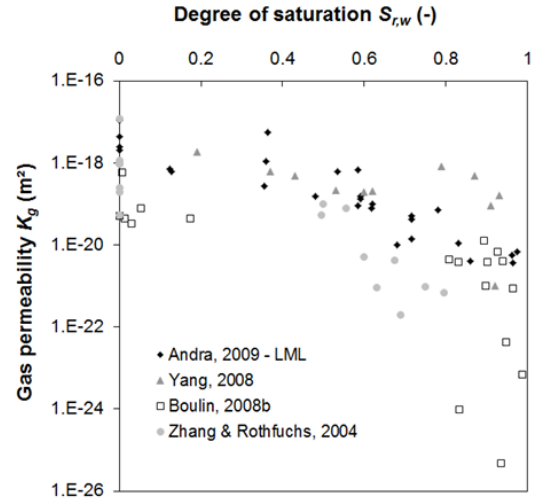


(b) Classified by the drying or wetting path

Figure 2.2: Experimental data for the retention curve of Callovo-Oxfordian argillite (Charlier et al. 2013)



(a) Water permeability



(b) Gas permeability

Figure 2.3: Experimental data for the water and gas permeability evolution with the degree of saturation for Callovo Oxfordian argillite (Charlier et al. 2013)

Table 2.2: Hydraulic parameters for the Callovo-Oxfordian argillite (Charlier et al. 2013)

$K_{w,hor}^{sat}$	Horizontal water permeability (m ²)	4×10^{-20}
$K_{w,vert}^{sat}$	Vertical water permeability (m ²)	1.33×10^{-20}
τ	Tortuosity (-)	0.25
P_r	van Genuchten air entry pressure (MPa)	15
n	van Genuchten parameter (-)	1.49
m	van Genuchten parameter (-)	0.32886

measurement points are closer ($< 1m$) to the gallery wall.

Of course, because of the presence of damages in the EDZ, some of the mechanical and hydraulic parameters of the COx claystone are affected. Typically, the permeability is multiplied by a factor 10^2 , the Young's modulus is divided by 10, and the air entry pressure drops from

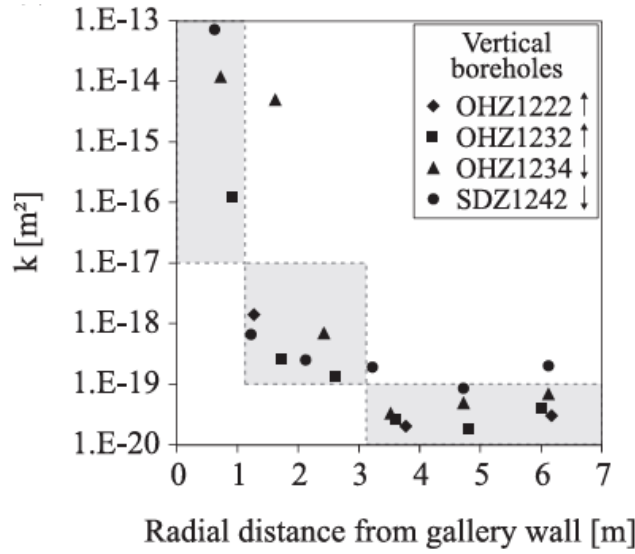


Figure 2.4: Evolution of permeability with the distance from gallery wall (Pardoen et al. 2016)

15MPa to 5MPa (Corman et al. 2025a).

2.2 Engineered barrier materials

The engineered barrier materials consist of the bentonite based seal, the backfill, and the concrete.

2.2.1 Bentonite based seal

In the following sections, the general characteristics, the mineralogical composition, and the behaviour of bentonite will be covered.

2.2.1.1 Generalities

Bentonite typically comes from the sedimentation of volcanic ashes or tuff. Nowadays, the term bentonite is used for clay rocks whose main minerals are from the smectite group. In particular, montmorillonite is the main species present in bentonite (María Victoria Villar 2004). Additionally, bentonite also contains different minerals such as quartz, feldspar, micas and amphiboles, among others.

The structure of clay minerals consists of different arrangements of tetrahedral sheets and octahedral sheets. The tetrahedral sheet is made of several SiO_4^{4-} or AlO_4^{5-} tetrahedra that are linked together by sharing three of their four oxygen ions. The octahedral sheet is made of several aluminium, potassium or magnesium octahedra linked together by sharing all their six oxygen ions or hydroxyls. In particular, smectite is composed of several 2:1 layers, meaning that one octahedral sheet is placed between two tetrahedral sheets in each layer. In the case of montmorillonite, the layers are not linked by hydrogen bonds, meaning that water can penetrate between the layers (Delage et al. 2001). Figure 2.5 shows the example of montmorillonite, whose sheet has a thickness of 1nm (Delage et al. 2001).

Moreover, the surface of each layer has a negative electrical charge, and for this reason, exchange cations usually occupy the space between two layers. Those cations are brought by

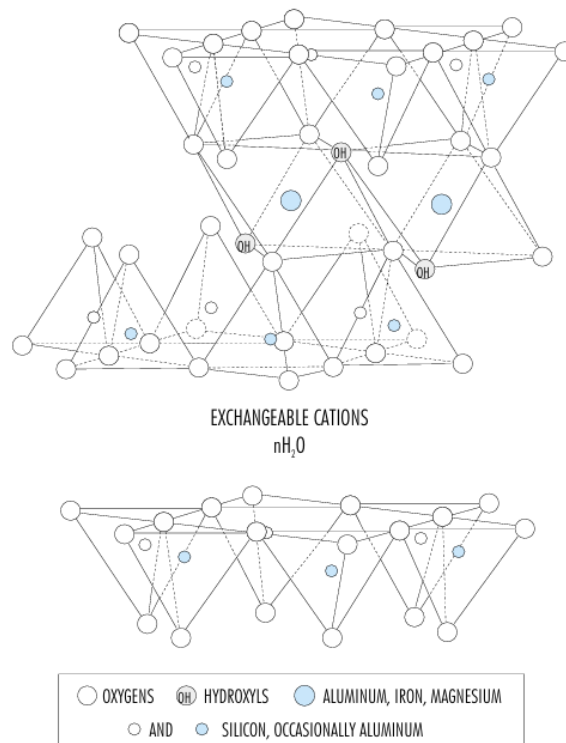


Figure 2.5: Structure of montmorillonite (María Victoria Villar 2000)

water. This causes an increase in the height of the crystalline network (María Victoria Villar 2000). The cation exchange capacity (CEC) measures the number of cations before reaching electrical equilibrium.

Furthermore, the interaction between the water and the negative charges of the clay results in the creation of a zone where the electrical potential varies, called the diffuse double layer, or the diffuse ion layer (DL) (María Victoria Villar 2000). Indeed, the number of cations decreases when the distance from the charged surface of the clay particle is greater, meaning that the electric potential also decreases. This creates a diffuse ion layer for each clay particle. When two particles are close to each other, their DL repels each other, which impacts the swelling of the clay (Meunier 2003).

In terms of porosity, clay materials present three different kinds of porosity (María Victoria Villar 2000), as shown in Figure 2.6. First, the interlaminar porosity, which corresponds to the porosity between the 2:1 layers, also called laminae. The stored water is part of the structure. It can also be noted that, in Figure 2.6, a range of sizes is specified for each type of porosity. However, this size can vary depending on the document considered. Then, the intra-aggregate porosity, also called microporosity, corresponds to the porosity inside the aggregates, between the laminae. In microporosity, water is retained by adsorption. Finally, the inter-aggregate porosity, also called macroporosity, corresponds to the porosity between the aggregates, where water is retained by capillarity (Wypych et al. 2022, Meunier 2003, María Victoria Villar 2000).

In addition, the pore size distribution of bentonite typically presents a bimodal distribution (Wang et al. 2013), as shown in Figure 2.7. The first mode corresponds to the microporosity, while the second mode corresponds to the macroporosity. As the pore size distribution was determined with mercury intrusion porosimetry tests, the interlaminar porosity is not detectable (Wypych et al. 2022, Meunier 2003, María Victoria Villar 2000).

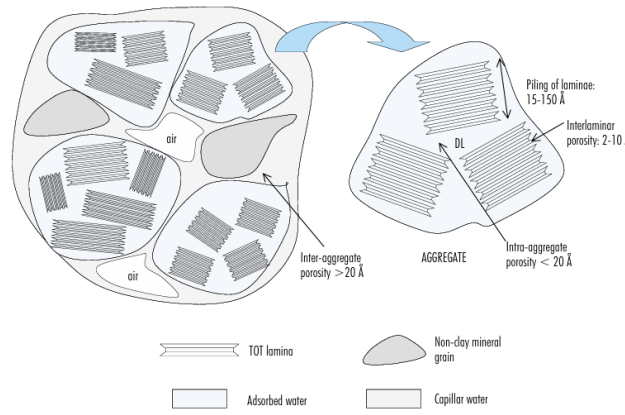


Figure 2.6: Schematic diagram of the organisation of a clay material (María Victoria Villar 2000)

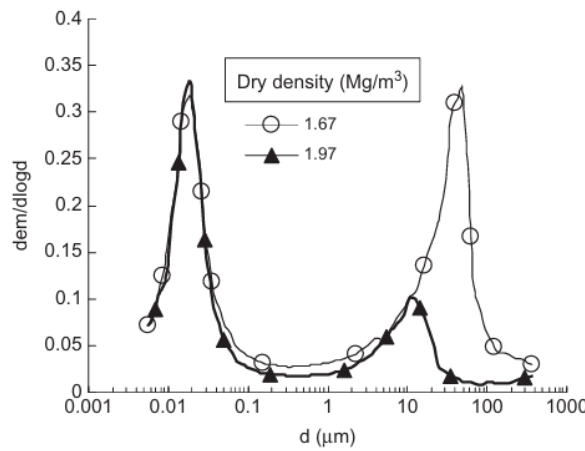


Figure 2.7: Pore size distribution of bentonite/sand mixture compacted to different dry density with a water content of about 11.0% (Wang et al. 2013)

2.2.1.2 Composition of bentonite

In the French concept, among all the possible kinds, pure MX-80 bentonite and a mixture of MX-80 bentonite and sand are considered for the bentonite based seal.

MX-80 bentonite comes from Wyoming, USA. Its properties typically follow the ranges presented in Table 2.3

The sand used in the considered mixture is a quartz sand, extracted in France. The unit mass of this sand is $2650 \frac{kg}{m^3}$, and its grain size distribution is close to the one of MX-80 bentonite, as shown in Figure 2.8. The proposed mixture can vary in terms of proportion; for instance Wang et al. 2013 studied a mixture composed of 70% of MX-80 bentonite and 30% of sand, but other possibilities are studied, up to 40% of MX-80 and 60% of sand.

2.2.1.3 Behaviour of bentonite

To describe bentonite, the same relation as for the natural materials can already be presented, to better understand its behaviour. However, due to its swelling behaviour, these relations are

Mineralogical composition	
Montmorillonite	65-90 %
Quartz	2.8-15.2 %
Feldspars	2-9 %
Cation exchange capacity and cations	
CEC	75-88 $\frac{meq}{100g}$
Na^+	61-67 $\frac{meq}{100g}$
Ca^{2+}	7-10 $\frac{meq}{100g}$
Mg^{2+}	3-5 $\frac{meq}{100g}$
Specific surface	512-800 $\frac{m^2}{g}$
Other properties	
$<2\mu m$ fraction	73-90 %
Liquid limit	520-575 %
Plastic limit	42-53 %
Plasticity index	478
Specific density ρ_s	2760-2820 $\frac{kg}{m^3}$

Table 2.3: Properties of MX-80 bentonite (M. V. Villar 2007, Madsen 1998, Tang et al. 2008, Wang et al. 2013, Gatabin et al. 2016)

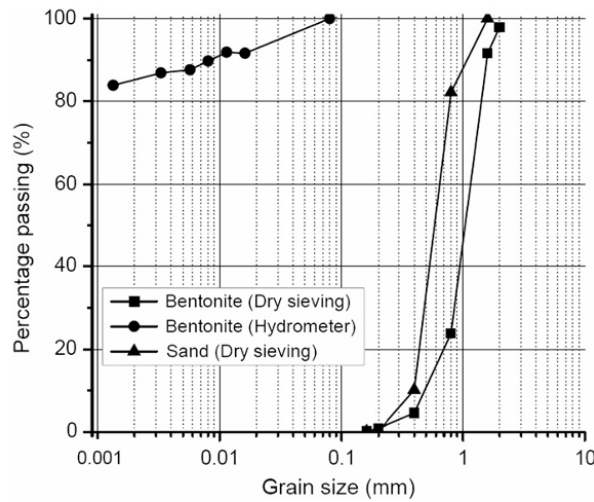


Figure 2.8: Grain size distribution MX-80 bentonite and sand (Wang et al. 2013)

more complex in the case of bentonite.

• Water retention behaviour

First, it is important to characterise the water retention behaviour of bentonite. Indeed, under repository conditions, bentonite will undergo a wetting path. A relationship between suction, defined as the difference between gas and water pressure, and water content can be determined. Seiphoori et al. 2014 determined the water retention curve of MX-80 for three void ratios, as shown in Figure 2.9. As it can be seen, an increase in the void ratio leads to an increase in the maximum water content. Furthermore, the higher value of water content for the same value of suction during drying as during wetting highlights the hysteresis behaviour of bentonite (Seiphoori et al. 2014).

Then, they investigated the water retention behaviour under wetting and drying cycles for the

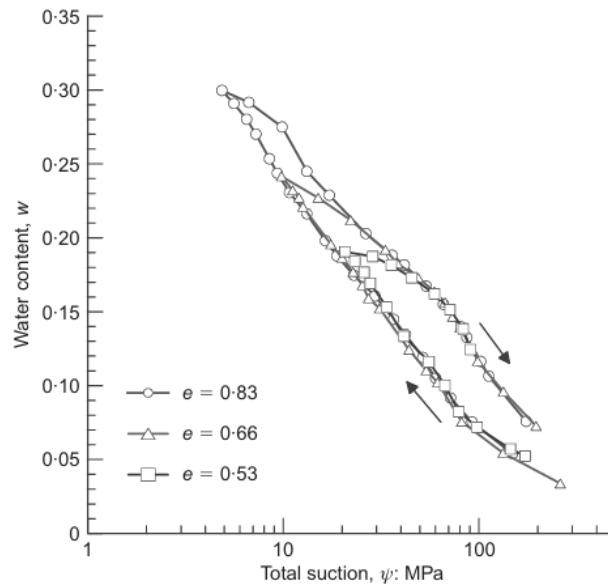


Figure 2.9: Effect of the void ratio on the water retention behaviour of MX-80 bentonite (Seiphoori et al. 2014)

sample with a void ratio of $e = 0.53$, as shown in Figure 2.10. The sample was wetted until full

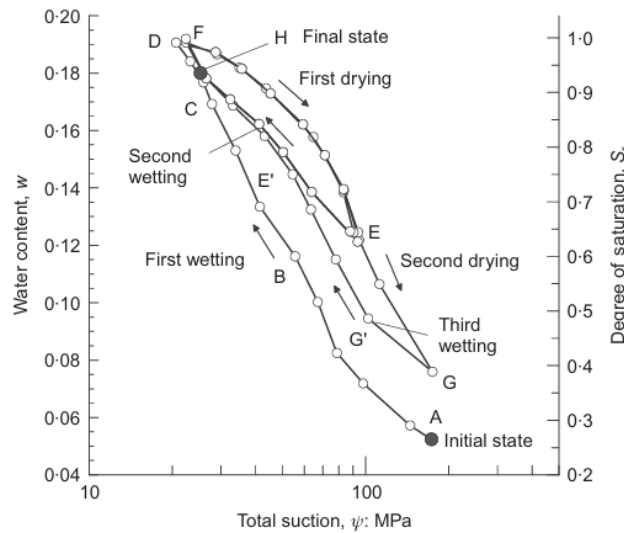


Figure 2.10: Water retention behaviour of highly compacted MX-80 granular bentonite during cyclic variation of the total suction (Seiphoori et al. 2014)

saturation three times (A to D, E to F, and G to H) and dried two times (D to E and F to G). Seiphoori et al. 2014 highlighted that the second and third wetting paths do not follow the first one, because the latter caused irreversible changes. As a result, during the second wetting, the water content is higher for the same suction compared to the first wetting.

Finally, the impact of sand on the water retention curve is shown in Figure 2.11. It can be seen that pure bentonite retains more water at low suction levels than the mixture, while the results at high suction values are similar.

• Hydraulic conductivity

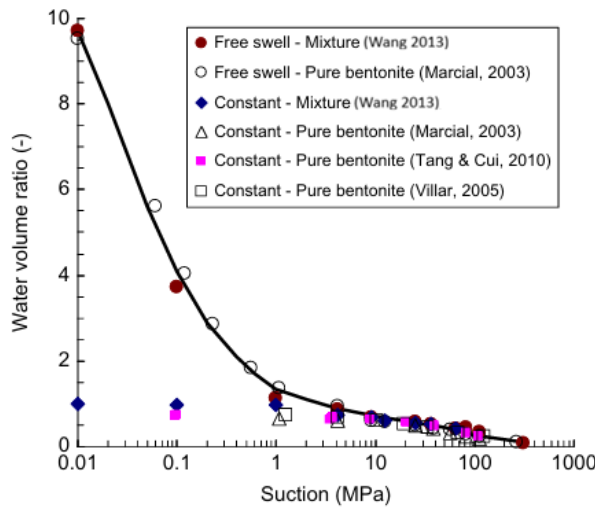


Figure 2.11: Water retention curves of bentonite-sand mixture and pure bentonite (Wang et al. 2013)

Upon saturation, the hydraulic properties of bentonite will also be modified. Indeed, as shown in Figure 2.12, the intrinsic permeability decreases when dry density increases.

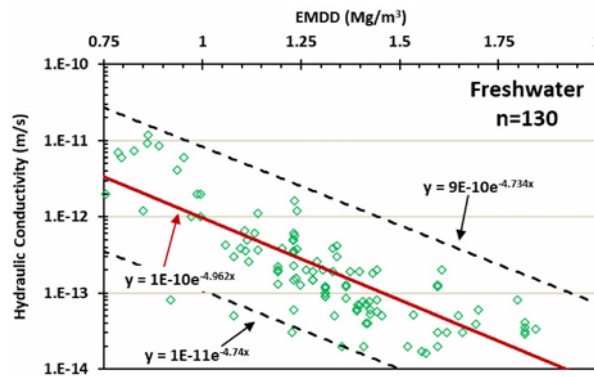


Figure 2.12: Evolution of hydraulic conductivity with the effective montmorillonite dry density (EMDD) (Dixon et al. 2023)

It is also possible to plot the evolution of intrinsic permeability with porosity (Figure 2.13), as done by María Victoria Villar 2000. As higher porosity means lower dry density, it confirms that hydraulic conductivity and intrinsic permeability evolve similarly with dry density. Then, upon saturation, the permeability of the bentonite buffer will increase.

• Mechanical behaviour

An important aspect of the behaviour of bentonite is the development of swelling pressure, which starts when bentonite is prevented from swelling. First of all, the swelling pressure evolves exponentially with dry density, as shown in Figure 2.14 for five different types of bentonite. Furthermore, dry density of bentonite also varies upon hydration. Indeed, at saturation, the final dry density is lower than the initial dry density, leading to a lower swelling pressure (M. V. Villar 2005).

On the other hand, M. V. Villar et al. 2008 showed that the initial water content had no influence on the swelling pressure, as can be seen in Figure 2.15, where three dry densities are

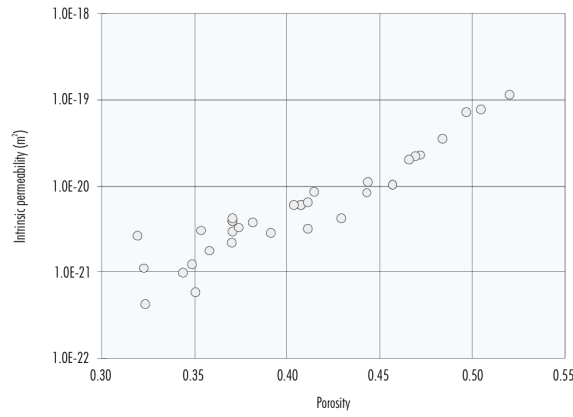


Figure 2.13: Variation in intrinsic permeability with porosity from saturated hydraulic conductivity tests (María Victoria Villar 2000)

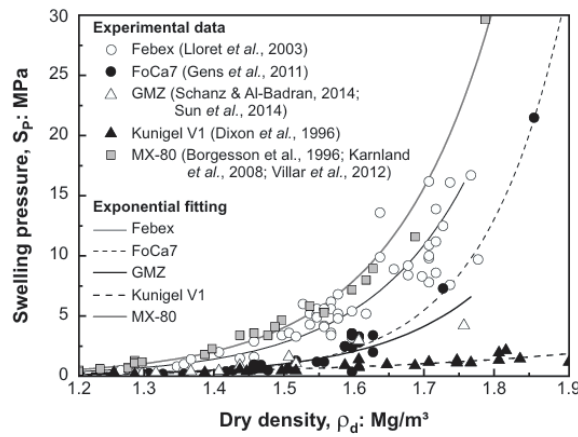


Figure 2.14: Evolution of the swelling pressure with dry density (Dieudonné 2016)

studied (1.4, 1.5, and 1.6 $\frac{Mg}{m^3}$). Indeed, as highlighted by M. V. Villar et al. 2008, the swelling

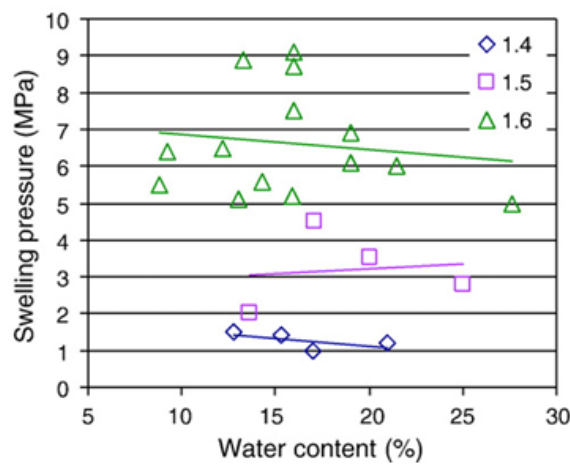


Figure 2.15: Influence of initial water content on the swelling pressure (M. V. Villar et al. 2008)

capacity is inversely related to the initial water content but only for vertical pressures that are lower than the swelling pressure value. For the latter, the influence of initial water content is negligible.

Moreover, the addition of sand impacts the compression behaviour of the material. Indeed, as shown in Figure 2.16, the yield stress for the same void ratio is higher in the case of the mixture than for pure bentonite, meaning that the mechanical strength is improved if the mixture of sand and bentonite is used (Wang et al. 2013).

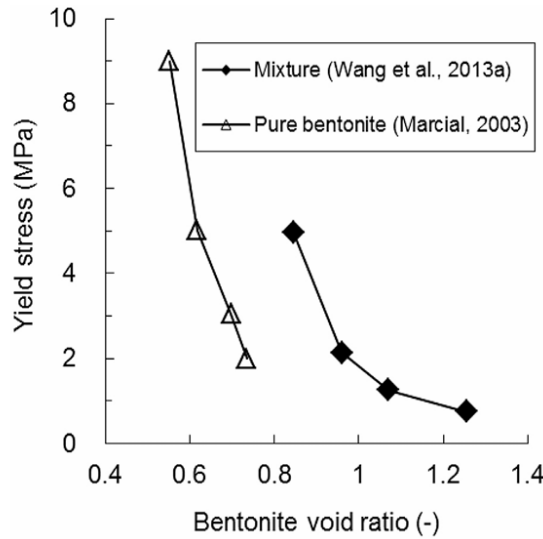


Figure 2.16: Evolution of the yield stress with bentonite void ratio for two mixtures (Wang et al. 2013)

2.2.2 COx-Sand backfill

To backfill the gallery, a mixture of reprocessed COx argillite and sand is considered. In particular, two options were studied by the ANDRA and simulated in Corman et al. 2025a, as illustrated in Figure 2.17: firstly, a homogeneous configuration, consisting only of a mixture of 30% COx and 70% sand. Secondly, a two layer configuration, with the higher third of the gallery filled with a mixture of 30% COx and 70% sand, and the two lower thirds filled with a mixture of 70% COx and 30% sand. The objective of this second configuration is to have a higher permeability at the top of the gallery than at the bottom, in order to allow for a path that will dissipate the gas overpressure.

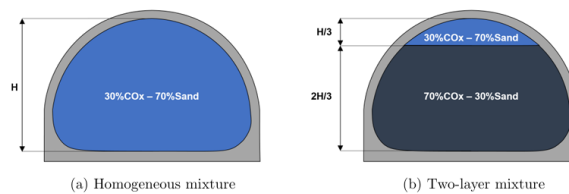


Figure 2.17: Backfill configurations

Typical parameter values for the two configurations are summarised in Table 2.4, from Corman et al. 2025a:

2.2.3 Concrete

In the French geological repository system, concrete is used to support the host rock, to confine the swelling material and to limit its deformations. It ensures the stability of the structure

Table 2.4: Mechanical and hydraulic parameters of backfills (Corman et al. 2025a)

<i>Parameters</i>		30% COx and 70% sand	30% sand and 70% COx
Young's modulus	E [MPa]	60	60
Poisson's coefficient	ν [-]	0.3	0.3
Density	ρ [$\frac{g}{cm^3}$]	2.65	2.65
Porosity	ϕ [-]	0.35/0.25	0.25
Tortuosity	τ [-]	0.25	0.25
Permeability	k [m^2]	$2 \cdot 10^{-16}$	$2 \cdot 10^{-16}$
Entry pressure	Pr [MPa]	0.07	0.25

(M. Alonso et al. 2024). As a porous material, concrete has a similar behaviour to that of the host rock. Accordingly, its model is similar to that of the EDZ or the host rock. In particular, in this work, a classical elasto-plastic model can be used. Typical mechanical and hydraulic parameters for the concrete of the French concept are presented in Table 2.5.

Table 2.5: Mechanical and hydraulic parameters of concrete (Corman et al. 2025a, Charlier et al. 2013)

<i>Parameters</i>		<i>Value</i>
Young's modulus	E [MPa]	40000
Poisson's coefficient	ν [-]	0.25 – 0.3
Density	ρ [$\frac{g}{cm^3}$]	2.3 – 2.65
Porosity	ϕ [-]	0.15 – 0.2
Tortuosity	τ [-]	0.25
Permeability	k [m^2]	$10^{-18} - 10^{-17}$
Entry pressure	Pr [MPa]	10

2.3 Numerical models

The models described in this section, which will be used later in the numerical simulations, are similar and based on those presented by Corman et al. 2025b and Dieudonné 2016.

To model the different materials of the numerical simulation, the finite element code **LAGAMINE** is employed. Thus, the balance equations of the finite element model are first presented. Then, it is necessary to express the variables of the problem in terms of the main unknowns, namely the displacements, the water pressure, and the gas pressure. To do so, three constitutive equations are needed. Furthermore, the materials of the natural and engineered barriers being quite different, they each need their own constitutive equations.

Before entering in the balance equations, a general framework can first be presented. Indeed, each material can be considered as an unsaturated porous medium. In particular, the mixture theory is adopted: the solid phase, which consists of grains, is filled by a liquid and a gas phase. In terms of species, the liquid phase consists of liquid water and dissolved gas, while the gas phase consists of water vapour and dry gas. It is assumed that the mineral species and the solid phase coincide, that solid and fluid phases are immiscible, that the gas phase is a perfect mixture of dry air and water vapour, and that the dissolved air does not affect the liquid phase, meaning that the liquid and water pressures are equal (Corman et al. 2025b). In the following, the solid, liquid and gas phases are respectively denoted by the subscripts s , l and g , while liquid water and water vapour are denoted by the subscripts w and v .

2.3.1 Balance equations

Firstly, balance equations for the material must be determined. The variables that describe the problem are the displacement field $\mathbf{u}$, the liquid pressure field $u_l (=u_w)$, and the gas pressure field u_g . Four balance equations are needed: the balance equations for each species, namely the solid mass and the fluids (water and gas species) mass, and the balance of momentum equation.

2.3.1.1 Solid mass balance equation

The mineral is assumed to be the only solid phase, whose mass balance equation is given by:

$$\frac{\partial}{\partial t} (\rho_s(1 - \phi)\Omega) = 0 \quad (2.1)$$

where Ω is the volume of the mixture, ρ_s is the density of solids, and $\phi = \frac{\Omega_v}{\Omega}$ is the porosity.

2.3.1.2 Fluids mass balance equation

The mass balance equations for water and air are respectively given by:

$$\dot{m}_l^w + \nabla \cdot \mathbf{f}_l^w + \dot{m}_g^w + \nabla \cdot \mathbf{f}_g^w = Q^w \quad (2.2)$$

$$\dot{m}_g^a + \nabla \cdot \mathbf{f}_g^a + \dot{m}_l^a + \nabla \cdot \mathbf{f}_l^a = Q^a \quad (2.3)$$

where m_l^w , m_g^w , m_g^a and m_l^a are the masses of liquid water, water vapour, dry air and dissolved air inside the volume Ω . $\mathbf{f}_l^w$, $\mathbf{f}_g^w$, $\mathbf{f}_g^a$ and $\mathbf{f}_l^a$ are the mass fluxes of liquid water, water vapour, dry air and dissolved air. Q_w and Q_a are the water and air source terms. The masses are calculated with:

$$\dot{m}_l^w = \frac{\partial}{\partial t} (\rho_w S_r \phi \Omega) \quad (2.4)$$

$$\dot{m}_g^w = \frac{\partial}{\partial t} (\rho_v (1 - S_r) \phi \Omega) \quad (2.5)$$

$$\dot{m}_g^a = \frac{\partial}{\partial t} (\rho_a (1 - S_r) \phi \Omega) \quad (2.6)$$

$$\dot{m}_l^a = \frac{\partial}{\partial t} (\rho_l^a S_r \phi \Omega) \quad (2.7)$$

where S_r is the degree of saturation, ρ_w , ρ_v , ρ_a and ρ_l^a are the densities of liquid water, water vapour, dry air, and dissolved air. If the volume Ω is taken equal to 1, Equations 2.2 and 2.3 become:

$$\frac{\partial}{\partial t} (\rho_w S_r \phi) + \nabla \cdot \mathbf{f}_l^w + \frac{\partial}{\partial t} [\rho_v (1 - S_r) \phi] + \nabla \cdot \mathbf{f}_g^w = Q^w \quad (2.8)$$

$$\frac{\partial}{\partial t} [\rho_a (1 - S_r) \phi] + \nabla \cdot \mathbf{f}_g^a + \frac{\partial}{\partial t} (\rho_l^a S_r \phi) + \nabla \cdot \mathbf{f}_l^a = Q^a \quad (2.9)$$

2.3.1.3 Momentum balance equation

The last balance equation is the equation for the balance of momentum. For quasi static loading:

$$\nabla \cdot \boldsymbol{\sigma}_t + \mathbf{b} = \mathbf{0} \quad (2.10)$$

where $\boldsymbol{\sigma}_t$ is the total Cauchy stress field and $\mathbf{b}$ is the body force vector. If there is no other force than gravity, then $\mathbf{b} = \rho \mathbf{g}$ where $\mathbf{g}$ is the vector of the acceleration of gravity and ρ is expressed as:

$$\rho = \rho_s(1 - \phi) + \rho_w S_r \phi + \rho_g(1 - S_r) \phi \quad (2.11)$$

2.3.2 Constitutive equations of the natural barrier material and the concrete

Starting with the natural barrier material and the concrete, three models are needed: a water retention model, a mechanical model, and a hydraulic model.

2.3.2.1 Water retention model

To describe the water retention properties, a van Genuchten 1980 model is used. It reads:

$$S_{rw} = S_{r_{res}} + (S_{max} - S_{r_{res}}) \left(1 + \left(\frac{s}{P_r} \right)^{\mathcal{N}} \right)^{\frac{1}{\mathcal{N}} - 1} \quad (2.12)$$

where S_{rw} is the water degree of saturation, P_r is a parameter corresponding to the gas entry pressure, S_{max} and $S_{r_{res}}$ are the maximum and residual degrees of water saturation, $\mathcal{N}$ is a parameter that controls the shape of the curve, and $s = u_g - u_w$ is the suction, also called capillary pressure.

Moreover, as the degree of saturation impacts the permeabilities, two equations are needed for the water (k_{rw}) and gas (k_{rg}) relative permeability (Corman et al. 2025b). Respectively, they are modelled by the Mualem-van Genuchten's model and by a cubic law:

$$k_{rw} = \sqrt{S_{rw}} \left(1 - \left(1 - S_{rw}^{\frac{1}{\mathcal{M}}} \right)^{\mathcal{M}} \right)^2 \quad (2.13)$$

$$k_{rg} = \mathcal{A}(1 - S_{rw})^{\mathcal{L}} \quad (2.14)$$

where $\mathcal{A}$, $\mathcal{M} = 1 - \frac{1}{\mathcal{N}}$, and $\mathcal{L}$ are model parameters.

2.3.2.2 Mechanical model

To describe the mechanical behaviour of the natural barrier components, it is necessary to take into account the presence of fluids within the pore space (Corman et al. 2025b). It is done by using Bishop's definition of effective stress:

$$\sigma_{ij} = \sigma'_{ij} + b_{ij} (S_{rw} u_w + (1 - S_{rw}) u_g) \delta_{ij} \quad (2.15)$$

where δ_{ij} is the Kronecker symbol, σ'_{ij} is the Bishop's effective stress, and b_{ij} is Biot's tensor, which represents the compressibility of the solid grain skeleton relative to the skeleton compressibility.

Next, the variation of solid density is coupled to the variation of pore water pressure, gas pressure, and mean effective stress. Its expression is:

$$\dot{\rho}_s = \rho_s \frac{(b_{ij} - \phi) (S_{rw} \dot{u}_w + S_{rg} \dot{u}_g) + \dot{\sigma}'_{ij}}{(1 - \phi) K_s} \quad (2.16)$$

Then, regarding the material behaviour, an isotropic linear elastic-perfectly plastic model with a Van Eekelen yield surface is adopted. It reads:

$$f \equiv II_{\hat{\sigma}} - m \left(I_{\sigma'} + \frac{3c}{\tan \varphi} \right) = 0 \quad (2.17)$$

where $\hat{\sigma}'_{ij}$ is the deviatoric part of the effective stress tensor, $I_{\sigma'} = \sigma'_{ij} \delta_{ij} = \sigma'_{ii}$ is the first invariant of stress, $II_{\hat{\sigma}} = \sqrt{0.5 \hat{\sigma}'_{ij} \hat{\sigma}'_{ij}}$ is the second invariant of stress, m is a parameter of the yield surface, c is the cohesion, and φ is the friction angle.

2.3.2.3 Hydraulic model

The last constitutive equation is the model for the fluxes of the different phases.

First, the fluxes of liquid water, water vapour, dry gas, and dissolved gas are respectively calculated as follows:

$$\mathbf{f}_l^w = \rho_w \mathbf{q}_l \quad (2.18)$$

$$\mathbf{f}_g^w = \rho_v \mathbf{q}_g + \mathbf{i}_g^w \quad (2.19)$$

$$\mathbf{f}_g^a = \rho_a \mathbf{q}_g + \mathbf{i}_g^a \quad (2.20)$$

$$\mathbf{f}_a^w = \rho_l^a \mathbf{q}_l + \mathbf{i}_l^a \quad (2.21)$$

where $\mathbf{q}_l$ and $\mathbf{q}_g$ are the advective fluxes of the liquid and gas phases, $\mathbf{i}_g^w$, $\mathbf{i}_g^a$ and $\mathbf{i}_l^a$ are the diffusive fluxes of water vapour, dry air, and dissolved air.

First, the advective fluxes are expressed thanks to the generalisation of the Darcy's law for partially saturated soils:

$$\mathbf{q}_\alpha = -\frac{k_\alpha}{\mu_\alpha} (\nabla u_\alpha + \rho_\alpha \mathbf{g}) \quad (2.22)$$

where $\alpha = w$ or g , depending on whether the flux is liquid or gaseous, $\mathbf{q}$ is the volumetric flux, k is the effective permeability of the partially saturated medium, μ is the dynamic viscosity, ∇u is the gradient of the pressure, ρ is the density of the phase, and $\mathbf{g}$ is the vector of gravity acceleration. The effective permeability of each phase is calculated using:

$$k_\alpha = K_\alpha k_{r\alpha}(S_r) \quad (2.23)$$

where K_α is the phase permeability for saturated conditions if $\alpha = w$ and for dry conditions if $\alpha = g$, $k_{r\alpha}$ is the phase relative permeability which depends on the degree of saturation. K_w is calculated using an extension of the Kozeny-Carman law:

$$K_w = K_{w0} \frac{\phi_M^N}{(1 - \phi_M)^M} \frac{(1 - \phi_{M0})^M}{\phi_{M0}^N} \quad (2.24)$$

where K_{w0} is a reference permeability of a sample with a reference macroporosity ϕ_{M0} and M and N are parameters. For k_{rw} and k_{rg} , equations 2.13 and 2.14 are used.

Then, the diffusive fluxes are expressed thanks to Fick's law:

$$\mathbf{i}_g^w = -D_g^{w*} \rho_g \nabla \left(\frac{\rho_v}{\rho_g} \right) = -\mathbf{i}_g^a \quad (2.25)$$

$$\mathbf{i}_l^a = -D_l^{a*} \rho_w \nabla \left(\frac{\rho_l^a}{\rho_w} \right) \quad (2.26)$$

where D_g^{w*} and D_l^{a*} are the effective diffusion coefficients of water vapour and dissolved air. It reads, as expressed by Philip et al. 1957

$$D_g^{w*} = \phi (1 - S_r) \tau D_g^w \quad (2.27)$$

$$D_l^{a*} = \phi S_r \tau D_l^a \quad (2.28)$$

where τ is the tortuosity, D_g^w and D_l^a are the diffusion coefficients of water vapour and dissolved air. While D_l^a is equal to $5.03 \times 10^{-9} \frac{m^2}{s}$, the expression for D_g^w , from Philip et al. 1957, is:

$$D_g^w = 5.893 \times 10^{-6} \left(\frac{T^{2.3}}{u_g} \right) \quad (2.29)$$

if the gas mixture consists of water vapour and dry air and

$$D_g^w = D_{g0}^w \left(\frac{T}{T_0} \right)^{1.75} \left(\frac{u_{g0}}{u_g} \right) \quad (2.30)$$

if the gas mixture consists of water vapour and any other gas. D_{g0}^w is the diffusion coefficient of the gas mixture for $T_0 = 273.15K$ and $u_{g0} = 0.1MPa$, and T is the temperature.

Additionally, the liquid water density ρ_w varies with the pressure of the water:

$$\rho_w = \rho_{w0} \left(1 + \frac{u_w - u_{w0}}{\chi_w} \right) \quad (2.31)$$

where ρ_{w0} is the liquid water density at the reference water pressure u_{w0} , and χ_w is the water compressibility, equal to $2 \times 10^9 Pa$ at $20^\circ C$.

The gas phase consists of dry air and water vapour. Its density is found thanks to Dalton's law:

$$\rho_g = \rho_a + \rho_v \quad (2.32)$$

where

$$\rho_a = \frac{M_a u_a}{RT} \quad (2.33)$$

$$\rho_v = \frac{M_w u_v}{RT} \quad (2.34)$$

where M_a and M_w are the molecular mass of dry air and water, respectively equal to $0.029 \frac{kg}{mol}$ and $0.018 \frac{kg}{mol}$.

2.3.3 Constitutive equations of the engineered barrier materials

Similarly to the natural barrier, three constitutive models are needed for the engineered barrier. As the concrete can be modelled as the natural barrier material, the constitutive equations of this section apply to the bentonite based seal and the backfill.

2.3.3.1 Water retention model

For the backfill, a van Genuchten equation (equation 2.12), already presented for the natural barrier materials, is used.

To determine the water retention behaviour of bentonite, a relationship between suction and the void ratio or between suction and the degree of saturation must be determined. Moreover, as bentonite present a pronounced swelling upon wetting, the model should take into account that the void ratio, and thus the dry density, evolve during wetting. This is why the model proposed by Dieudonné 2016 will be used. This model is expressed in terms of water ratio e_w , with:

$$e_w = e_{wM} + e_{wm} \quad (2.35)$$

where e_{wM} is the water ratio in the macropores and e_{wm} is the water ratio in the micropores.

• Water ratio in micropores

To obtain an expression for e_{wm} , Dieudonné 2016 used Dubinin's isotherm:

$$\Omega_{wm} = \Omega_m \exp \left(- \left[\frac{RT}{\beta_D E_0} \ln \left(\frac{u_v}{u_v^0} \right) \right]^{n_{ads}} \right) \quad (2.36)$$

with Ω_{wm} the volume of water adsorbed in the micropores at temperature T and relative pressure $\frac{u_v}{u_v^0}$, the universal gas constant $R = 8.314 \frac{J}{mol \cdot K}$, the volume of micropores Ω_m , n_{ads} which is a parameter called the heterogeneity factor, controlling the curvature of the water retention curve in the high suction range, β_D is the similarity constant, and $E = \beta_D E_0$ is the characteristic adsorption energy for the given system with E_0 the characteristic energy of adsorption for a reference vapour for which $\beta_D = 1$.

Assuming thermodynamic equilibrium between the phases of the species, Kelvin's law allows to develop the term $\frac{u_v}{u_v^0}$:

$$\frac{u_v}{u_v^0} = RH = \exp \left(\frac{-s M_w}{RT \rho_w} \right) \quad (2.37)$$

with RH the relative humidity, $M_w = 0.018 \frac{kg}{mol}$ the molecular mass of water and ρ_w the density of water.

By using equation 2.37, it is possible to simplify equation 2.36 to:

$$e_{wm} = e_m \exp \left(- (C_{ads} s)^{n_{ads}} \right) \quad (2.38)$$

where $C_{ads} = \frac{M_w}{\rho_w \beta_D E_0}$ is a parameter related to the rate of desaturation of the soil.

• Water ratio in macropores

The model used for macropores is the Van Genuchten model (Genuchten 1980). In terms of water ratio, it reads:

$$e_w = e \left(1 + \left(\frac{s}{\alpha} \right)^{\mathcal{N}} \right)^{-\mathcal{M}} \quad (2.39)$$

where $\mathcal{M}$ and $\mathcal{N}$ are material parameters, and α is related to the air-entry suction s_{AE} . The air-entry suction is the suction corresponding to saturation of the sample.

For the macropores only, it becomes:

$$e_{wM} = (e - e_m) \left(1 + \left(\frac{s}{\alpha} \right)^{\mathcal{N}} \right)^{-\mathcal{M}} \quad (2.40)$$

where

$$\alpha = \frac{A}{e - e_m} \quad (2.41)$$

with A is a parameter that controls the dependence of the air-entry pressure on the macrostructural void ratio.

• Microstructure evolution

To take into account the evolution of e_m during wetting, Dieudonné 2016 proposed the following model:

$$e_m = e_{m0} + \beta_0 e_w + \beta_1 e_w^2 \quad (2.42)$$

where e_{m0} is the microstructural void ratio for the dry material, and β_0 and β_1 are parameters quantifying the swelling potential of the aggregates. Moreover, Dieudonné 2016 imposed a limit on this model: the microstructural void ratio cannot exceed the void ratio, so whenever the model predicts that $e_m > e$, e_m must be taken equal to e . Dieudonné 2016 calibrated the parameters against experimental data for a MX-80 bentonite and a mixture of MX-80 and sand:

Table 2.6: Calibrated parameters of the microstructure evolution model (Dieudonné 2016)

Material	e_{m0}	β_0	β_1
MX-80	0.31	0.10	0.48
MX-80/sand mixture (70/30)	0.29	0.10	0.18

2.3.3.2 Mechanical model

In unsaturated porous media, the independent variables approach can be used. Indeed, to describe partially saturated soils, two independent variables among the tensor of net stress $\boldsymbol{\sigma} = \boldsymbol{\sigma}_t - u_g$, the tensor of effective stress $\boldsymbol{\sigma}' = \boldsymbol{\sigma}_t - u_w$, and the suction $s = u_g - u_w$ are enough. In particular, an extension of the Barcelona basic model (BBM) is used in this work. The BBM was first presented by E. E. Alonso et al. 1990, and the extension explained hereafter was proposed by Dieudonné 2016. In this model, the two independent variables are the net stress and the suction.

•Isotropic stress states

In isotropic stress states, $\sigma_1 = \sigma_2 = \sigma_3$. The variation of volumetric strain $d\epsilon_v$ is calculated by adding together an elastic component and a plastic component:

$$d\epsilon_v = d\epsilon_v^e + d\epsilon_v^p \quad (2.43)$$

with $d\epsilon_v^e$ the elastic component and $d\epsilon_v^p$ the plastic component

First, the elastic component can be divided into a term accounting for the changes in mean net stress $d\epsilon_{vp}^e$ and a term accounting for the changes in suction $d\epsilon_{vs}^e$. Indeed, variation in suction also contributes to the changes in volumetric strain as, under wetting, a reversible swelling is observable. The first term, $d\epsilon_{vp}^e$, is calculated as follow:

$$d\epsilon_{vp}^e = \frac{\kappa}{1+e} \frac{dp}{p} \quad (2.44)$$

where κ is the elastic stiffness parameter for changes in net mean stress, which defines the slope of the unloading-reloading curve (Figure 2.18), and e is the void ratio. This equation can be simplified by using the bulk modulus $K = p \frac{(1+e)}{\kappa}$.

After that, the second term, $d\epsilon_{vs}^e$, is:

$$d\epsilon_{vs}^e = \frac{\kappa_s}{1+e} \frac{ds}{s + u_{atm}} \quad (2.45)$$

where u_{atm} is the atmospheric pressure, and κ_s is the slope of the reversible wetting-drying line for changes in suction.

In total, the elastic strain component:

$$d\epsilon_v^e = d\epsilon_{vp}^e + d\epsilon_{vs}^e = \frac{\kappa}{1+e} \frac{dp}{p} + \frac{\kappa_s}{1+e} \frac{ds}{s + u_{atm}} = \frac{dp}{K} + \frac{ds}{K_s} \quad (2.46)$$

with $K_s = \frac{(1+e)(s+u_{atm})}{\kappa_s}$, the bulk modulus for changes in suction.

Then, the plastic component can be calculated. Plastic strains are generated once the preconsolidation pressure p_0 is reached, as can be seen in Figure 2.18, where saturation is reached. Under saturated conditions, the evolution of $d\epsilon_v^p$ reads:

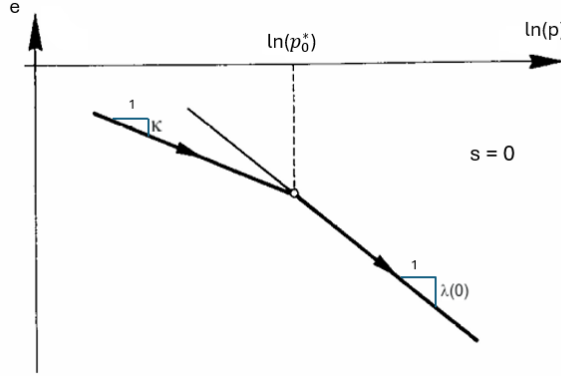


Figure 2.18: Compression curve for saturated condition (adapted from E. E. Alonso et al. 1990)

$$d\epsilon_v^p = \frac{\lambda(0) - \kappa}{1+e} \frac{dp_0^*}{p_0^*} \quad (2.47)$$

where p_0^* is the preconsolidation stress corresponding to the yield limit for the saturated state and λ_0 is the slope of the virgin consolidation line.

However, under unsaturated conditions, suction has an impact on the compression curve of the soil. Indeed, as shown in Figure 2.19, an increase in suction yields an increase in the preconsolidation pressure p_0 and a decrease in the slope of the consolidation line $\lambda(s)$.

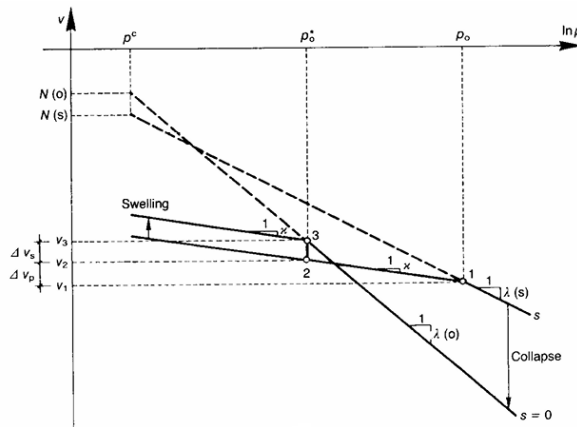


Figure 2.19: Compression curves for saturated and unsaturated states (E. E. Alonso et al. 1990)

In particular, the slope of the consolidation line evolves as:

$$\lambda(s) = \lambda(0) ((1-r)\exp(-\omega s) + r) \quad (2.48)$$

where r is a model parameter accounting for the maximum stiffness of the soil (in the case of infinite suction), and ω is another parameter that accounts for the rate of increase in soil stiffness with suction.

Then, the preconsolidation stress p_0 , and thus the yield limit, evolves with suction as:

$$p_0(s) = p_c \left(\frac{p_0^*}{p_c} \right)^{\frac{\lambda(0) - \kappa}{\lambda(s) - \kappa}} \quad (2.49)$$

where p_c is a reference net pressure. This equation defines a curve that limits the elastic domain and is called the loading-collapse (LC) curve.

Furthermore, variations in suction also lead to additional strain increments in the plastic domain. Indeed, after a threshold suction $s = s_0$, irreversible strains are produced. This threshold defines a second yield curve, the suction increase (SI) curve. The LC and SI curves are represented in Figure 2.20.

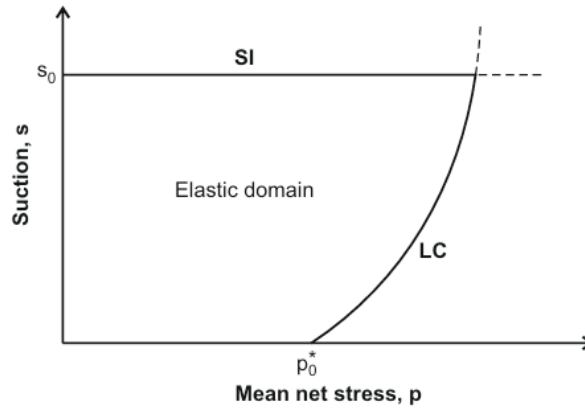


Figure 2.20: Yield curves of the Barcelona Basic Model for isotropic stress states (Dieudonné 2016)

Then, if the mean net stress exceeds the LC curve, the plastic strain increment is calculated as:

$$d\epsilon_v^p = \frac{\lambda(s) - \kappa}{1 + e} \frac{dp_0}{p_0} \quad (2.50)$$

On the other hand, if suction exceeds the SI curve, the plastic strain increment is calculated with:

$$d\epsilon_v^p = \frac{\lambda_s - \kappa_s}{1 + e} \frac{ds_0}{s_0 + u_{atm}} \quad (2.51)$$

• Triaxial stress states

Under triaxial stress states, $\sigma_1 \neq \sigma_2 = \sigma_3$. The three variables used to describe the stress states are the mean net stress, suction, and deviatoric stress ($q = \sigma_1 - \sigma_3$). The latter is added in order to account for shear stresses (E. E. Alonso et al. 1990).

The contribution of changes in deviatoric stress is taken into account in the elastic domain by calculating $d\epsilon_d^e$, the elastic increment of deviatoric/shear strain:

$$d\epsilon_d^e = \frac{Gdq}{3} \quad (2.52)$$

where G , the shear modulus, is expressed as:

$$G = \frac{3(1 - 2\nu)K}{2(1 + \nu)} \quad (2.53)$$

with ν , the Poisson's ratio.

If the suction is fixed, the yield surface, represented in Figure 2.21a, is given by an ellipse whose equation is:

$$q^2 - M_\theta^2(p + p_s)(p_0 - p) = 0 \quad (2.54)$$

where M_θ is the slope of the critical state line and p_s is the left intersection between the yield limit and the p axis.

The expression of M_θ depends on φ , the friction angle:

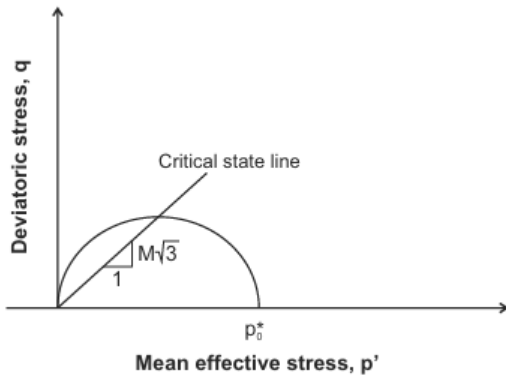
$$M_\theta = \frac{6\sin(\varphi)}{3 - \sin(\varphi)} \quad (2.55)$$

To account for changes in apparent cohesion with suction, p_s is expressed as:

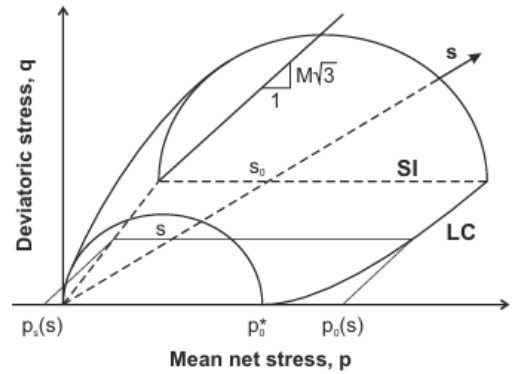
$$p_s(s) = \frac{c(0) + ks}{\tan(\varphi)} \quad (2.56)$$

where $c(0)$ is the cohesion under saturated conditions and k is a parameter that controls the increase in cohesion.

To represent the yield surface in three dimensions (Figure 2.21b), the SI curve is simply extended parallel to the q axis.



(a) Modified Cam-Clay model



(b) Barcelona Basic Model

Figure 2.21: Representation of the Modified Cam-Clay model and the Barcelona Basic Model in the (p - q) plane (Dieudonné 2016)

To determine the plastic strain increments, a non-associated flow rule is proposed. It reads:

$$\frac{d\varepsilon_d^p}{d\varepsilon_v^p} = \frac{2q\alpha}{M_\theta^2(2p + p_s - p_0)} \quad (2.57)$$

where α is a parameter chosen so the flow rule predicts zero lateral strain for stress states corresponding to Jaky's K_0 values, that is to say:

$$\alpha = \frac{M_\theta(M_\theta - 9)(M_\theta - 3)}{9(6 - M_\theta)} \frac{1}{1 - \frac{\kappa}{\lambda(0)}} \quad (2.58)$$

•Extension of the BBM

As highlighted by Dieudonné 2016, a drawback of the model is the overestimation of the swelling pressure of the materials. This is why the Barcelona research group modified the expressions of the slopes of the unloading-reloading for changes in mean net stress (κ) and for changes in suction (κ_s), which become:

$$\kappa(s) = \kappa_0 \left[1 - \alpha_1 s - \alpha_2 \ln \left(\frac{s + u_{atm}}{u_{atm}} \right) \right] \quad (2.59)$$

$$\kappa_s(p) = \kappa_{s,0} \left[1 + \alpha_p \ln \left(\frac{p}{p_{ref}} \right) \right] \exp(\alpha_s s) \quad (2.60)$$

where κ_0 is the elastic stiffness in saturated conditions, α_1 and α_2 are model parameters, $\kappa_{s,0}$, α_p and α_s are model parameters and p_{ref} is the reference pressure.

Furthermore, Dieudonné 2016 highlighted that bentonite is saturated before reaching $s = 0$, because of its important air entry value and its capacity not to desaturate under high suction values. In the BBM, the suction is taken into account to know if swelling strains need to be generated but not the saturation. This is why Dieudonné 2016 proposed the introduction of a threshold suction s^* , which is translated into a threshold degree of saturation S_r^* for which $\kappa_s = 0$ if $S_r > S_r^*$.

Nevertheless, in this work, κ will be considered constant, and another formulation is adopted for κ_s , still with the objective of limiting the overestimation of the swelling pressure:

$$\kappa_s = \kappa_{s0} \exp(-\alpha_p p) \quad (2.61)$$

where α_p controls the dependency of κ_s on the stress level. Moreover, κ_s depends on the evolution of dry density, which leads to the two following equations:

$$\alpha_p = \alpha_{p,ref} \exp(-M_{\alpha_p}(\rho_d - \rho_{d,ref})) \quad (2.62)$$

where $\alpha_{p,ref}$ is equal to the initial α_p , $\rho_{d,ref}$ is equal to the initial ρ_d , and M_{α_p} is a model parameter. κ_{s0} reads as:

$$\kappa_{s0} = \kappa_{s0,ref} \left(\frac{\rho_d}{\rho_{d,ref}} \right)^{N_{ks}} \quad (2.63)$$

where $\kappa_{s0,ref}$ is equal to the initial κ_s , and N_{ks} is a model parameter. Unless specified otherwise, in this work, $M_{\alpha_p} = 0.0037$ and $N_{ks} = 3.66$.

2.3.3.3 Hydraulic model

The hydraulic model of the engineered barrier materials is exactly the same as for the natural barrier materials. The presentation of the model is done in Section 2.3.2.3.

2.3.4 Interface

To represent the interface between concrete and EBS materials, the implementation in the numerical model of a contact element with its own mechanical and hydraulic models is needed (Corman et al. 2025b).

2.3.4.1 Mechanical model

Mechanically, the interface is described using Coulomb's yield criterion. As the contact between the two materials is not perfect, a penalty method is used. A relation between the variation of stresses and gap function is assumed, from Cerfontaine et al. 2015:

$$\begin{bmatrix} \dot{p}_N \\ \dot{\tau}_T \end{bmatrix} = \begin{bmatrix} K_N & 0 \\ 0 & K_T \end{bmatrix} \begin{bmatrix} \dot{g}_N \\ \dot{g}_T \end{bmatrix} \quad (2.64)$$

where p_N is the normal contact constraint, related to the interpenetration of the two bodies in the contact zone, and τ_T is the tangential contact constraint, related to the shear behaviour of the interface. K_N and K_T are penalty coefficients, respectively on the contact pressure and on the shear frictional stress. $\dot{g}_N$ and $\dot{g}_T$ are respectively the normal and tangential velocities. Moreover, g_N is also called the gap function, and measures the distance between two bodies (Dieudonné 2016).

If sliding occurs, the maximum shear stress τ_{max} is limited by a Coulomb's criterion:

$$\tau_{max} = \mu p'_N \quad (2.65)$$

with μ the Coulomb's friction coefficient.

2.3.4.2 Hydraulic model

The contact elements are fluid preferential pathways. To calculate the mass flow of liquid water through ($f_{w,N}$) and along ($f_{w,L}$) the interface, the following equations are proposed in Cerfontaine et al. 2015:

$$f_{w,N} = \rho_w T_w \Delta p_w \quad (2.66)$$

$$f_{w,L} = -\rho_w \frac{k_l}{\mu_w} \Delta p_w + \rho_w g \nabla z \quad (2.67)$$

where T_w is the transmissivity.

2.4 Existing numerical works

To study multi-barrier systems, several numerical simulations have already been performed. For instance, M. Alonso et al. 2024 represented the gallery of the French concept using a 3D model, as shown in Figure 2.22. In the latter, BBM was also used to describe the evolution of the bentonite based seal. Among its results, one can cite that the sealing core is completely saturated after around 2,000 years. In terms of swelling pressure, M. Alonso et al. 2024 found that the maximum value remains lower than $4MPa$, as shown in Figure 2.23. In its simulation, the maximum displacement experienced by the sealing is equal to about $0.5m$, at the interface between the sealing core and the plug (Figure 2.24).

Of course, other national concepts have also been studied. For example, the Finnish concept has been analysed, both with 2D models (among others: Pintado et al. 2013, Toprak et al. 2019) and 3D models, such as in Toprak et al. 2024, also with the BBM as the mechanical model. In the latter, depicted in Figure 2.25, Toprak et al. 2024 showed that, for example, the evolution of the swelling pressure or the evolution of the vertical displacement (Figure 2.26) met the performance targets imposed by the Finnish situation, as shown in Figure 2.27. More precisely, Toprak et al. 2024 showed that a maximum swelling pressure of $8.2MPa$ was developed, with a maximum displacement of $12.8cm$.

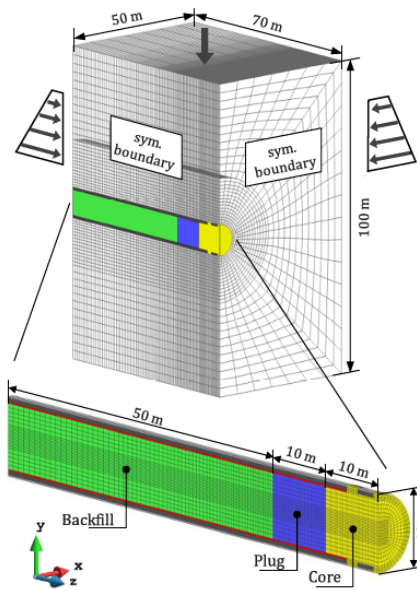


Fig. 5 Model geometry and mesh: general overview

Figure 2.22: Model geometry used by M. Alonso et al. 2024

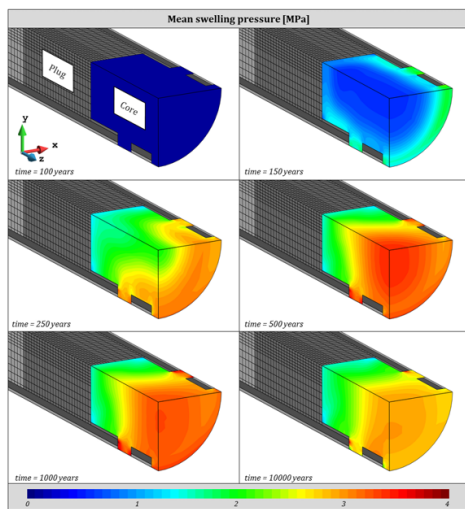


Figure 2.23: Evolution of the mean swelling pressure (M. Alonso et al. 2024)

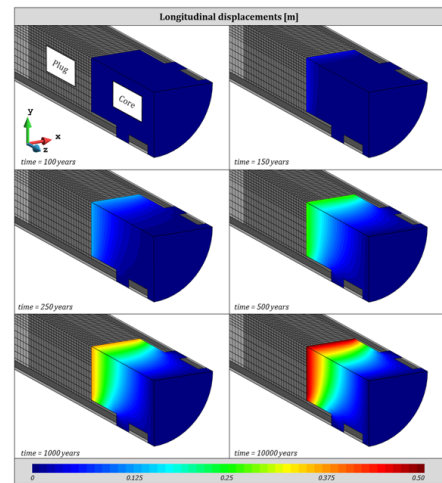


Figure 2.24: Evolution of the longitudinal displacement (M. Alonso et al. 2024)

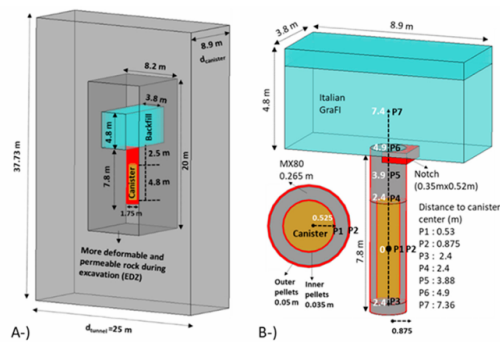


Figure 2.25: Detailed geometry of the model used in Toprak et al. 2024

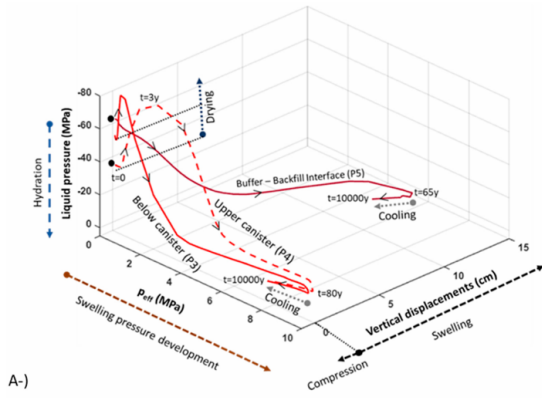


Figure 2.26: Evolution of swelling pressure, liquid pressure, and vertical displacements (Toprak et al. 2024)

Performance target code	Rationale	Performance target (Posiva, 2021)	Base Case response
1.3-BUF-35	Limiting mineral transformation	$T^{min} < 100^\circ\text{C}$	78°C
1.3-BUF-34	Maintain canister integrity	$T^{min} < -2.5^\circ\text{C}$	10.5
1.3-BUF-29	Retardation of radionuclide transportation	$k_{eff} < 10^{-13} \text{ m}^2$	$5.6 \times 10^{-11} \text{ m}^2$
1.3-BAC-22		$k_{backfill} < 10^{-17} \text{ m}^2$	$2.5 \times 10^{-11} \text{ m}^2$
1.3-BUF-32	Maintain canister integrity	$P_{max} \text{ around canister} < 10 \text{ MPa}$	5 MPa
1.3-BUF-17	Avoid canister sinking	$P_{max} \text{ around canister} > 0.2 \text{ MPa}$	5 MPa
1.3-BAC-29	Mechanical stability of deposition tunnels	$P_{max} \text{ in backfill} < 3 \text{ MPa}$	2.5 MPa
1.3-BAC-24	Ensure tight contact with rock	$P_{max} \text{ in backfill} > 0.1 \text{ MPa}$	2.5 MPa
1.4-BUF-16	Ensure tightness and self-sealing	$P_{max} \text{ in buffer} > 1 \text{ MPa}$	7 MPa
1.5-BUF-9	Ensure mechanical protection of canister	$\rho_{bu} \text{ of buffer} < 2050 \text{ kg/m}^3$	2040 kg/m^3
1.5-BUF-14	Prevent colloid transport	$\rho_{bu} \text{ of buffer} > 1,650 \text{ kg/m}^3$	2040 kg/m^3
1.3-BAC-25	Mechanical stability of EBS	Upheaved backfill volume $< 0.7 \text{ m}^3$	0.2 m^3

Figure 2.27: Results and performance targets (Toprak et al. 2024)

2.5 Preliminary conclusion

Through a review of the existing literature, this chapter made it possible to better understand the different materials used in the deep geological repository system. In addition, it also provided a description of the models that will be employed in the later simulations and examples of existing numerical simulations.

In the following, numerical simulations will be performed using exercises from an ongoing European partnership on radioactive waste management (EURAD2), intended to finish in 2029. This partnership brings together EU member states in order to collaborate toward safe radioactive waste management. It involves waste management organisations, technical safety organisations, and research entities (European Union 2025). The objectives of the following simulations will be to study the behaviour of the bentonite based core, such as its swelling behaviour, its stress path, or the development of water pressure for instance.

Chapter 3

Benchmark exercises

3.1 Introduction

To begin with, this chapter aims to introduce the behaviour of bentonite and the impact of some of the mechanical parameters of the Barcelona Basic Model. To do so, a small scale model is used, solely consisting of a bentonite sample. In particular, benchmarking exercises from the ANCHORS work package 10 (Toprak et al. 2025) will be used, in order to better understand the influence of the parameters of the model. The ANCHORS project, which is part of the EURAD2 framework, aims to improve the bentonite based barrier systems and the numerical modelling capabilities. To do so, eleven benchmarking exercises were proposed, exercises for which the different parameters needed for the model were calibrated against experimental data. These parameters will also be employed later for higher scale test cases (Chapter 4). The exercises each focus on the modification of one specific feature of the entire model at a time, in order to better understand the influence of each of these feature. In this section, some of the benchmarking exercises conducted by the ULiège team will be used to test the sensitivity of the results on certain chosen parameters. In particular, the models of this first chapter were developed by Dieudonné 2016.

3.2 Numerical models and their objective

The first exercise aims to calibrate the parameters of the model against experimental data. It consists of a 0D plane-strain geometry, shown in Figure 3.1, with a single element mesh with one isoparametric eight-node finite element. In terms of boundary and initial conditions, an initial water pressure of $-100MPa$ is applied, every node of the boundaries ramps from $-100MPa$ to $0.1MPa$ after 200 days, an initial stress of $0.15MPa$ is applied, and each node of the boundaries is fixed to maintain a constant volume (Toprak et al. 2025).

The second model is a reference configuration. Its objective is to act as a baseline for other models, allowing to highlight variations caused by modifications of specific features of the model. As shown in Figure 3.2, it consists of a 2D axisymmetric geometry, with a 10×28 structured mesh with isoparametric eight-node finite elements. In terms of boundary and initial conditions, an initial water pressure of $-100MPa$ is applied, the pressure at the bottom boundary ramps from $-100MPa$ to $0.1MPa$ after 20 days and is then maintained for 180 days, an initial total stress of $0.15MPa$ is applied, and each node of the boundaries is restrained in the normal direction, to keep a constant volume. The parameters of the model are the same as for the calibration model (Table 3.2) (Toprak et al. 2025).

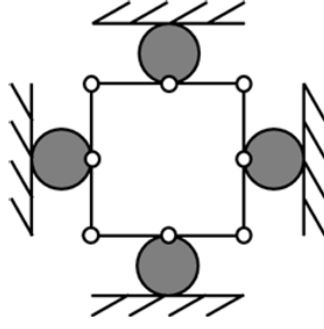


Figure 3.1: Geometry of the model for calibration (Toprak et al. 2025)

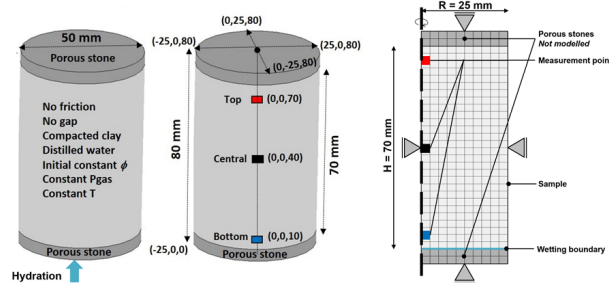


Figure 3.2: Representation of the reference configuration (Toprak et al. 2025)

The third studied model is similar to the reference configuration, except that a technological gap of 2 mm was added on the right side of the sample, as shown in Figure 3.3. It aims to study the impact of gaps on the model. Compared to the reference configuration, the geometry changes on the right side, where a zero-thickness element is added to the outer side of the sample. In terms of boundary and initial conditions, the only difference is that the nodes of the right boundary of the sample are no longer fixed in the x direction. Their displacement is only prevented by the contact element, which is infinitely rigid (Toprak et al. 2025). This model aims to study the impact of interfaces, which will be implemented in the later higher scale geometry.

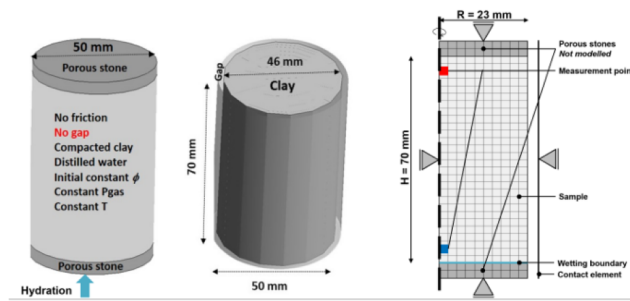


Figure 3.3: Representation of the model for technological gaps

The parameters for the contact element are presented in Table 3.1. Even if Toprak et al. 2025 suggests a friction angle of 8° , this model does not aim to study friction, thus the friction angle is taken equal to 0° in this model.

Table 3.1: Mohr-Coulomb model parameters for the contact element (Toprak et al. 2025)

Parameter	Symbol	Value	Unit
Normal penalty coefficient	K_N	1e12	[N/m ³]
Tangential penalty coefficient	K_T	1e12	[N/m ³]
Friction angle	ϕ	0	[°]
Cohesion	c	0	[Pa]
Max. contact friction	$\tau_{\max}$	1e20	[Pa]

3.3 Calibration

Starting with the exercise represented in Figure 3.1, it can be said that the model parameters have been calibrated against experimental data from Dixon et al. 2023, Dueck et al. 2010, and Pintado et al. 2013. Furthermore, it is possible to calibrate the parameters for different initial dry densities. In this case, an initial dry density of $1600 \frac{kg}{m^3}$ was chosen.

First, the water retention curve is presented in Figure 3.4. It can be seen that the double struc-

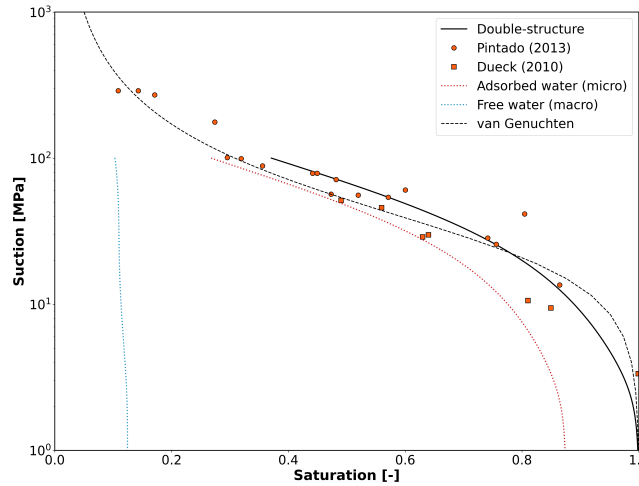


Figure 3.4: Calibration - Water retention curve

ture model, used in this work, meets the experimental data from Pintado et al. 2013. Logically, suction, defined as $s = u_g - u_w$, decreases when the degree of saturation increases because it induces an increase in water pressure.

Then, the evolution of permeability with dry density is depicted in Figure 3.5. It shows that the Kozeny-Carman law seems to be in accordance with the experimental data points, as the permeability of the sample decreases when the dry density increases. Indeed, when the sample is more compacted, its porosity, and thus its permeability, is lower.

After that, the evolution of swelling pressure with dry density is shown in Figure 3.6. As expected, the swelling pressure decreases with dry density.

Finally, the evolution of the mean net stress with suction and the LC curve are shown in Figure 3.7. It shows that the targeted pressure and LC curve are both met when the calibrated parameters are used. Moreover, it can also be seen that the rate of increase in the mean net stress clearly decreases as soon as the stress path encounters the LC curve, corresponding to the transition from elasticity to plasticity. Additionally, the parameters that define the LC curve

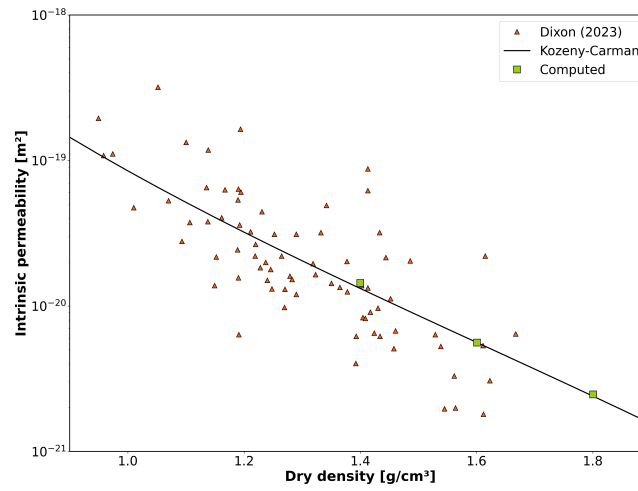


Figure 3.5: Calibration - Evolution of permeability with dry density

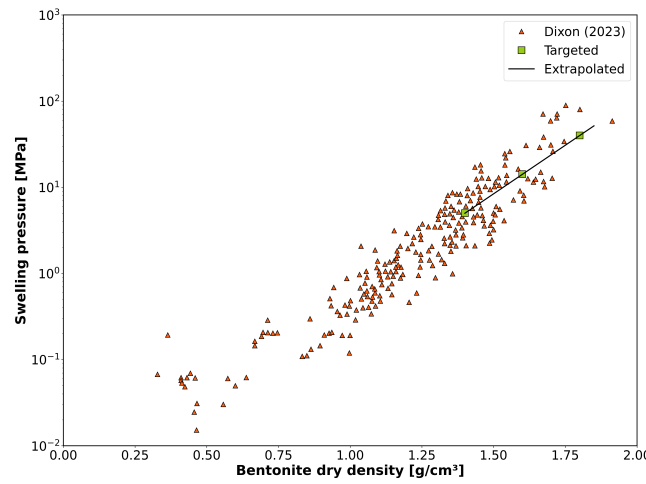


Figure 3.6: Calibration - Evolution of swelling pressure with dry density

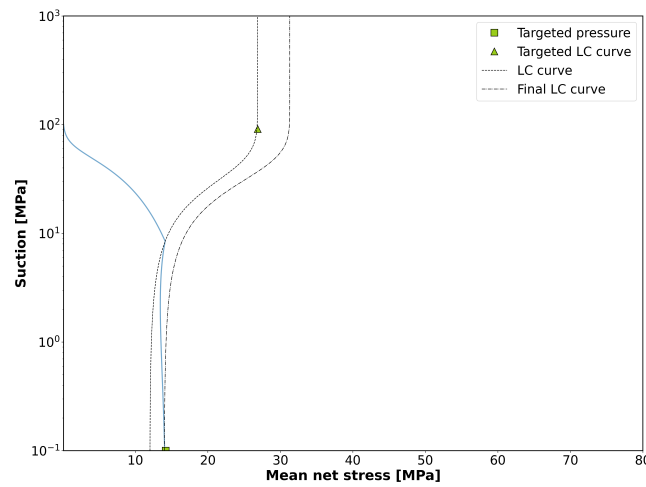


Figure 3.7: Calibration - Evolution of mean net stress with suction

are modified upon hydration. Indeed, hydration impacts the dry density, on which is based the value of the parameters. Thus, the LC curve is modified during hydration explaining why the stress path and the final LC curve both meet at the targeted pressure.

To obtain these graphs the parameters presented in Table 3.2 were used.

Table 3.2: Calibrated HM parameters of MX-80 bentonite (Toprak et al. 2025)

Parameter	Symbol	Value	Unit
Mechanical			
Dry density	ρ_d	1600	$[\frac{kg}{m^3}]$
Porosity	ϕ_0	0.42	[-]
Poisson ratio	ν	0.3	[-]
Elastic compressibility coefficient (mean net stress)	κ	0.040	[-]
Elastic compressibility coefficient (suction)	κ_s	0.28	[-]
Parameter controlling swelling strain	α_p	2.5e-7	$[Pa^{-1}]$
Friction angle	φ	25	$[^\circ]$
Cohesion (saturated)	$c(0)$	0.1	$[MPa]$
Slope of virgin consolidation line	$\lambda(0)$	0.25	[-]
Minimum soil compressibility	r	0.40	[-]
Soil stiffness parameter	ω	0.075	$[MPa^{-1}]$
Suction cohesion parameter	k	0.0046	[-]
Pre-consolidation pressure	p_0^*	12	$[MPa]$
LC curve shape parameter	p_c	8.70	$[MPa]$
Hydraulic			
Retention – van Genuchten			
P_r		27	$[MPa]$
$\mathcal{M}$		0.47	[-]
$\mathcal{N}$		1.9	[-]
Retention – Macrostructure			
Λ		0.2	$[MPa]$
$\mathcal{N}$		0.3	[-]
$\mathcal{M}$		3	[-]
Retention – Microstructure			
C_{ads}		0.75e-8	$[Pa^{-1}]$
n_{ads}		1.5	[-]
e_{m0}		0.31	[-]
β_0		0.48	[-]
β_1		0.1	[-]
Advective Darcy's law			
$k_{ij,0}^{int}$		5.6e-21	$[m^2]$
ϕ_0		0.42	[-]
M		3.0	[-]
N		1.5	[-]
$\phi_{M,0}$		0.11	[-]
$e_{M,0}$		0.189	[-]
Liquid relative permeability			
Λ		1	[-]
$\mathcal{L}$		3.0	[-]
Gas relative permeability			
Λ		100	[-]
$\mathcal{L}$		3.0	[-]

3.4 Reference configuration

Now that the parameters of the models are known, the model described in Figure 3.2 can be studied. To present the result of the different exercises, three measurement points are plotted on each graph. These points are highlighted in Figure 3.2.

First, Figure 3.8 shows the evolution of suction with time and Figure 3.9 presents the evolution of the degree of saturation. It shows that the bottom part of the sample logically saturates

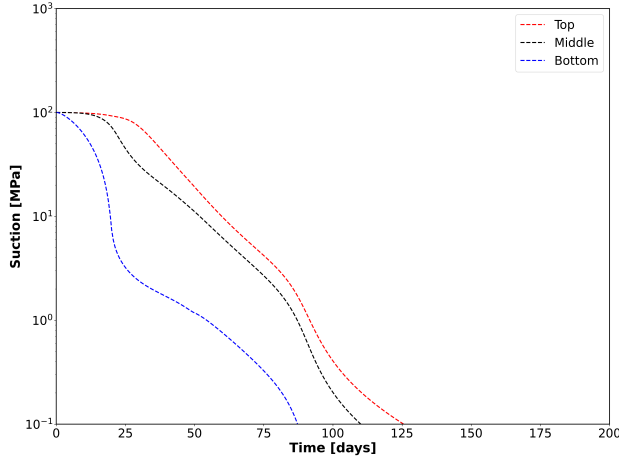


Figure 3.8: Reference configuration - Evolution of suction

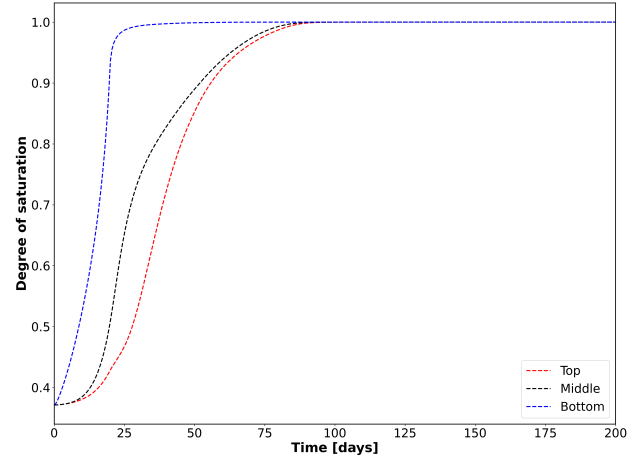


Figure 3.9: Reference configuration - Evolution of the degree of saturation

faster as the hydration comes from the bottom boundary.

Then, the evolution of the mean total stress is presented in Figure 3.10. It shows that the

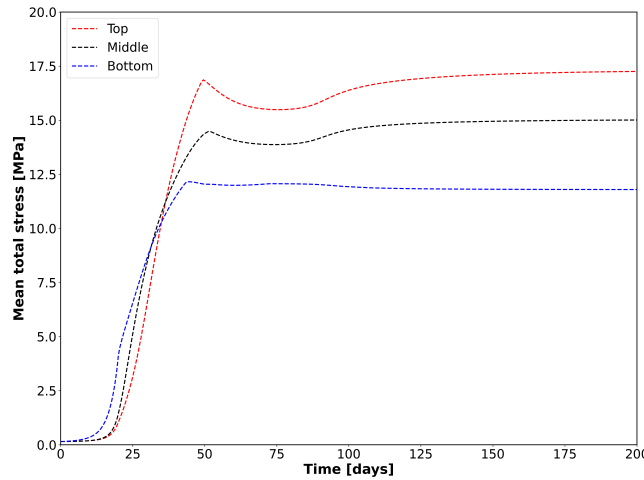


Figure 3.10: Reference configuration - Mean net stress evolution

mean total stress increases until plasticity is reached at around 44 days for the bottom point, 51 days for the middle point, and 49 days for the top point. Then, the mean total stress slightly decrease before increasing again to reach its final value, where it stabilises until the end of the simulation. Due to the hydration happening from the bottom, the mean total stress at the bottom is initially higher than at the middle or at the top but stabilises at a slightly lower value at the end, because swelling of the upper locations happens after the swelling of the lower location.

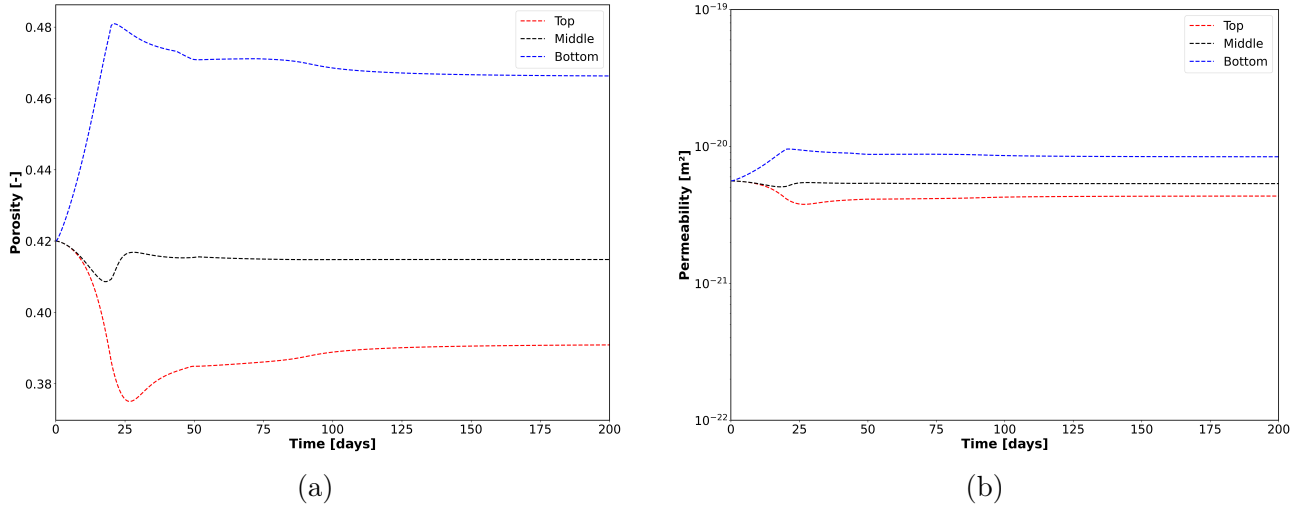


Figure 3.11: Reference configuration - (a) Evolution of porosity and (b) Evolution of permeability

Next, Figure 3.11a shows the evolution of porosity with time. At the beginning, only the bottom location experiences an increase in porosity. This is because the bottom part is the first to swell as it is closer to the hydration point. It thus compresses the middle and top parts, which undergo a decrease in porosity. Once the lower part is saturated, its porosity stabilises and the middle part experiences a small swelling. This swelling is smaller because the bottom part, which has already swelled, takes a lot of space and prevents it to swell more. Moreover, these variations of porosity have a direct impact on the dry density of the sample. Indeed, dry density evolves as:

$$\rho_d = (1 - \phi)\rho_s \quad (3.1)$$

Since solid density is constant, dry density decreases upon saturation. This is shown in Figure 3.12, where the evolution of dry density is indeed inversely related to that of porosity. Accordingly, several parameters of the BBM undergo variations during the saturation process, as they depend on the dry density.

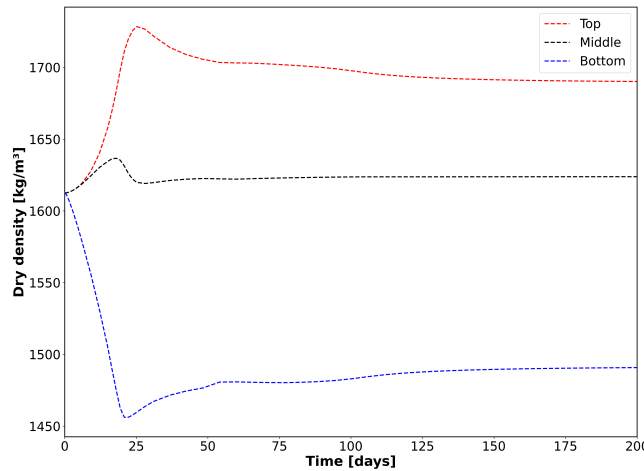


Figure 3.12: Reference configuration - Evolution of dry density

After that, Figure 3.11b presents the evolution of permeability with time. Similarly to the porosity, at the beginning, permeability only increases at the lower location before stabilising once saturation at the bottom is reached. Logically, the evolution of permeability is strongly

linked to the evolution of porosity, as a greater porosity leads to an increase in permeability.

Finally, the stress path in the (p-s) plane is shown in Figure 3.13. The mean net stress increases

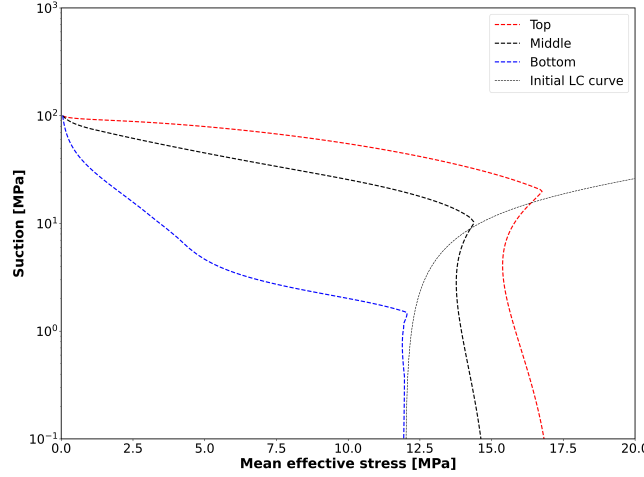


Figure 3.13: Reference configuration - Evolution of suction with the mean effective stress

upon hydration and reaches the LC curve. At this point, it continues to increase but at a slower rate, as plasticity is reached. At $s = 0.1 \text{ MPa}$, the sample is saturated and the stress path has reached the final LC curve.

3.5 Technological gaps

The addition of technological gaps can now be studied using the model presented in Figure 3.3. To begin with, the temporal evolution of the gap aperture is presented in Figure 3.14. It shows

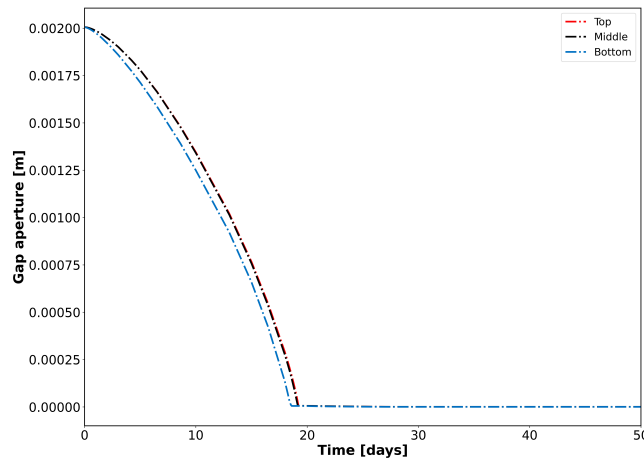


Figure 3.14: Technological gaps - Gap aperture

that bentonite swells over about 20 days before filling the gap. Once more, the bottom part being closer to the hydration front, the gap is filled faster there.

Then, the evolutions of the normal and tangential contact pressures are presented in Figure 3.15. The normal pressure only starts to develop after 20 days, once the gap is filled. It then reaches its maximum value after around 25 days, once the sample is saturated. On the other hand, no tangential pressure develops because the contact element and the sample is considered

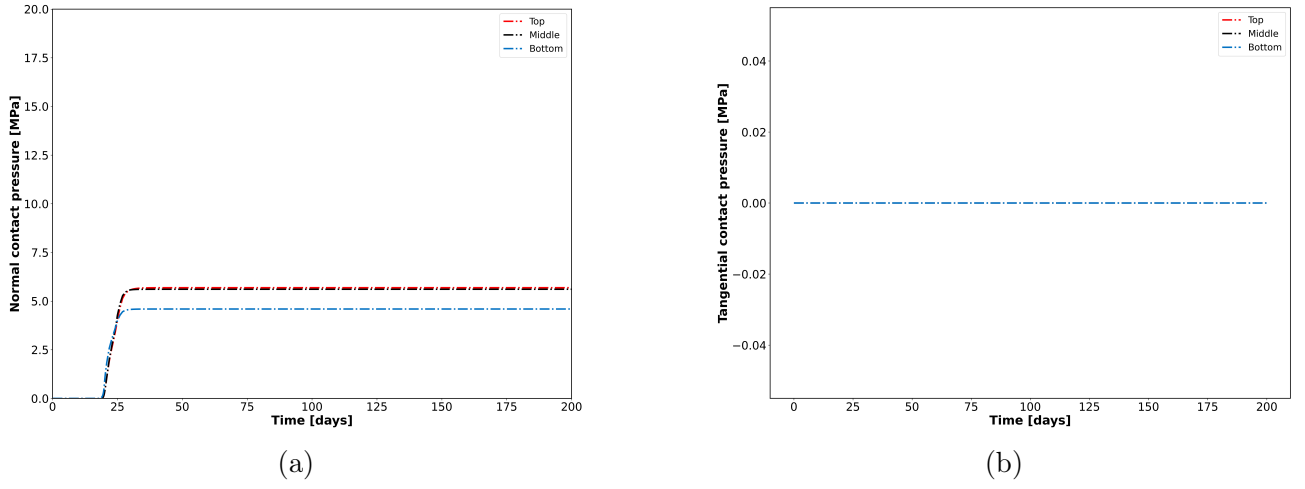


Figure 3.15: Technological gaps - (a) Evolution of the normal contact pressure and (b) Evolution of the tangential contact pressure

null in the model.

Next, Figure 3.16 shows the evolution of the radial stress at three locations of the sample (that can be seen in Figure 3.2), and for certain chosen times. The plotted variable is similar as in Figure 3.15, only the locations of the points that are plotted change. It shows that, similarly

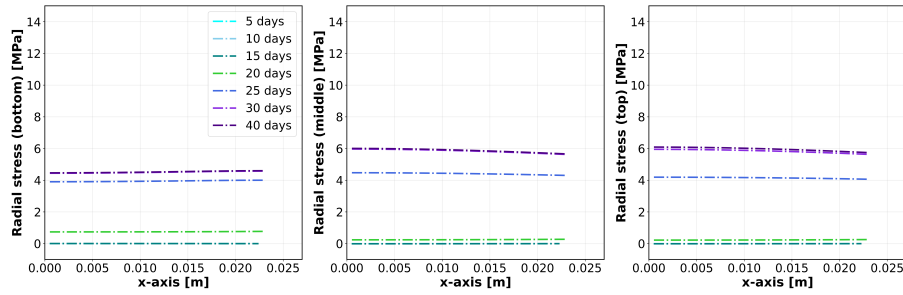


Figure 3.16: Evolution of the radial stress depending on the x-axis

to the normal contact pressure, the radial stress in the sample only starts to increase once the gap is filled, and stops increasing once the sample is saturated, as it can be seen that the curves corresponding to 30 and 40 days are indistinguishable.

After that, Figure 3.17 shows the evolution of porosity at certain times. Here, it can be seen

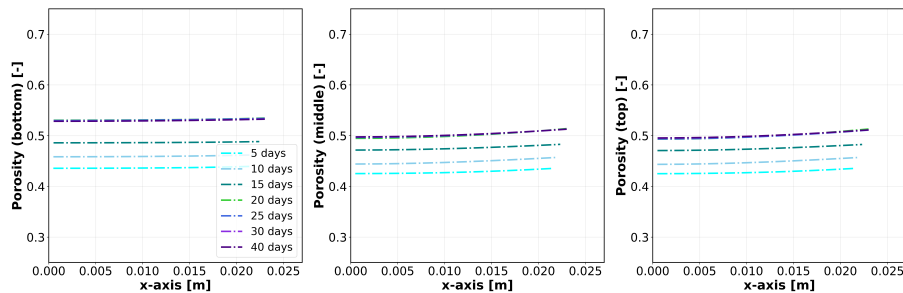


Figure 3.17: Technological gaps - Evolution of porosity depending on the x-axis

that porosity initially increases upon closure of the gap. After about 20 days, the gap is totally filled; thus, porosity remains constant.

To better understand the impact of the addition of technological gaps to the model, it is possible to plot the same figures as in section 3.4. In the following, the curves corresponding to the reference configuration will be plotted in light grey to allow for a better comparison with the curves corresponding to the technological gap model.

Firstly, the evolution of porosity and permeability is presented in Figure 3.18. Due to the

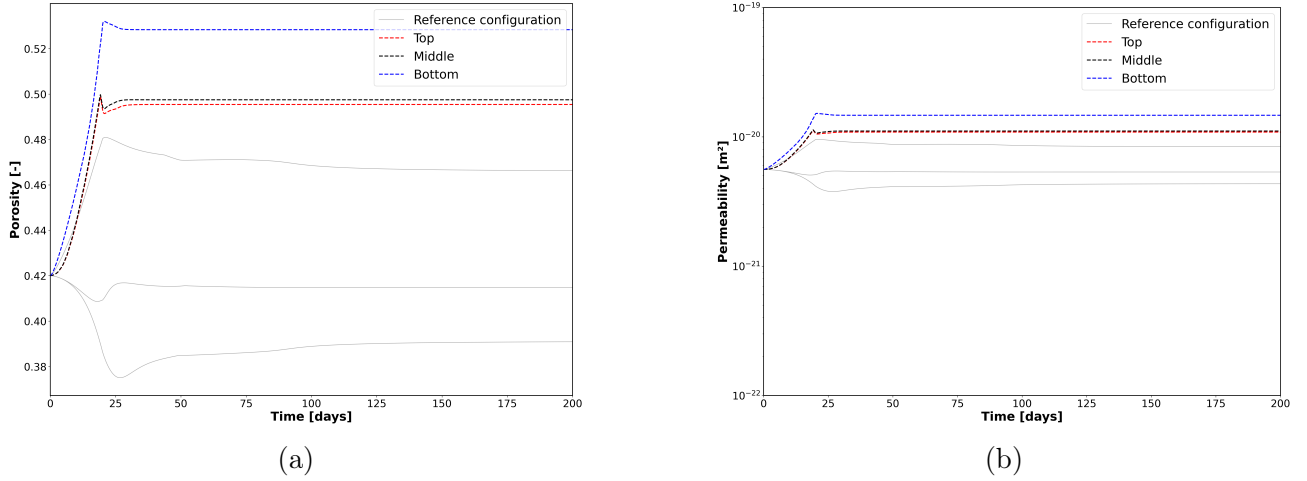


Figure 3.18: Technological gaps - (a) Evolution of porosity and (b) Evolution of permeability

fact that the sample can swell, the porosity increases to a higher value than in the reference configuration. In particular, at the beginning of the hydration, the bottom part of the sample no longer swells vertically, thus compressing the middle and upper parts, but swells horizontally, allowing the two other parts to swell as well, yielding a clear increase in porosity, especially at the middle and top locations. The increase in porosity has a direct impact on permeability, which is larger when technological gaps are present. Dry density is also impacted, as can be seen in Figure 3.19, where a significant decrease is observable at every location of the sample.

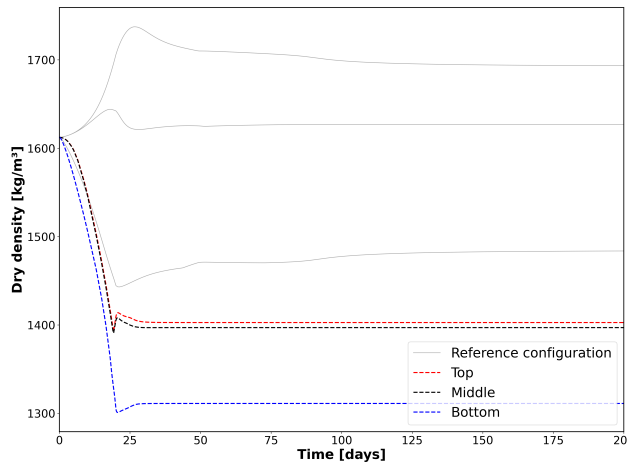


Figure 3.19: Technological gaps - Evolution of dry density

Secondly, Figures 3.20 and 3.21 show that the suction and degree of saturation develop faster than in the reference configuration. Indeed, saturation is reached in about 25 days. This can be explained by the increase in permeability, presented in Figure 3.18b.

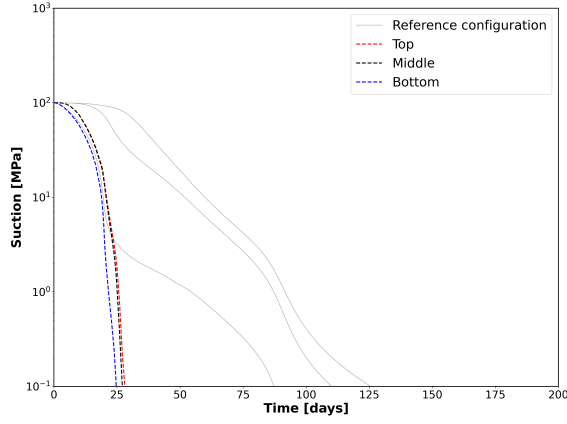


Figure 3.20: Technological gaps - Evolution of suction

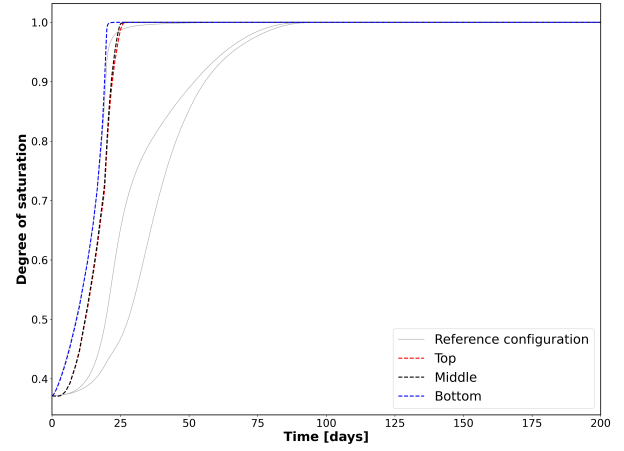


Figure 3.21: Technological gaps - Evolution of the degree of saturation

Finally, the evolution of the mean total stress and the stress path are presented in Figures 3.22 and 3.23. It can be seen that, due to the fact that saturation occurs faster, the LC curve is not

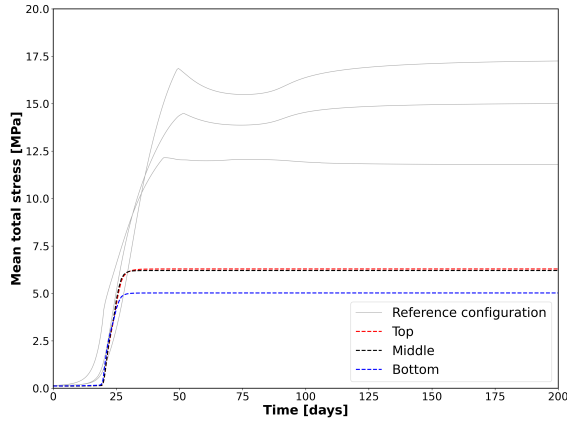


Figure 3.22: Technological gaps - Evolution of the mean total stress

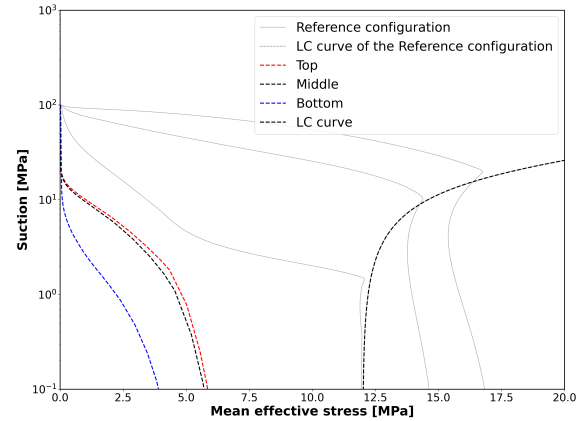


Figure 3.23: Technological gaps - Stress path

reached, meaning that there is no development of plasticity. It is also visible in the development of the mean total stress, as the stresses directly stabilises after reaching their maximum value.

3.6 Sensitivity analysis

Now that the initial graphs have been presented, it is possible to introduce variations in the parameters of the model. To better understand the impact of the changes, the initial graphs (Figures 3.8 to 3.13) will be plotted in light grey on each figure. In the following, the focus will be on the mechanical behaviour of the model.

3.6.1 Influence of the mechanical parameters on the stress path

In the (p-s) plane, the stress path is mainly affected by κ , κ_s , α_p , p_0^* , and λ_0 . In this section, the reference configuration is used to analyse the influence of these parameters.

Firstly, Figure 3.24 shows the influence of κ , whose value has been increased to 0.07. κ being

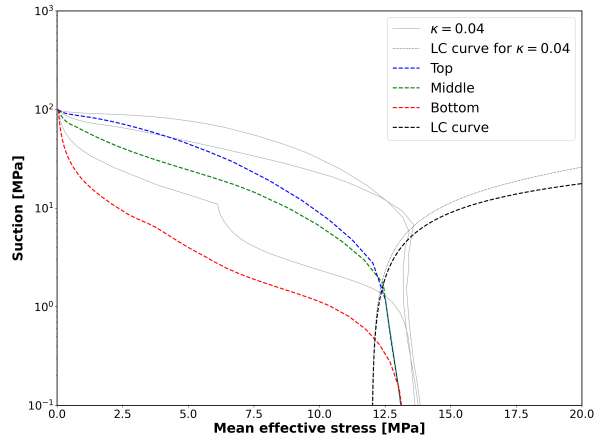
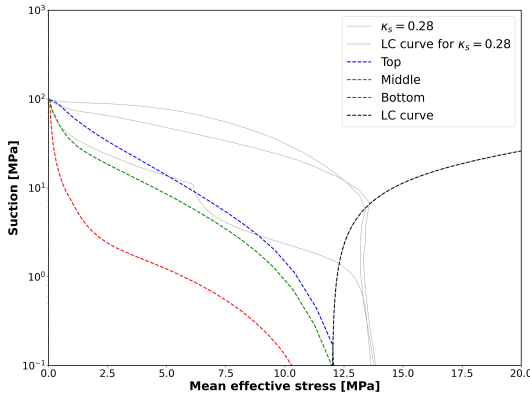


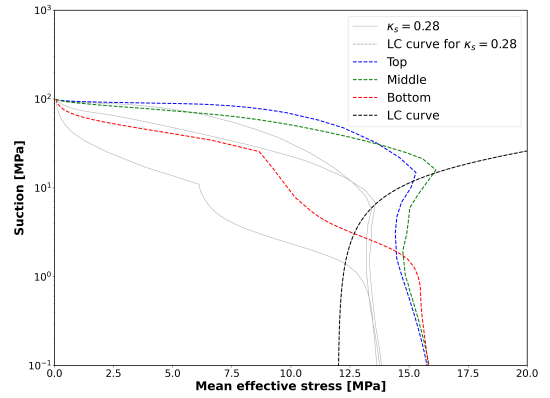
Figure 3.24: Influence of κ on the stress path

the elastic stiffness parameter, it controls the slope of the unloading-reloading line. Its increase yields a delayed development of plasticity, as the LC curve is reached for a lower value of suction. Moreover, for a same hydration level, the mean effective stress decreases. Both the LC curve and the slope of the elastic part of the stress path are affected by a change in κ .

Next, the influence of κ_s is shown in Figure 3.25. κ_s is also an elastic stiffness parameter, but



(a) $\kappa_s = 0.1$



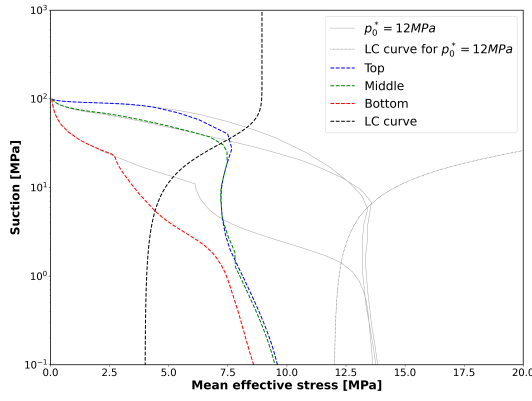
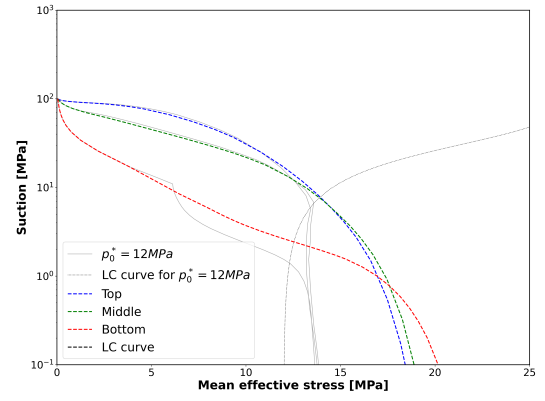
(b) $\kappa_s = 0.6$

Figure 3.25: Influence of κ_s on the stress path

this time for changes in suction. It thus controls the slope of the wetting-drying line. Unlike the previous case, the LC curve is left unchanged, as κ_s does not enter equation 2.49. However, regarding the stress path, it reaches the LC curve at a lower value of suction when κ_s decreases, meaning that plasticity develops at a higher value of saturation. Moreover, κ_s and κ have opposite impacts. It can be explained by the fact that the reference configuration does not allow swelling of the sample, meaning that $d\epsilon_v = 0$. Thus, equation 2.46 becomes:

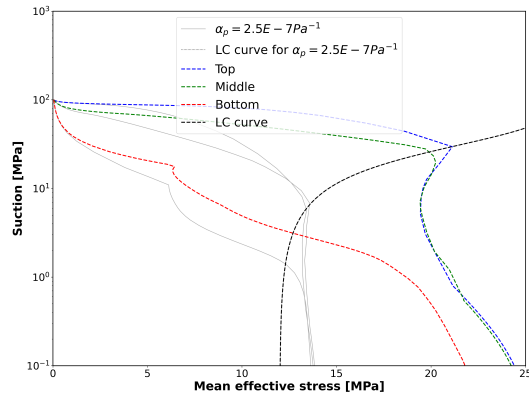
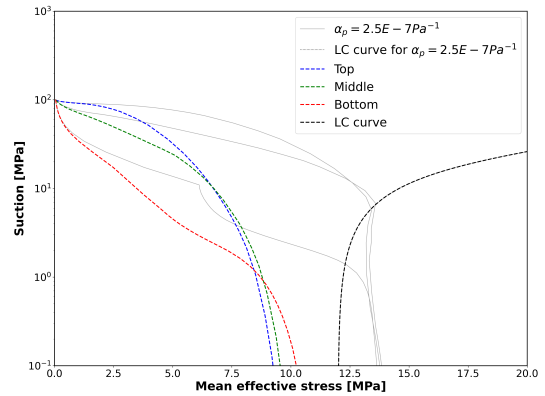
$$\frac{\kappa}{1+e} \frac{dp}{p} = \frac{-\kappa_s}{1+e} \frac{ds}{s+u_{atm}} \quad (3.2)$$

where the minus sign explains the difference between changes in κ_s and κ .

(a) $p_0^* = 4 \text{ MPa}$ (b) $p_0^* = 30 \text{ MPa}$ Figure 3.26: Influence of p_0^* on the stress path

Then, the impact of p_0^* is shown in Figure 3.26. It can be seen that p_0^* controls the shape of the LC curve. Indeed, as the pre-consolidation pressure for saturated state, it imposes the value of the final point of the LC curve, when saturation is reached. In consequence, the LC curve shifts to the left if p_0^* decreases and shifts to the right if p_0^* increases. Moreover, an increase in p_0^* also tends to increase the difference between the preconsolidation pressure at high suction and at low suction. These changes impact the stress path by delaying or advancing the point at which plasticity develops.

After that, the influence of α_p is shown in Figure 3.27. α_p controls the stress dependency of

(a) $\alpha_p = 1e - 7 \text{ Pa}^{-1}$ (b) $\alpha_p = 5e - 7 \text{ Pa}^{-1}$ Figure 3.27: Influence of α_p on the stress path

the swelling strain for change in suction. As can be seen, the effect of α_p is opposed to the one of κ_s , as, if it decreases, the LC curve is reached at a higher value of suction.

Finally, Figure 3.28 shows the influence of $\lambda(0)$. Here, $\lambda(0)$ is increased to 0.6. As the slope of the saturated virgin consolidation line, $\lambda(0)$ controls the slope of the stress path once the LC curve is reached. Indeed, the curves are nearly identical before reaching plasticity but part ways just before reaching it. When $\lambda(0)$ increases, lower values of mean effective stress are reached at saturation. In addition, $\lambda(0)$ appears in equation 2.49, meaning that it affects the LC curve. In particular, if $\lambda(0)$ increases, the LC curve shifts to the left, in particular for higher values of suction.

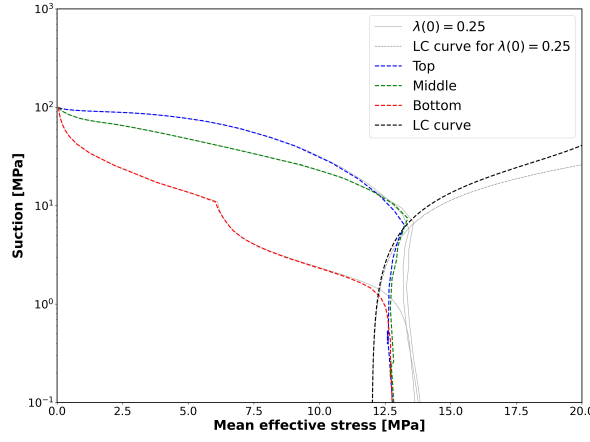


Figure 3.28: Influence of $\lambda(0)$ on the reference configuration

3.6.2 Influence of the mechanical parameters on the mean total stress

In this section, the reference configuration is used to study the evolution of the mean total stress with time. The same parameters as in the previous section are the most impactful.

First, Figure 3.29 shows the influence of κ , which has been increased to 0.07. It can be seen

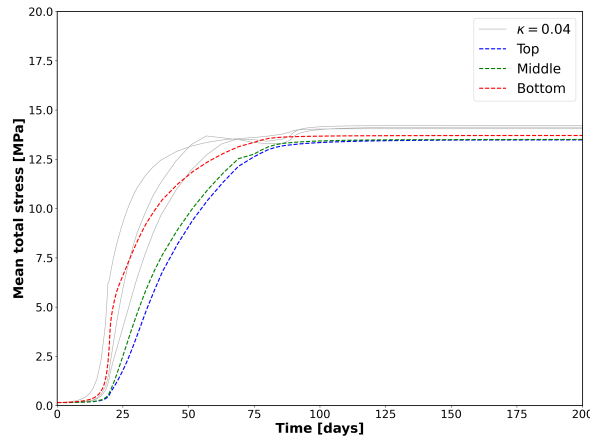


Figure 3.29: Influence of κ on the mean total stress

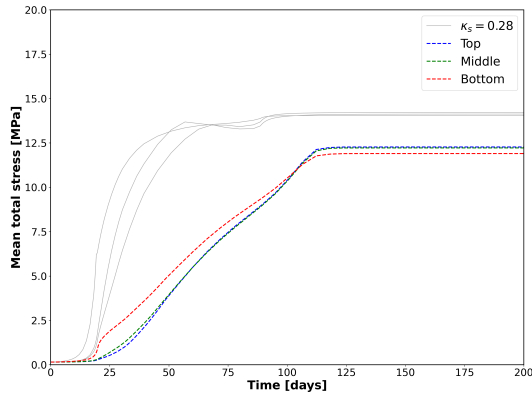
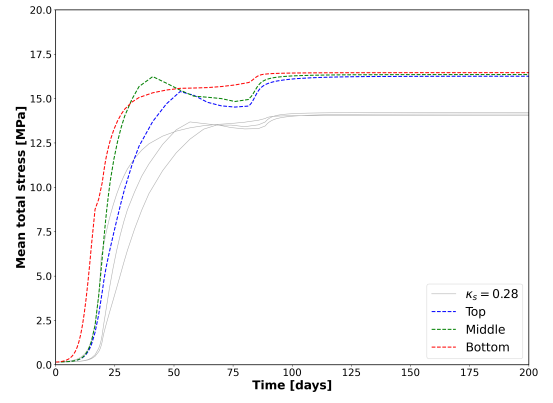
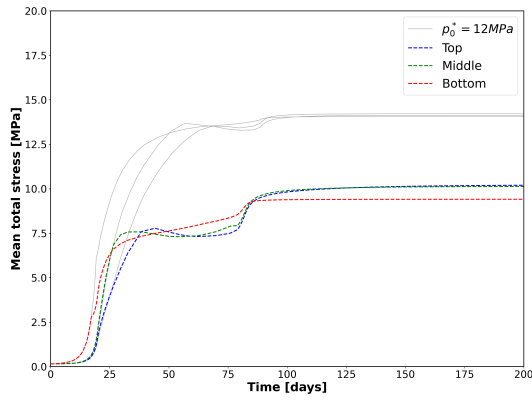
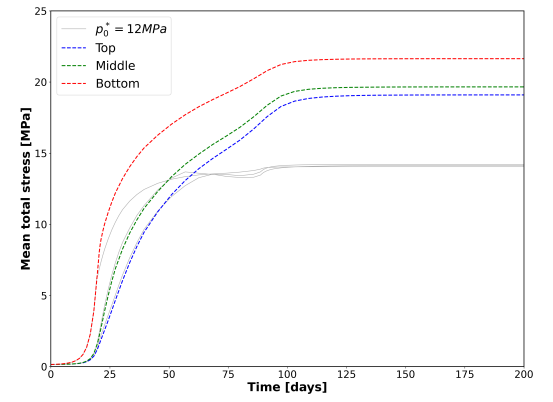
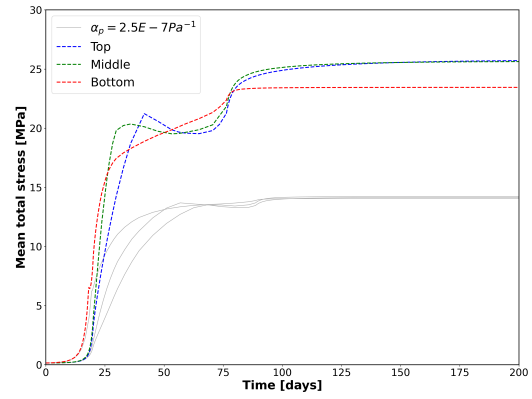
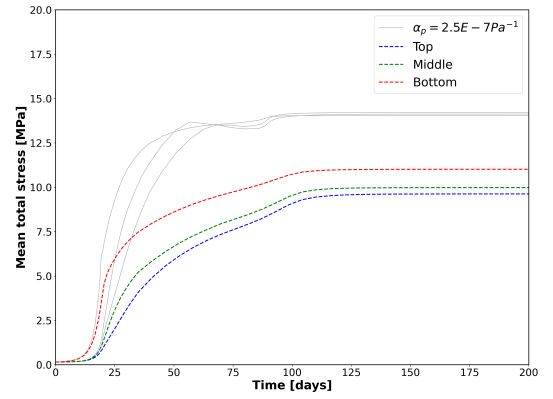
that an increase in κ yields an increase in the mean total stress, with a steeper slope before reaching saturation.

Next, Figure 3.30 shows the influence of κ_s on the mean total stress. Unlike the previous case, an increase in κ_s leads to a steeper slope before saturation, with saturation occurring sooner in time.

Then, Figure 3.31 shows the impact of p_0^* . An increase in p_0^* results in an increase in the mean stress, with saturation occurring later in time.

After that, the influence of α_p is shown in Figure 3.32. Similarly to the influence of κ , an increase in α_p causes a decrease of the mean total stress and a delay in the development of saturation.

Finally, Figure 3.33 shows the influence of $\lambda(0)$, whose value has been increased to 0.6. An

(a) $\kappa_s = 0.1$ (b) $\kappa_s = 0.6$ Figure 3.30: Influence of κ_s on the evolution of the mean total stress(a) $p_0^* = 4 MPa$ (b) $p_0^* = 30 MPa$ Figure 3.31: Influence of p_0^* on the evolution of the mean total stress(a) $\alpha_p = 1e-7 Pa^{-1}$ (b) $\alpha_p = 5e-7 Pa^{-1}$ Figure 3.32: Influence of α_p on the evolution of the mean total stress

increase in $\lambda(0)$ leads to a decrease in the mean total stress.

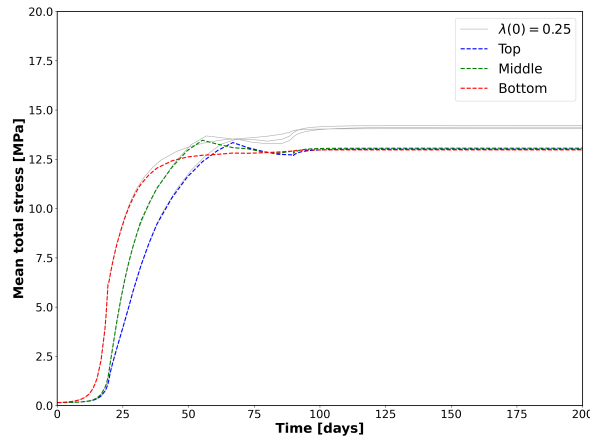


Figure 3.33: Influence of $\lambda(0)$ on the mean total stress

3.7 Preliminary conclusion

In this chapter, first small-scale models were presented. Next, a first set of parameters for the following simulations has been selected thanks to a calibration exercise. Then, the behaviour of a bentonite sample has been analysed, enabling a better understanding of its hydro-mechanical behaviour. Lastly, a sensitivity analysis was carried out on the mechanical parameters of the BBM to better understand the latter.

The next chapter will propose to get closer to reality, by the mean of an increase in the size of the sample but also by the implementation of additional features in the model. For instance, friction between bentonite and the repository walls or the presence of backfill in the sealing structure will be considered.

Chapter 4

Test cases for the engineered barrier material

4.1 Introduction

Now that the model's parameters have been calibrated and discussed, it is possible to study situations that are closer to the final repository conditions. To that end, several test cases of increasing complexity are proposed in Task 4 of the ANCHORS Workpackage (Talandier et al. 2025). These test cases are mainly inspired by a few national concepts, among which is the French one. These exercises are thus representative of the studied situation in this work, even if they are simplified in order to capture specific features of the sealing structures in geological repositories. In particular, a first case will aim to further study some typical properties of bentonite when the size of the bentonite sample is increased to be closer to the geometry of the actual situation, while a second test case will focus on the addition of backfill within the barrier.

4.2 Numerical models

The first proposed test case, shown in Figure 4.1, aims to study the swelling behaviour of bentonite. It consists of a column of bentonite, which is 7.8 metres high and has a diameter of 1.75 metres. The column is maintained by a rigid wall on each side, except at the top where a spring is applied to act as the backfill, which is deformable. The walls, which represent the host rock of the repository concepts, have an important rigidity such that the swelling of bentonite in their direction is prevented. The water supply differs based on the case: in case 1a, the supply is uniform along the sides of the sample, while in case 1b, the supply is localised at one and two thirds of the height of the sample, to mimic the situation in which water will arrive through fractures or preferential pathways. Moreover, friction is implemented at the interface between clay and rock. The objective of this test case is to study the swelling behaviour of bentonite, which will only be able to swell toward the spring, as this is the only boundary that is not completely rigid.

The model used to represent test case 1 is a 2D axisymmetric geometry, as can be seen in Figure 4.2a. The mesh is composed of 156 elements along the height and 18 elements along the width. Each element is an isoparametric eight-node finite element. Additionally, each finite element has a different width. Indeed, it was chosen to have a mesh that becomes progressively denser as it approaches the right side, where the hydration takes place. Contact elements are also placed at the bottom, right, and top boundaries, where the interfaces between bentonite and the walls representing the rock are located. To prevent movements toward the bottom and right contact elements, the latter have an important rigidity, equal to $10 \frac{GPa}{m}$.

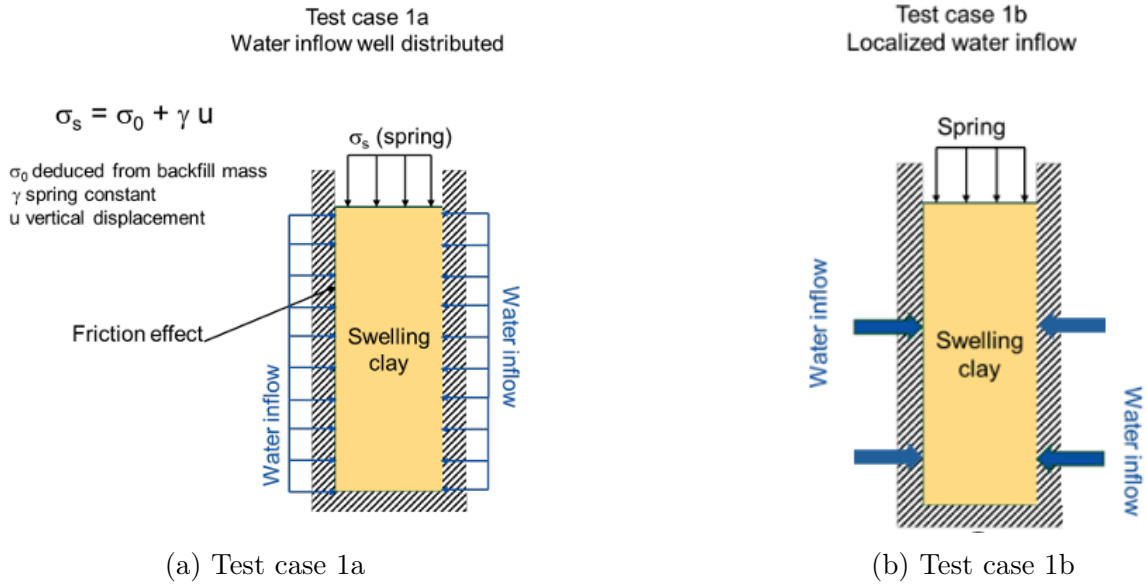


Figure 4.1: First test cases of ANCHORS project (Talandier et al. 2025)

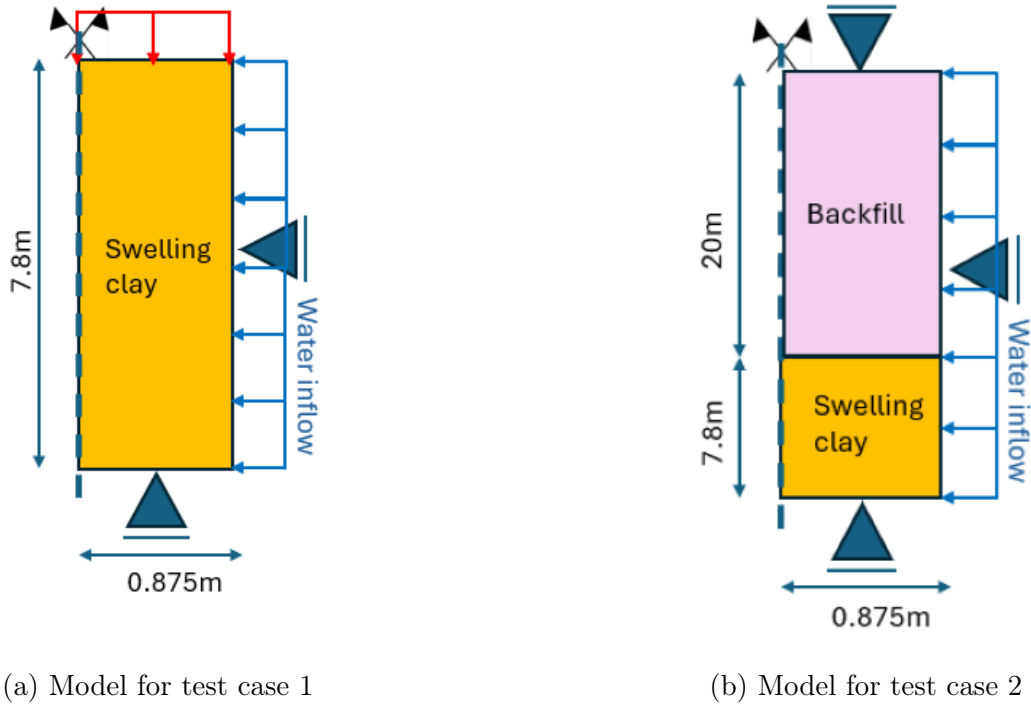


Figure 4.2: Schematic representation of the models for the two first test cases (not at scale)

Regarding the initial conditions, an initial stress of $-50kPa$ and an initial suction of $100MPa$ are applied within the sample, meaning that the sample is initially in an unsaturated state. The temperature and the gas pressure are both assumed constant during the entire simulation, respectively equal to $30^{\circ}C$ and $0.1MPa$. The gravity is also not taken into account. The friction angle between bentonite and the wall is taken equal to 10° , as proposed by Talandier et al. 2025, and the rigidity of the spring (represented in red in Figure 4.2a) is taken equal to $20 \frac{MPa}{m}$. The bottom and right boundaries are mechanically fixed, to prevent swelling in their direction. To simulate hydration of the sample, the right boundary undergoes an increase in water pressure, ramping from $-99.9MPa$ (which corresponds to the initial suction of $100MPa$) to $0.1MPa$

over one day.

The second test case, shown in Figure 4.3, introduces the presence of backfill in the barrier. In terms of geometry, the dimensions of the bentonite sample stay similar, and the height of

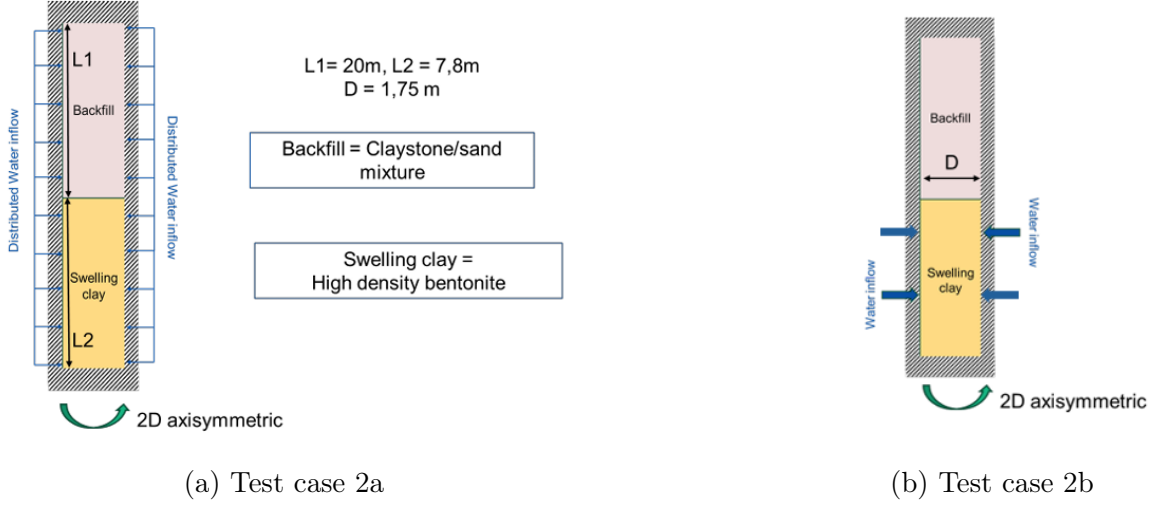


Figure 4.3: Second test cases of ANCHORS project (Talandier et al. 2025)

the backfill sample is fixed at 20 metres. As backfill is more permeable and swells less than bentonite, it will strongly impact the development of swelling pressure, the stress distribution in the sample, and the saturation process. Moreover, the interaction at the bentonite-backfill interface will also be studied. Unlike the first test case, the rigid wall is also placed at the top of the sample, meaning that the volume will remain constant. In terms of water supply, similarly to the first test, two cases are proposed: one with distributed inflow and one with localised inflow.

In test case 2, it is necessary to determine the models and the associated parameters for the backfill. For the French concept, the authors of the test cases (Talandier et al. 2025) suggested to use the BBM as the mechanical model, a Van Genuchten equation (Equation 4.1) as the water retention model, and Equation 4.2 for the relative permeability to water. A model for relative gas permeability is also provided, but, in the model described later, gas pressure is assumed constant.

$$S_e = \left[1 + \left(\frac{s}{P_r} \right)^n \right]^{-m} \quad (4.1)$$

where S_e is the effective saturation, P_r is the reference capillary pressure, and $m = 1 - \frac{1}{n}$ is a parameter.

$$k_{rl} = S_e^\eta \left[1 - (1 - S_e^{1/m})^m \right]^2 \quad (4.2)$$

where k_{rl} is the relative permeability of the liquid phase, and η is an empirical exponent. Talandier et al. 2025 also provided suggested values of the different parameters, as shown in Table 4.1.

The numerical model of test case 2 consists of the addition of 20 metres of backfill on top of the bentonite. Similarly to the first case, the mesh becomes denser when approaching the right boundary. However, the density also changes in the y-direction when approaching the interface

Table 4.1: Backfill parameters (Talandier et al. 2025)

(a) Van Genuchten and water permeability parameters			(b) BBM parameters		
Property	Symbol	Value	Symbol	Unit	Value
Reference pressure	P_r	2 MPa	κ	[–]	0.02
van Genuchten exponent	n	1.6	κ_s	[–]	0
Intrinsic permeability (water)	$K_{int,w}$	10^{-16} m^2	ν	[–]	0.3
Intrinsic permeability (gas)	$K_{int,g}$	10^{-17} m^2	M	[–]	1
Liquid rel. perm. exponent	η	0.5	p_c	MPa	0.1
			r	[–]	0.8
			β	MPa^{-1}	0.1
			k_s	[–]	0.1
			p_{s0}	MPa	0
			$\lambda(0)$	[–]	0.083
			p_0^* (initial)	MPa	0.15

between backfill and bentonite. Indeed, the mesh there is also denser. More precisely, the mesh is chosen with the same proportions as in the first test case: 18 elements across the width, 200 elements along the height of the backfill sample, and 78 elements along the height of the bentonite sample. In this case, a rigid wall is also added at the top boundary, restraining volume variations of the sample. Another modification concerns the initial state: the initial suction is now taken as 70 MPa in the bentonite core and only 4.4 MPa in the backfill. The initial stresses, however, remain equal to -50 kPa throughout the entire column. In terms of hydration, once again, the suction is imposed to decrease from its initial state (70 MPa for bentonite, 4.4 MPa for backfill) to 0 MPa after one day. However, at the node of the interface between bentonite and backfill, an intermediate value is taken at the initial time, equal to 37.2 MPa .

4.3 Test case 1

For the first test case, the results will be plotted at six measurement points shown in Figure 4.4. The objective is to study the development of stresses and saturation once the scale of the sample is closer to reality.

4.3.1 Distributed water inflow

First, the development of suction and saturation are shown in Figures 4.5 and 4.6. It can be seen that saturation logically occurs faster at the right boundary, where the hydration takes place. Accordingly, suction also develops much faster at those points, reaching 0.1 MPa in less than 10,000 days. At the left boundary, saturation is reached after about 15,000 days at the bottom and middle measurement points, and only after about 25,000 days at the top. It can also be observed that the two measurement points at the top boundary reach saturation later than the points with the same x-coordinate but at lower locations. It is explained by the fact that they are closer to the only non-rigid boundary, meaning that they undergo a higher swelling that delays their saturation. It is also visible in Figure 4.5, where a higher suction develops at the top points.

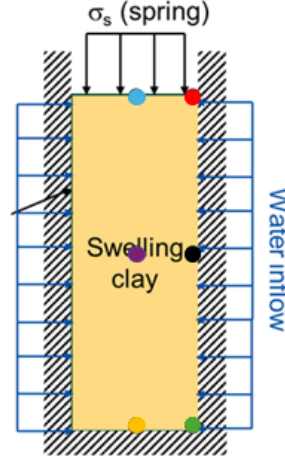


Figure 4.4: Measurement points for test case 1

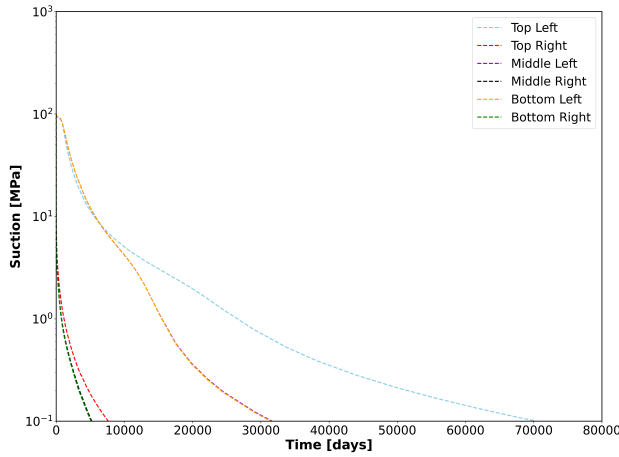


Figure 4.5: Case 1a - Temporal evolution of suction

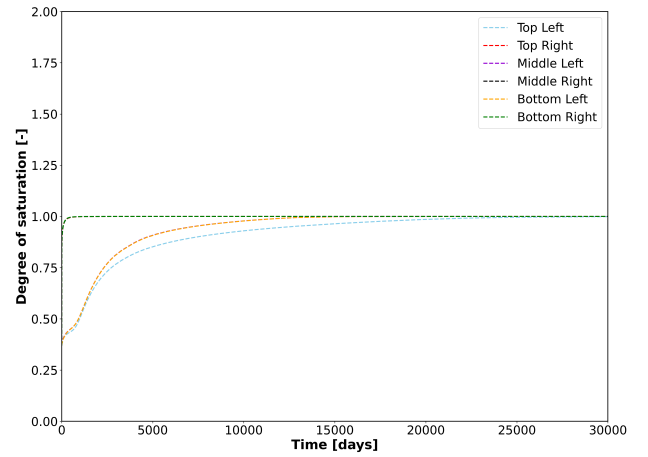


Figure 4.6: Case 1a - Temporal evolution of the degree of saturation

This slower development of saturation at the top points can be linked to the evolution of porosity, which is shown in Figure 4.7. Indeed, it can be seen that, due to the larger swelling they undergo (as explained in the next paragraph), the top points present a higher porosity than the two points at lower locations with the same x-coordinate. It can also be observed that, at first, the nodes at the right boundary experience an increase in porosity from the initial value. After that, their porosity slightly decreases due to the development of stresses, and finally stabilises at a value higher than the initial porosity. At the left boundary, the porosity initially decreases as the right part of the sample swells and compresses the left part. Then, the porosity stabilises a bit later than for the right points, as they are hydrated later.

Concerning swelling, Figure 4.8 shows the vertical displacement of the different nodes. The maximum swelling logically occurs at the top boundary, as the spring is less rigid than the walls. It also explains the higher value of porosity observed previously. At most, the sample swells by about 12cm. It is also visible that the vertical displacement strongly varies as a function of the height location of the measurement point. At the middle of the sample, slight

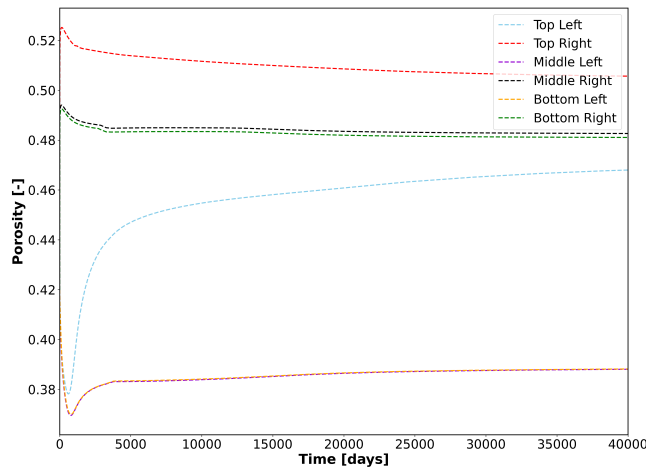


Figure 4.7: Case 1a - Temporal evolution of porosity

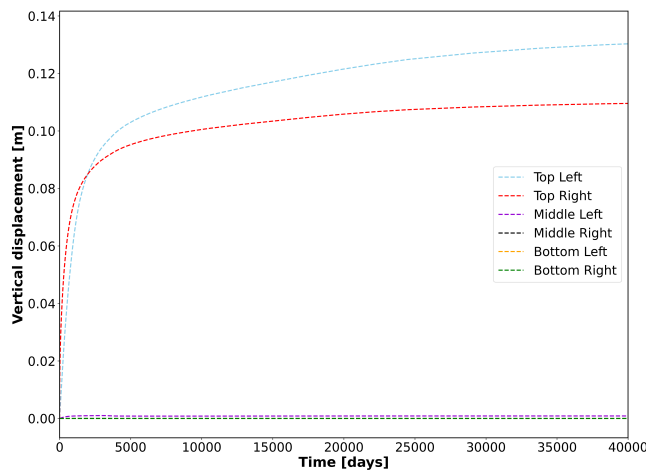


Figure 4.8: Case 1a - Vertical displacements

displacements occur, of about a few millimetres, while there are no vertical displacements at the bottom.

As the top locations experience more swelling, they develop lower pressures than the lower locations, as shown in Figure 4.9. Indeed, the top points are less compressed due to their proximity to the lower rigidity spring. On the contrary, the other locations do not swell much; thus,

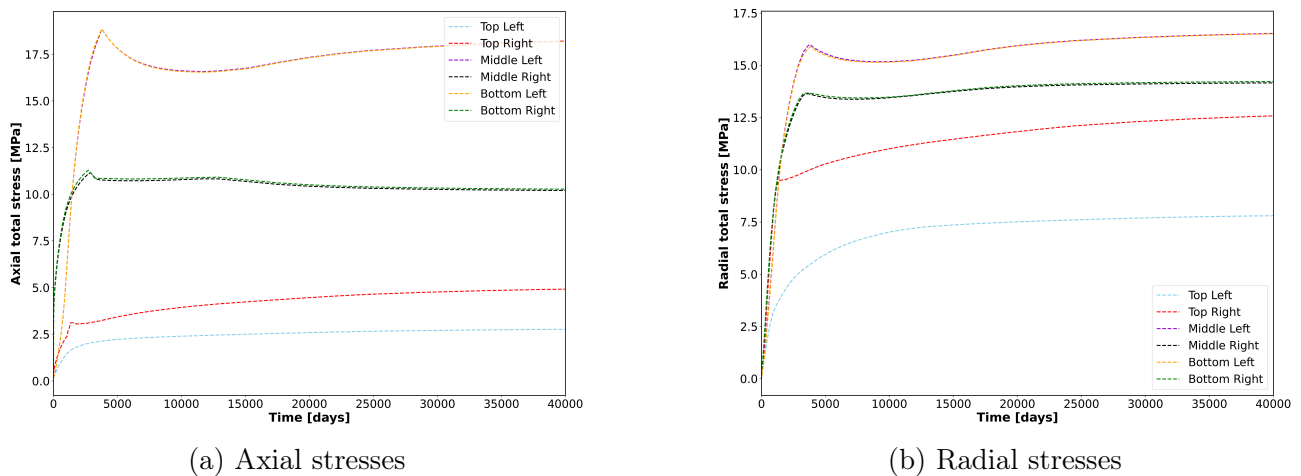


Figure 4.9: Case 1a - Temporal evolution of stresses

higher stresses develop there. This is particularly the case on the left of the sample, which is further from the water inflow, meaning that it swells last. At most, the middle and bottom left measurement points experience an axial stress of 18.7MPa and a radial stress of 16.8MPa .

Nevertheless, at the French URL, the in situ minor stress is only equal to 7MPa . This means that the developed swelling pressure in the EBS must stay lower than this value to maintain stability (Chen et al. 2018). Otherwise, new fractures would appear, which is the opposite of the desired effect. In addition, as the geometry approaches realistic dimensions, the simulation has more difficulties to converge. For the second test case, with the addition of 20 metres of backfill on top of the bentonite core, the simulation can not even reach the final time, due to numerical instability caused by the development of high pressure and the difficulty of the bentonite sample to swell. Hence, a new material is considered, in order to respect the threshold stress of the URL and to facilitate the swelling. This chosen material is still a MX-80 bentonite, but under the form of a mixture of 20% of powder and 80% of pellet.

The objective of this change is to have a mixture that develops lower swelling pressure and is more permeable than pure MX-80 bentonite. The main parameters and those whose value changed compared to the pure MX-80 bentonite are shown in Table 4.2.

4.3.1.1 Results for the new mixture

Compared to the pure MX-80 bentonite, Figure 4.11 shows that saturation occurs faster with the new mixture, in less than 10,000 days, even if the latter starts at a lower initial degree of saturation. For a better highlight of the impact of the new mixture, the results from Figures 4.5 to 4.9, which correspond to the results for the pure MX-80 bentonite, are plotted in light grey. Here, at worst, suction reached 0.1MPa after around 15,000 days at the top left location,

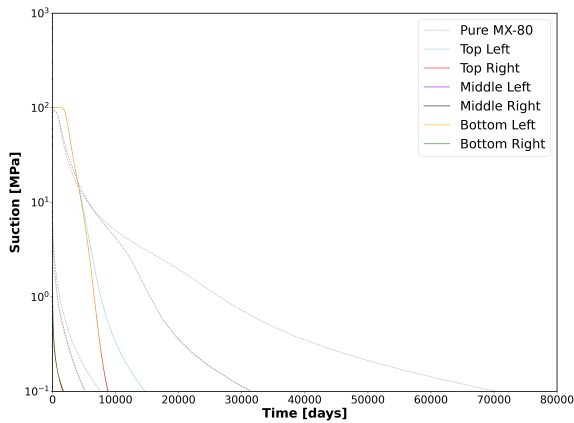


Figure 4.10: Powder-Pellet - Temporal evolution of suction

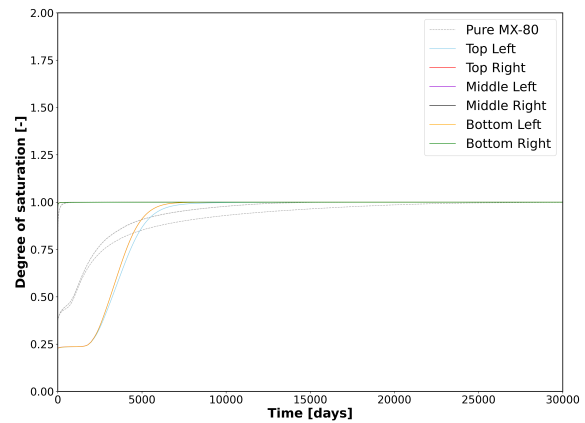


Figure 4.11: Powder-Pellet - Temporal evolution of the degree of saturation

and the difference between the points with the same x-coordinate is less pronounced, as can be seen in Figure 4.10. Especially, the middle and bottom curves are nearly identical, only slight differences are visible at lower values of suction.

This faster development of suction can be explained by the variations in porosity and permeability, which can be seen in Figures 4.12 and 4.13. Indeed, the new mixture presents a higher initial porosity at every point except the top left one. Accordingly, permeability is also greater, allowing for a faster progression of the water front in the sample. Only the top points undergo

Parameter	Symbol	Value	Unit
Mechanical			
Solid density	ρ_s	2780	$[\frac{kg}{m^3}]$
Initial porosity	ϕ_0	0.46	[-]
Poisson ratio	ν	0.3	[-]
Elastic compressibility coefficient (mean net stress)	κ	0.01	[-]
Initial elastic compressibility coefficient (suction)	κ_s	0.07	[-]
Parameter controlling swelling strain	α_p	4.5e-7	$[Pa^{-1}]$
Friction angle	φ	25	$[^\circ]$
Cohesion (saturated)	$c(0)$	0.1	$[MPa]$
Slope of virgin consolidation line	$\lambda(0)$	0.095	[-]
Minimum soil compressibility	r	0.8	[-]
Soil stiffness parameter	ω	0.05	$[MPa^{-1}]$
Suction cohesion parameter	k	0.0046	[-]
Pre-consolidation pressure	p_0^*	1	$[MPa]$
LC curve shape parameter	p_c	0.1	$[MPa]$
Hydraulic			
Retention – Macrostructure			
Λ		7.473	$[MPa]$
$\mathcal{N}$		1.9	[-]
$\mathcal{M}$		1.5	[-]
Retention – Microstructure			
C_{ads}		0.74e-8	$[Pa^{-1}]$
n_{ads}		1	[-]
e_{m0}		0.3519	[-]
β_0		0.15	[-]
β_1		0.1	[-]
Advective Darcy's law			
$k_{ij,0}^{int}$		1e-20	$[m^2]$
ϕ_0		0.46	[-]
M		3.0	[-]
N		1.5	[-]

Table 4.2: Parameters of the 20% powder 80% pellet mixture

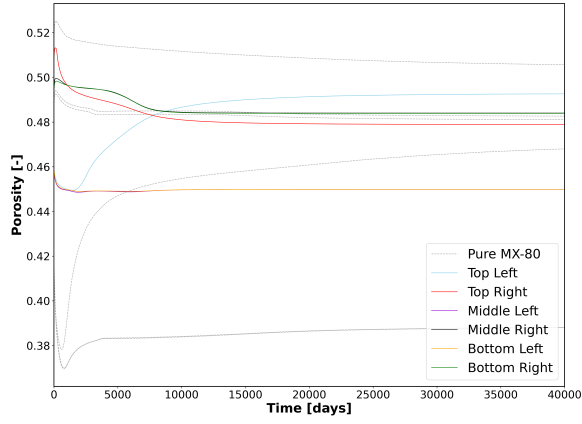


Figure 4.12: Powder-Pellet - Evolution of porosity

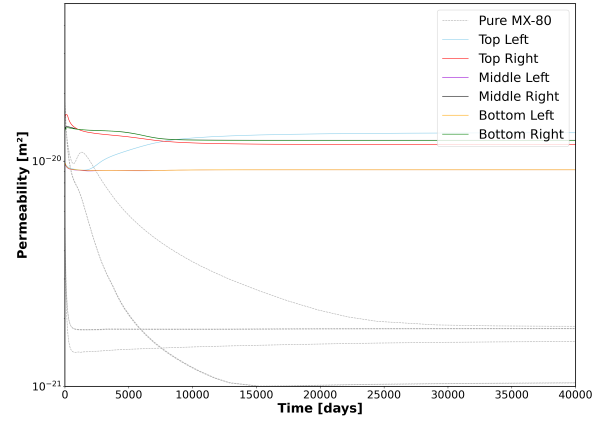


Figure 4.13: Powder-Pellet - Evolution of permeability

an increase in porosity, because they are the only points of the three that experience a significant vertical displacement, as is shown later in Figure 4.14. Of course, permeability is strongly linked to porosity, and thus their behaviour is similar. Moreover, with the new parameters, the model predicts a larger swelling at the top left point than at the top right point. This explains why the porosity of the top left point reaches a final value which is greater than that of the top right point.

Another impact of the pellet-powder mixture is the reduction of swelling, with a maximum value that has decreased to 6cm , as illustrated in Figure 4.14.

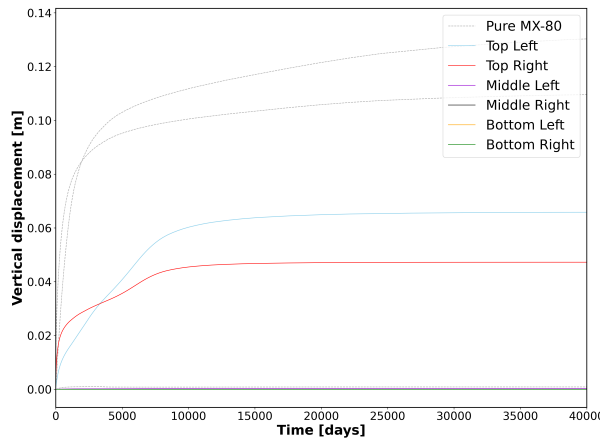
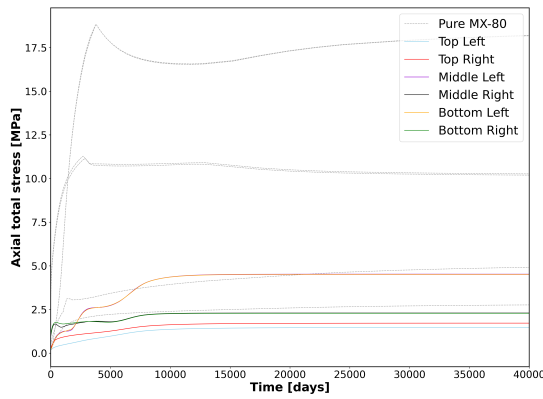


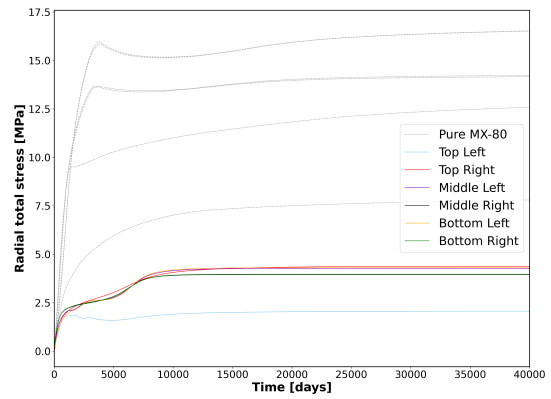
Figure 4.14: Powder-Pellet - Vertical displacements

In terms of stresses, the desired result has been attained, as the maximum axial and radial stresses are now respectively equal to 4.5MPa and 4.3MPa as presented in Figures 4.15a and 4.15b, which are below the in-situ stresses of the French URL. These results are in accordance with the decrease in swelling; if the sample is more permeable and swells less, the stresses must decrease. As was explained earlier, the point experiencing the most important swelling also experiences the lowest axial stresses.

In the following sections, unless specified otherwise, the material used in the simulations will always be the powder-pellet mixture.



(a) Axial stresses



(b) Radial stresses

Figure 4.15: Powder-Pellet - Temporal evolution of stresses

4.3.2 Localised water inflow

As explained earlier, in test case 1b, the water inflow is localised at one and two thirds of the height of the sample only, to simulate the presence of fractures in the host rock. Of course, it strongly impacts the progression of the water front, as depicted in Figure 4.16, where the propagation of water in the sample is shown for certain chosen times. As can be seen, water

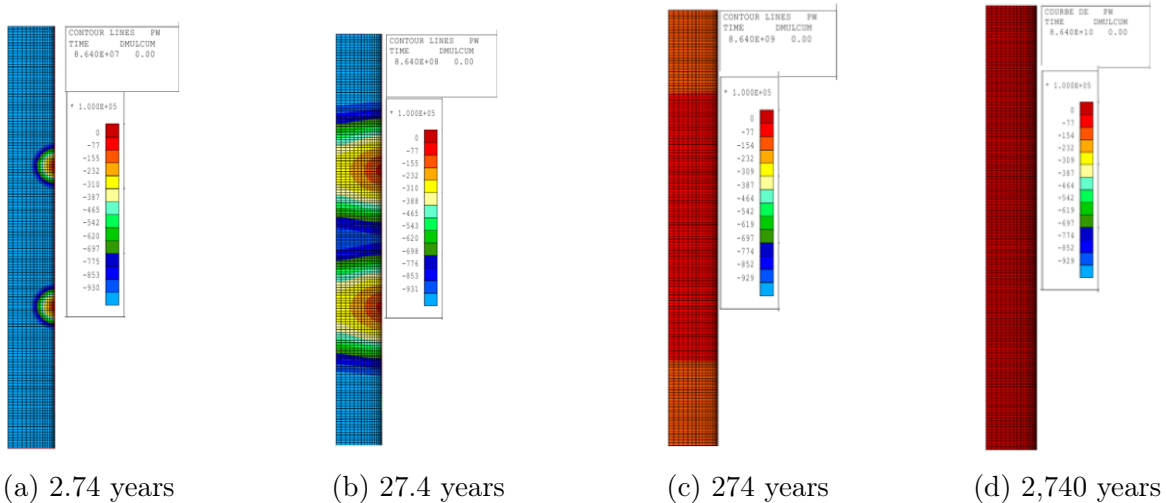


Figure 4.16: Case 1b - Water pressure at four chosen times

propagates radially from the entry points. Due to the fact that there are fewer entry points than in case 1a, saturation is still not reached in the sample after 100,000 days (≈ 274 years), as can be seen in Figure 4.18. The middle points, which are located between the two water entry points, experience a faster saturation process than the other points, which reach saturation after more than 200,000 days. It can be noted that, as saturation was already reached after 100,000 days, the simulation for test case 1a was not run for a longer time, explaining why the curve stops after 100,000 days. Figure 4.17 shows the temporal evolution of suction, compared to test case 1a, at the six measurement points of Figure 4.4. Due to the symmetry of the water inflow, the curves related the top and bottom points nearly coincide. Only the middle points, which are between the two water inlets, undergo a faster saturation process. Moreover, due to the slen-

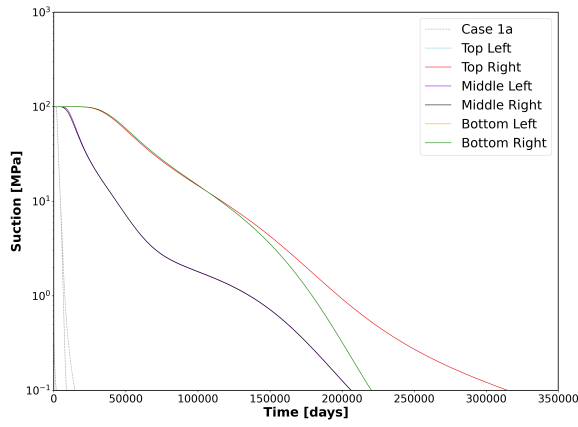


Figure 4.17: Case 1b - Evolution of suction

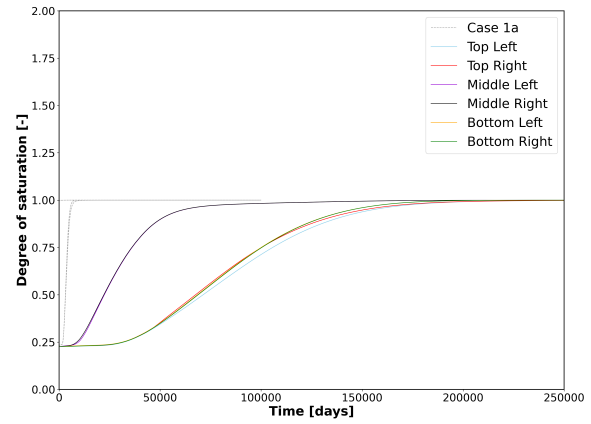


Figure 4.18: Case 1b - Time evolution of the degree of saturation

derness of the sample, the difference between the left and right points is not visible on the graph.

Next, the vertical displacement is shown in Figure 4.19. In this case, vertical swelling develops

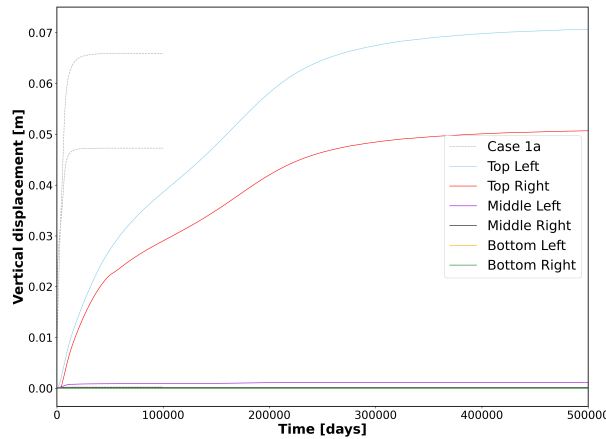
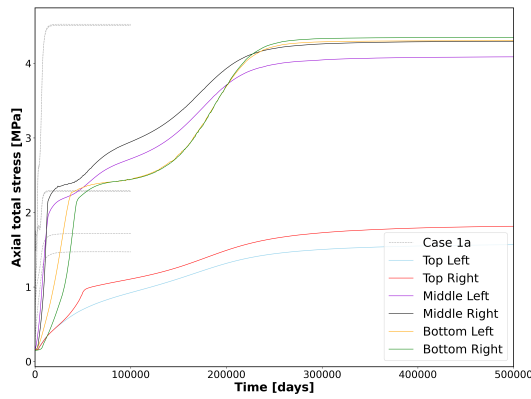


Figure 4.19: Case 1b - Vertical displacement

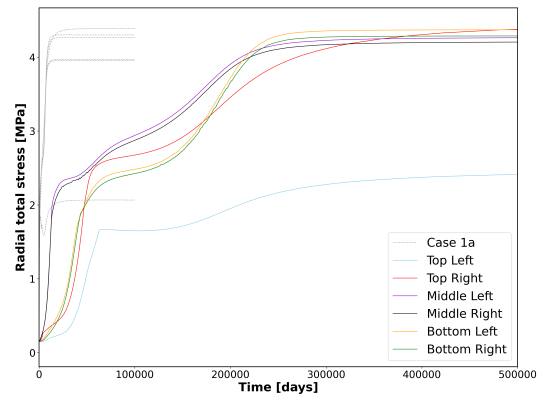
slower but reach a final value which is slightly larger compared to test case 1a. This result can be linked to the variation in axial stress, which is shown in Figure 4.20, along the evolution of radial stress. Indeed, it can be seen that both stresses also develop more slowly compared to case 1a, due to the slower evolution of suction. However, the final value of both stresses remains similar at nearly every measurement point. Only the axial stresses at the bottom and middle right points increase, as they are no longer adjacent to the water entry points.

4.3.3 Preliminary conclusions

This first test case highlighted that the pure MX-80 bentonite developed pressures that were too high for the in-situ conditions of the Bure URL. Accordingly, a new mixture of MX-80, made of 20% of powder and 80% of pellet, has been employed and made it possible to reduce the stresses, due to its higher permeability. The new mixture also has an impact on the development of swelling, which decreased. Then, the variation of the water inflow showed that a smaller amount of entry points does not strongly impact the final value of stresses or the final



(a) Axial stresses



(b) Radial stresses

Figure 4.20: Case 1b - Evolution of axial and radial stresses

swelling of the sample, but that it delays them.

4.4 Test case 2

For test case 2, the results will be plotted at the measurement points shown in Figure 4.21. However, for a better understanding, the points for backfill and bentonite will be plotted on two separate graphs, using the same colour code as in the previous section.

The objective of this second test case is to study the impact of the addition of 20m of backfill in the barrier on the development of swelling pressure and saturation. It makes it possible to get a step closer to the realistic repository situation.

4.4.1 Distributed water inflow

Firstly, Figure 4.22 shows the propagation of water in the sample at certain chosen times. Unlike the other test cases, two water fronts now propagate through the bentonite sample. In addition to the one coming from the right boundary, where the water inflow is applied, there is also a second one coming from the backfill sample, as depicted in Figure 4.22b. Indeed, backfill is initially closer to saturation and is more permeable than bentonite, meaning that its saturation occurs significantly faster. After that, it acts as a source of water for bentonite. Figure 4.23 shows that saturation is reached after less than 10 days in the backfill. Moreover, it can be seen that the bottom points both reach saturation later than the middle and top points with the same x-coordinate as them. Alongside the decrease in degree of saturation experienced by the bottom left point, decrease that is not visible at the bottom right point due to its proximity to the water inflow, it confirms that part of the water from the backfill goes toward the bentonite core. In bentonite, saturation is reached after more than 10,000 days at the bottom and middle left points. A zoom on the first 20 days of the simulation is provided in Appendix A for Figure 4.23a, showing that the top left point is already nearly saturated after 20 days, due to the water coming from the backfill.

The strong difference in the saturation process between the two cores can be related to the evolution of permeability, which differ by three orders of magnitude between the bentonite and

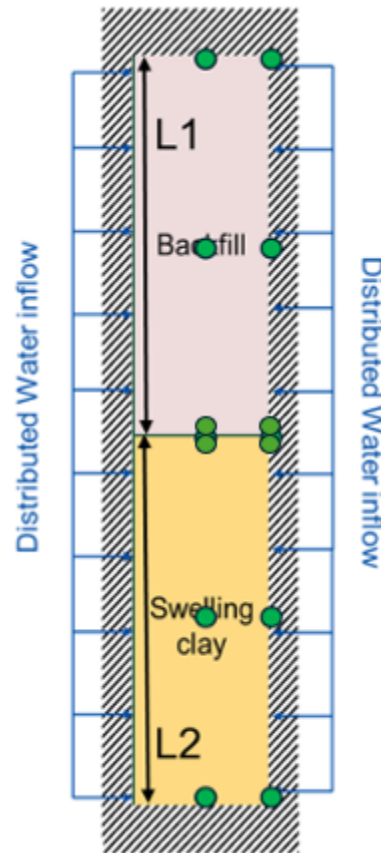


Figure 4.21: Test case 2 - Measurement points

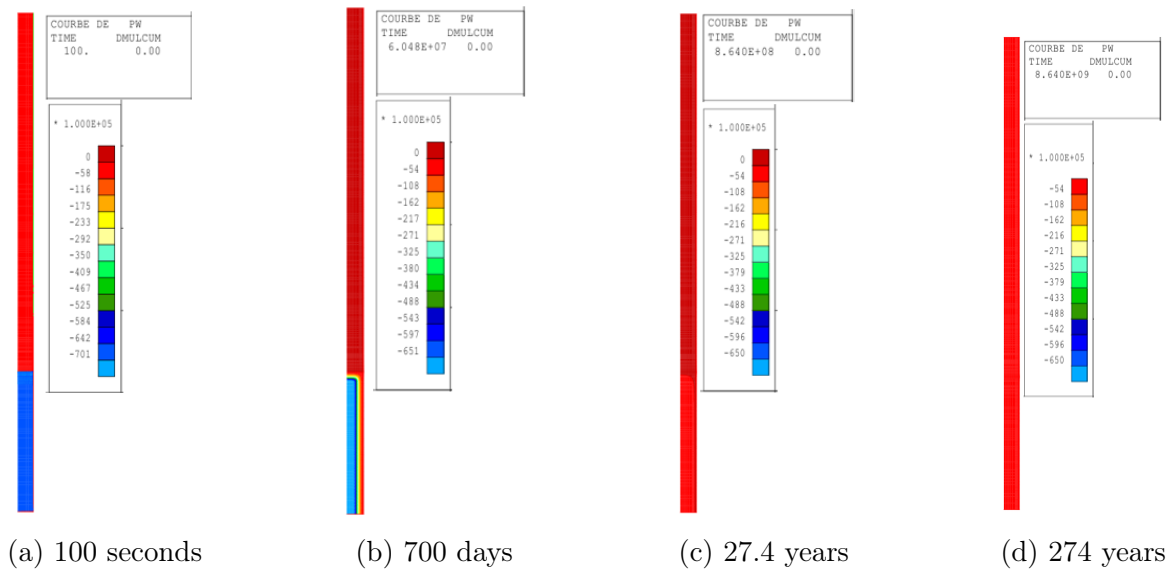
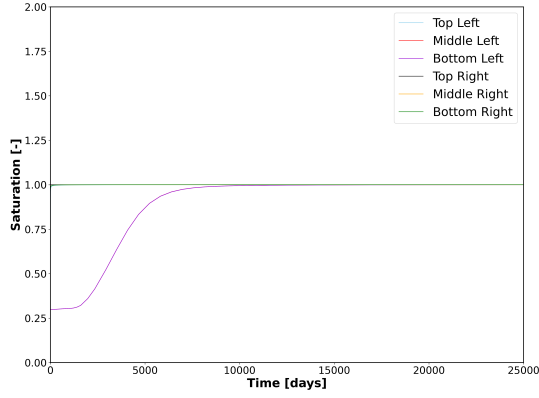


Figure 4.22: Case 2a - Water pressure

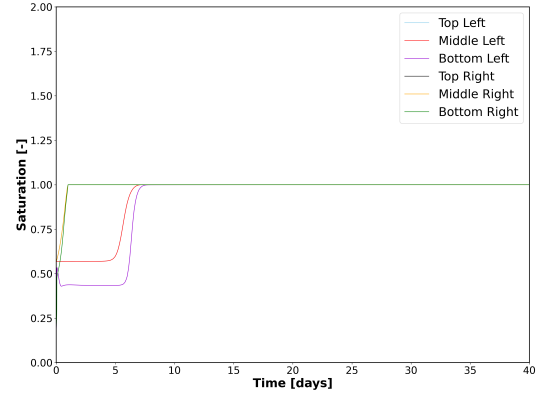
backfill, as shown in Figure 4.25.

It can also be seen in Figure 4.24, where the development of suction is shown at the different measurement points. In the backfill, at the left measurement points, the curves for the top and middle points overlap as there are no strong enough differences due to the fast propagation of water. The slight difference between the bottom and the two other points is explained by the

location of the bottom point, which is close to the border with bentonite. Logically, the bottom left point reaches 0.1MPa a bit later because of the water that goes down toward bentonite.

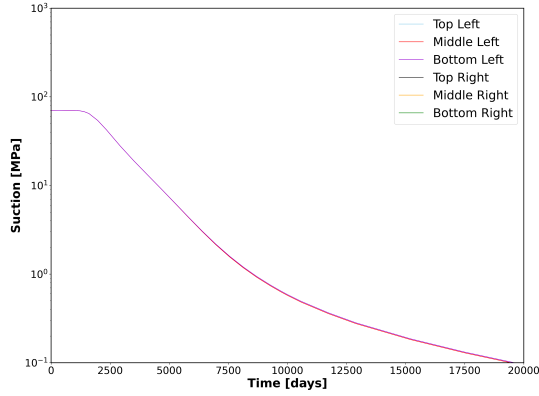


(a) Bentonite

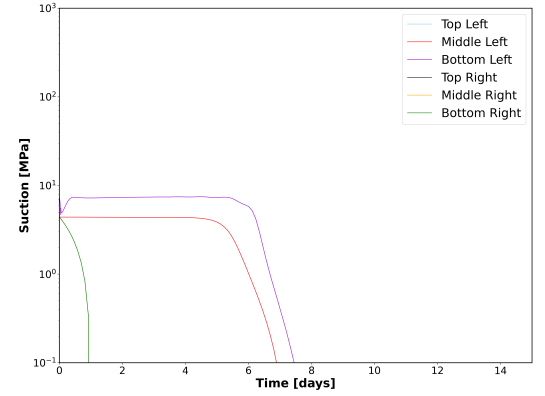


(b) Backfill

Figure 4.23: Case 2a - Evolution of the degree of saturation in the two samples



(a) Bentonite



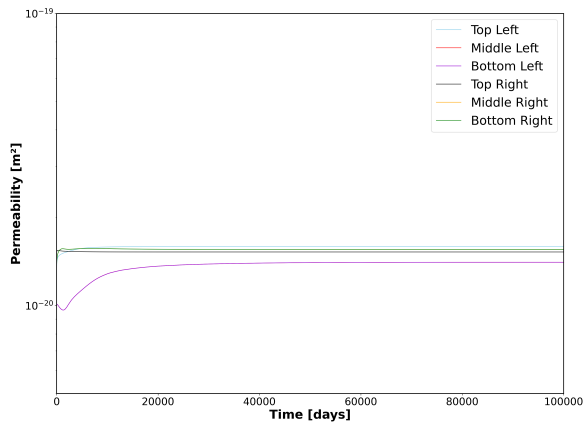
(b) Backfill

Figure 4.24: Case 2a - Evolution of suction in the two samples

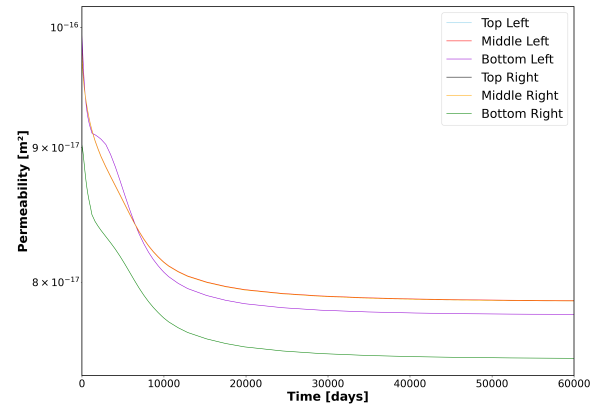
In the bentonite sample, due to the slender shape of the model, the water front coming from the backfill only reaches the top left point but not the middle and bottom points. This is why the results for the mid left and bottom left points are similar, while the curves for the right and top left points are too close to the y-axis to be visible.

In terms of stresses, Figures 4.26 and 4.27 show the evolution of radial and axial stresses at the measurement points. Compared to test case 1a, both stresses have decreased in the bentonite sample. Indeed, in bentonite, the axial stress does not even reach 1MPa , while the maximum radial stress is worth about 1.4MPa . In the backfill, even lower stresses develop.

Even though axial stresses are lower, the vertical displacement increased compared to test case 1a, as presented in Figure 4.28, where a minus sign means that the sample size decreases. Here, the vertical displacement reaches a maximum value of about 0.7m , while for test case 1a, it was about 0.06m . As the only change compared to the previous test case is that backfill has been added, it means that the rigidity of the backfill thus appears to be much lower than the rigidity of the spring that was previously applied. Concerning the size variation of the backfill, the latter logically experience a reduction in size that exactly corresponds to what is gained by

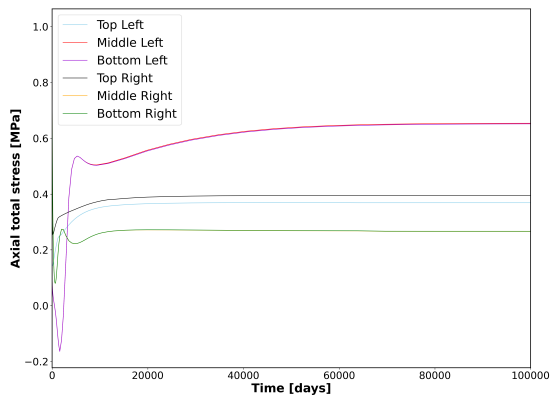


(a) Bentonite

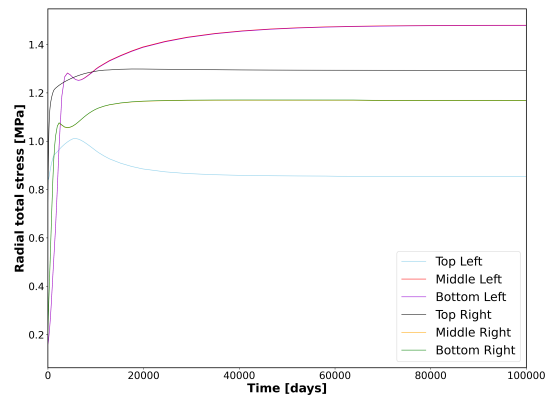


(b) Backfill

Figure 4.25: Case 2a - Evolution of permeability with time in the two cores

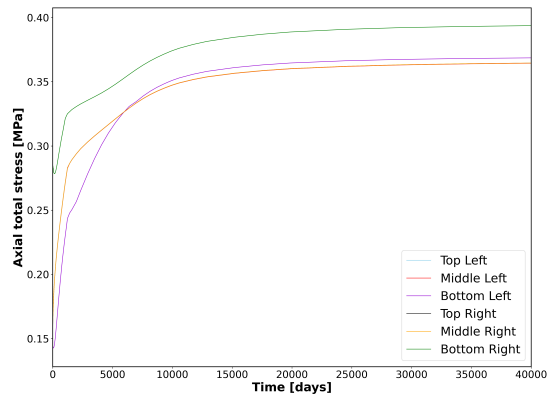


(a) Axial stresses - Bentonite

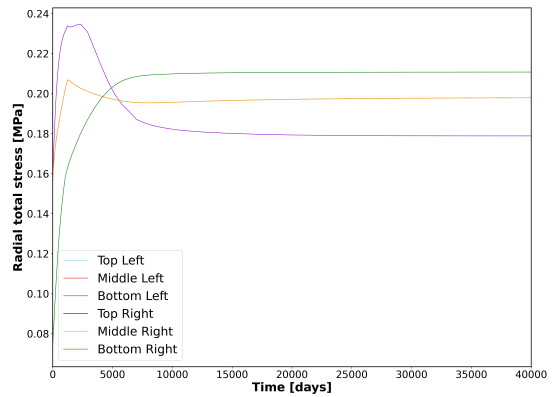


(b) Radial stresses - Bentonite

Figure 4.26: Case 2a - Evolution of axial and radial stresses in bentonite



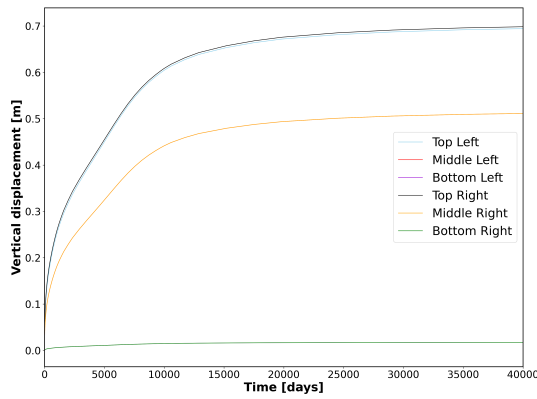
(a) Axial stresses - Backfill



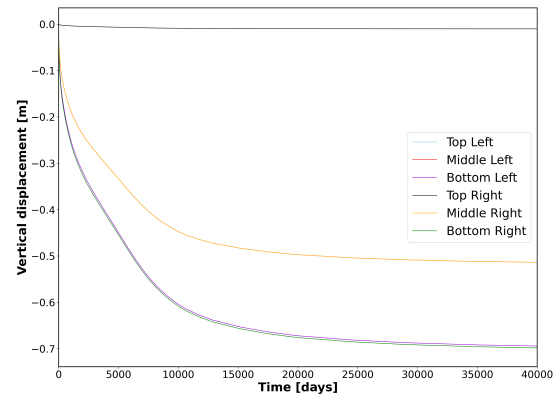
(b) Radial stresses - Backfill

Figure 4.27: Case 2a - Evolution of axial and radial stresses in backfill

the bentonite core.



(a) Bentonite



(b) Backfill

Figure 4.28: Case 2a - Evolution of the vertical displacement

4.4.2 Localised water inflow

As explained earlier, the water inflow is now only localised at one and two thirds of the height of the bentonite sample. To begin with, Figure 4.29 shows the evolution of water pressure at four different chosen times. Similarly to test case 1b, the water front from the inlets propagates

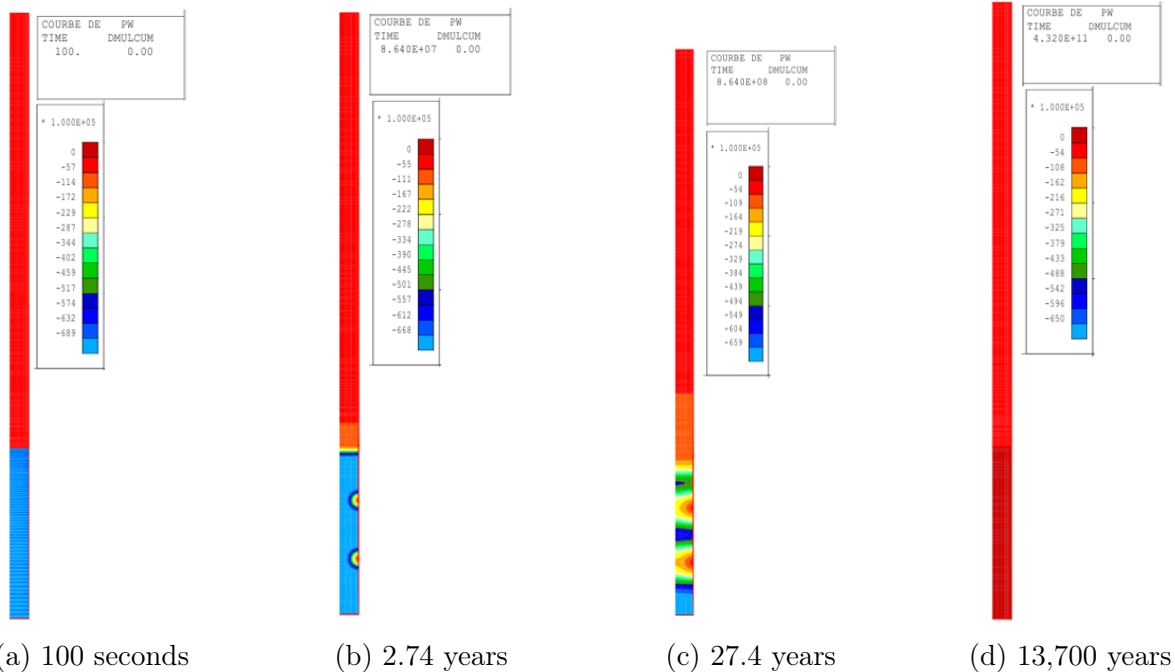
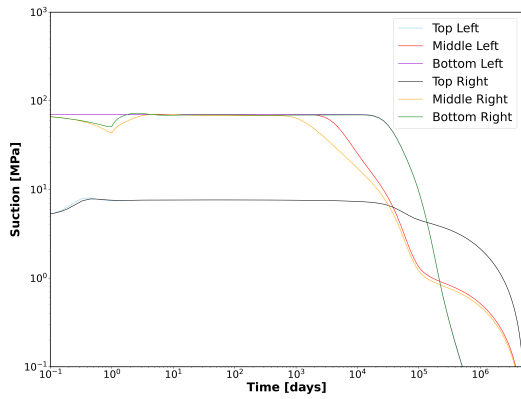


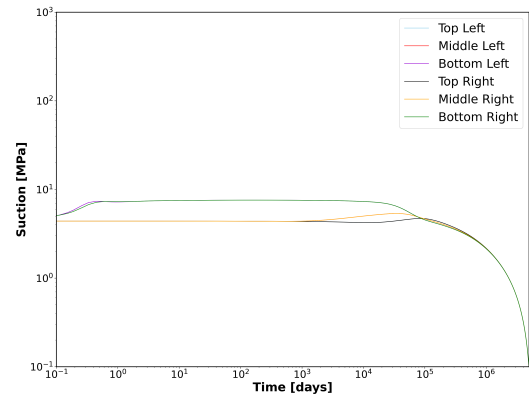
Figure 4.29: Case 2b - Water pressure at four chosen times

radially. However, as there is no more water injection at the boundary of the backfill sample and because water from the backfill migrates toward the bentonite sample, the backfill core experiences a desaturation front at its lower location. After 13,700 years, the water pressure has stabilised at a lower value in bentonite than in backfill. The evolution of suction is plotted in Figure 4.30, where it can be seen in Figure 4.30b that suction initially undergoes an increase at the bottom location, confirming the desaturation.

Figure 4.31 shows the evolution of the degree of saturation. To observe the initial decrease in the degree of saturation at the bottom points of the backfill, a zoom on the first 40 days

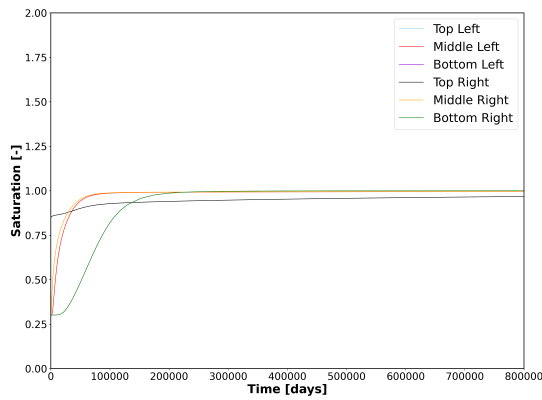


(a) Bentonite

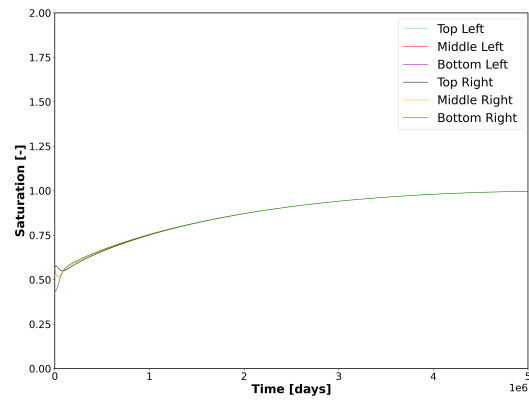


(b) Backfill

Figure 4.30: Case 2b - Evolution of suction with time



(a) Bentonite



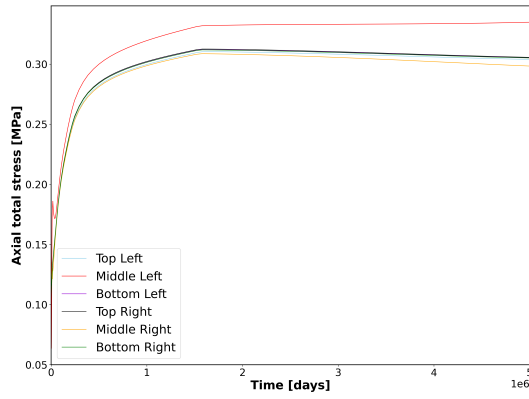
(b) Backfill

Figure 4.31: Case 2b - Evolution of the degree of saturation with time

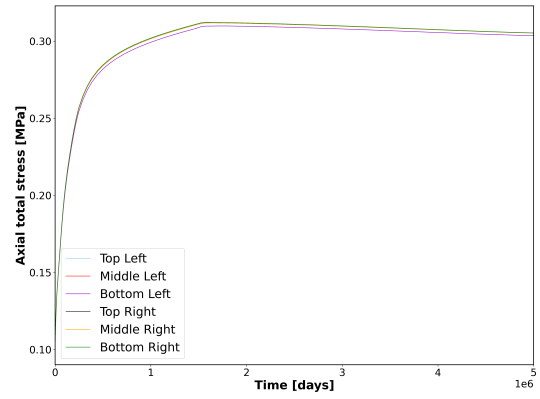
of the graph is shown in Figure A.2 in Appendix A. Moreover, it can be seen in Figure 4.31b that backfill is still not totally saturated after 5,000,000 days ($\approx 13,700$ years). Concerning saturation in bentonite, the top right and left points are the slowest to reach saturation, as they are closer to backfill. Indeed, as there are no water entry points on the side of the backfill, water from the bentonite sample migrates toward the backfill core, but this process is much slower due to the low permeability of bentonite.

In terms of stresses, as shown in Figures 4.32 and 4.33, both axial and radial stresses remain at values that are similar to the case with the distributed water inflow. As for the first test case, the difference is that the development is slower when the water inflow is localised instead of distributed.

The same comments can be made about swelling. As shown in Figure 4.34, the final vertical displacement develops slower than in test case 2a but has a final value of about $0.7m$ at most, similarly to test case 2a.

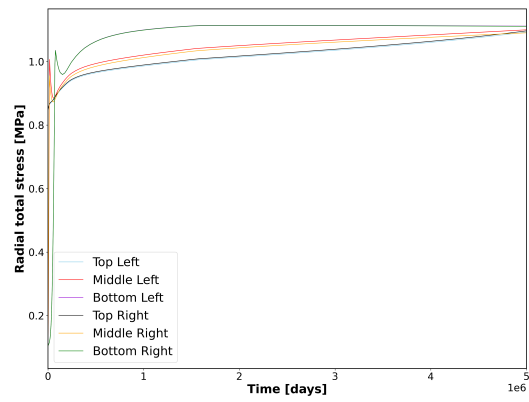


(a) Bentonite

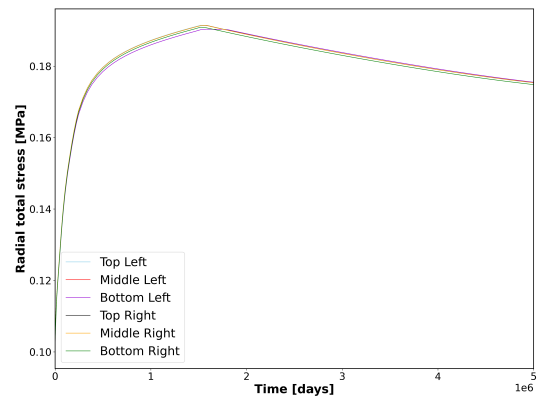


(b) Backfill

Figure 4.32: Case 2b - Evolution of axial stresses with time

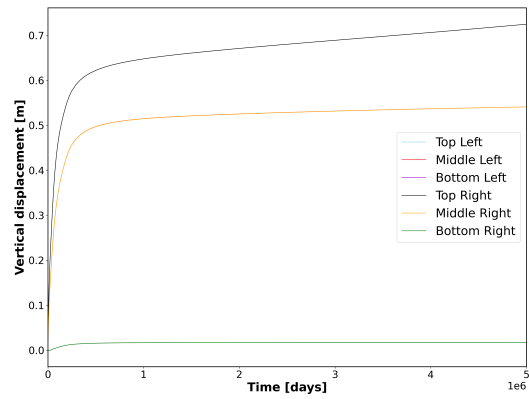


(a) Bentonite

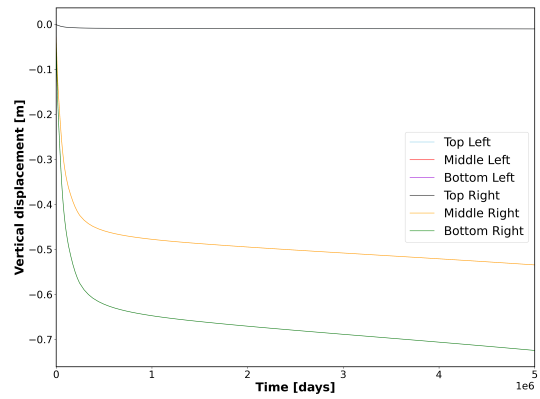


(b) Backfill

Figure 4.33: Case 2b - Evolution of radial stresses with time



(a) Bentonite



(b) Backfill

Figure 4.34: Case 2b - Evolution of vertical displacement with time

4.4.3 Preliminary conclusion

With this second test case, the impact of the addition of backfill has been studied. In particular, it highlighted that the rigidity that was chosen for the spring in test case 1 was probably too high. As a result, the bentonite sample experienced a larger swelling in test case 2. Moreover,

it pointed out that the backfill core can also act as a source of water for the bentonite core. Indeed, the increase in the degree of saturation is more pronounced for the points that are located closer to the boundary between bentonite and backfill because of the water front that propagates from the backfill.

In the following chapter, the full sealing structure will finally be modelled. Furthermore, different phases of the lifetime of the gallery will also be represented.

Chapter 5

Full test case

5.1 Introduction

Finally, the reference sealing structure will be studied by adding the host rock, the EDZ, and the concrete lining to the model. One half of the whole structure will be represented, as shown in the schematic representation of Figure 5.1. This last test case will make it possible to compare

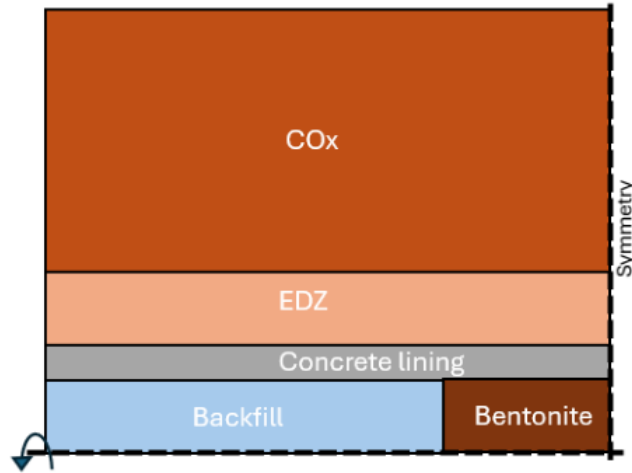


Figure 5.1: Schematic representation of the studied structure (not at scale)

the behaviour of the bentonite based seal to the previous test cases once the whole structure is modelled.

5.2 Numerical model

To represent this third test case, the model depicted in Figure 5.2 is employed, with a geometry of $50m$ in length and $25m$ in radius. Only one half of the barrier is modelled, with two axes of symmetry on the right and bottom boundaries. Once more, Talandier et al. 2025 already provided the parameters necessary to represent the EDZ, host rock, and concrete lining. These parameters can be seen in Table 5.1. While the hydraulic model of the three materials remains similar to what has previously been presented, the mechanical model can be simplified. Indeed, an elasto-plastic constitutive law is sufficient to represent them. Thus, the model presented in Section 2.3.2.2 can be used.

In the full test case, the geometry becomes more complex because of the additional materials. It remains a 2D axisymmetric geometry, with a first axis of symmetry on the right boundary,

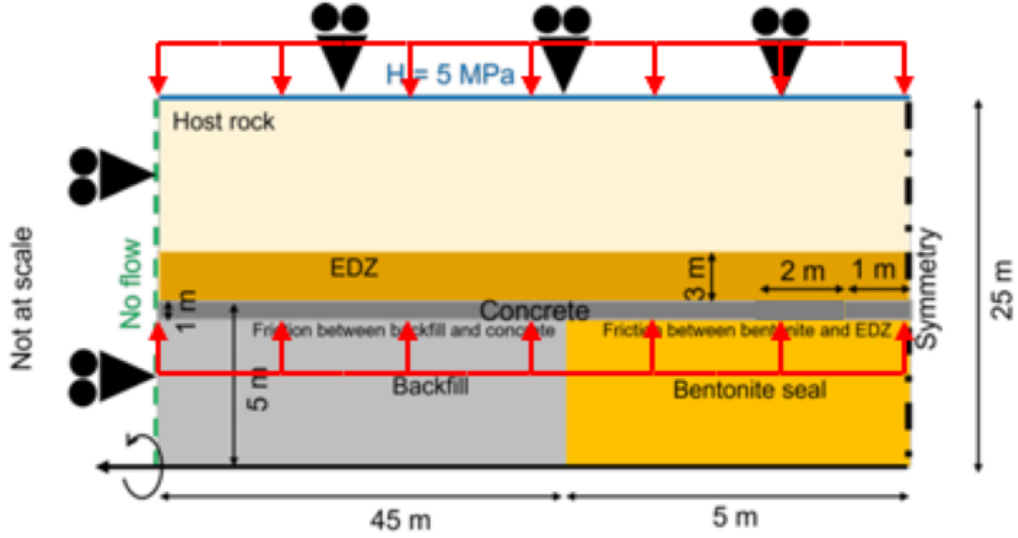


Figure 5.2: Model for the third test case from ANCHORS project (Talandier et al. 2025) (not at scale)

Table 5.1: Hydraulic and mechanical parameters of the host rock, EDZ, and concrete lining

<i>Hydraulic parameters</i>				<i>Mechanical parameters</i>	
Concrete liner	Reference pressure	P_r	0.5 MPa	E	40 GPa
	van Genuchten exponent	n	1.6	ν	0.25 (-)
	$K_{\text{int},w}$		10^{-17} m^2	b (Biot coefficient)	0.8 (-)
	$K_{\text{int},g}$		10^{-15} m^2	c	14.6 MPa
	η		0.5	ϕ	38 °
Host rock	Reference pressure	P_r	20 MPa	E	5 GPa
	van Genuchten exponent	n	1.5	ν	0.3 (-)
	$K_{\text{int},w}$		10^{-20} m^2	b (Biot coefficient)	0.8 (-)
	$K_{\text{int},g}$		10^{-19} m^2	c	0.5 MPa
	η		0.5	ϕ	20 °
EDZ	Reference pressure	P_r	15 MPa	E	5 GPa
	van Genuchten exponent	n	1.5	ν (-)	0.3
	$K_{\text{int},w}$		10^{-18} m^2	b (Biot coefficient)	0.8 (-)
	$K_{\text{int},g}$		10^{-15} m^2	c	0.5 MPa
	η		0.5	ϕ	20°

and a second one on the bottom boundary. The gallery is assumed to have a 5m diameter and its length is split into 45m of backfill on the left and 5m of bentonite on the right. Above the gallery is a concrete lining with a thickness of 1m, then an EDZ with a thickness of 3m and, finally, the host rock with a thickness of 16m. Regarding the mesh, each element does not have the same width or height. First, it becomes denser in the x-direction as it approaches the limit between bentonite and backfill. Then, in the y-direction, the mesh becomes progressively denser when going toward three locations: at the limit between backfill/bentonite and concrete, at the limit between concrete and EDZ, and at the limit between EDZ and host rock. Unlike the previous cases, a distributed stress of 12.4MPa, represented in red in Figure 5.2, is applied at the top of the host rock and under the EDZ. Moreover, a contact element is placed at the interface between concrete and bentonite/backfill.

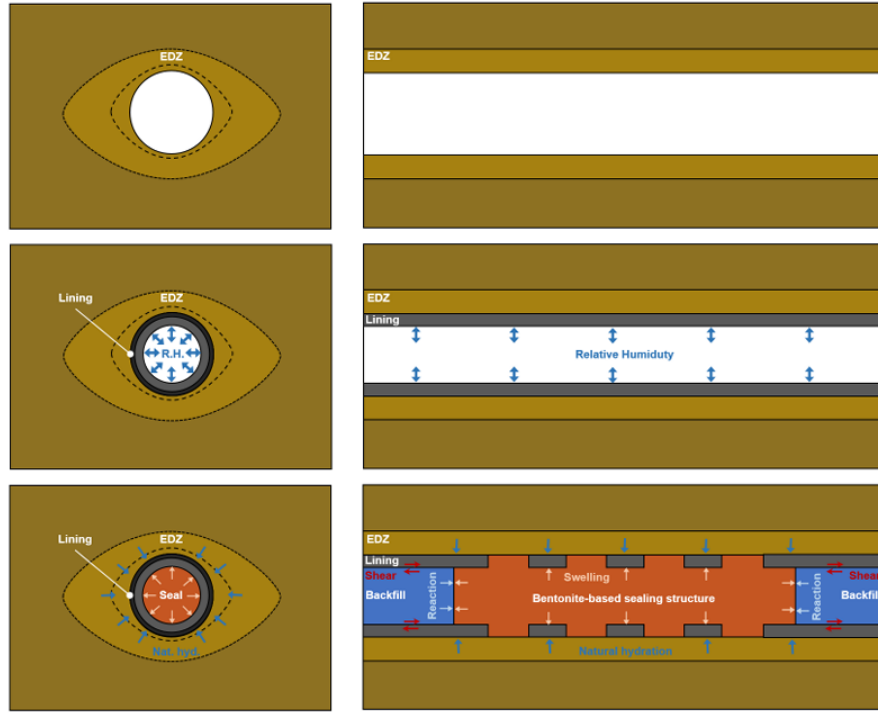


Figure 5.3: Phases undergone by the gallery from Corman et al. 2025b

In terms of initial conditions, the water pressure starts at 5MPa in the host rock and in the EDZ, at 0MPa in the concrete lining, at -4.3MPa in the backfill, and at -69.9MPa in the bentonite. There are initial stresses only in the host rock and the EDZ, where $\sigma_{x,0} = \sigma_{y,0} = -9.58\text{MPa}$. For the boundary conditions, the water pressure is kept constant at its initial value of 5MPa at the top of the host rock. Mechanically, supports prevent normal displacements of the boundary of the sample at the left and top boundaries.

Furthermore, test case 3 will undergo different phases during the simulation, represented in Figure 5.3, with the objective of being more representative of the evolution of the actual sealing structure. Accordingly, modifications of the boundary conditions will be necessary.

Firstly, an excavation phase will be modelled. During the latter, the bentonite, backfill, and concrete elements will be deactivated, the initial distributed force applied under the EDZ will be progressively reduced to simulate the ongoing excavation, and the water pressure at the bottom boundary of the EDZ will also be reduced, from its initial value to 0.1MPa after 16 days. This phase will last 50 days.

Secondly, a phase of drainage, lasting from 50 days to 100 years, will be simulated. It consists of the addition of the concrete lining, which is activated during this phase. At this point, the gallery only acts as a connection gallery. This phase is important due to the long term creep behaviour and the hydro-mechanical evolution toward equilibrium (M. Alonso et al. 2024, Corman et al. 2025b). The only variation of boundary condition is that the decreasing water pressure is no longer imposed beneath the EDZ, but beneath the concrete liner, where the water pressure is kept constant at the initial value of 0MPa , which is also the initial value of the water pressure in the concrete, during the 100-year period.

Thirdly, a saturation phase is modelled. In the latter, the bentonite and backfill cores are activated, meaning that the structure is in its final form. This phase starts at 100 years and is simulated until 10,000 years. During the latter, no boundary condition is applied on the water

pressure, except at the top where the 5MPa pressure is kept constant. Moreover, a phase of gas release should also be modelled before the saturation phase; however, since gas pressure is assumed constant in this work, this phase is neglected.

The results will be displayed at different measurement points in each material, as shown in Figure 5.4.

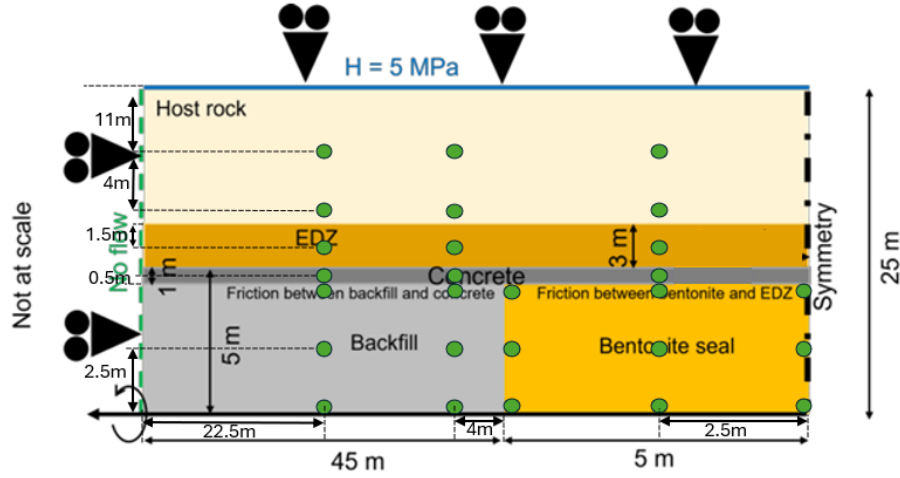


Figure 5.4: Measurement points for the full test case

5.3 Results

In this section the results will be presented for each phase.

5.3.1 Excavation phase

As explained earlier, during the first phase (0 to 50 days), only the host rock and the EDZ are activated during the simulation. To begin with, the water pressure at the end of the phase is presented in Figure 5.5. Compared to the initial value of $u_w = 5\text{MPa}$, it shows that the water

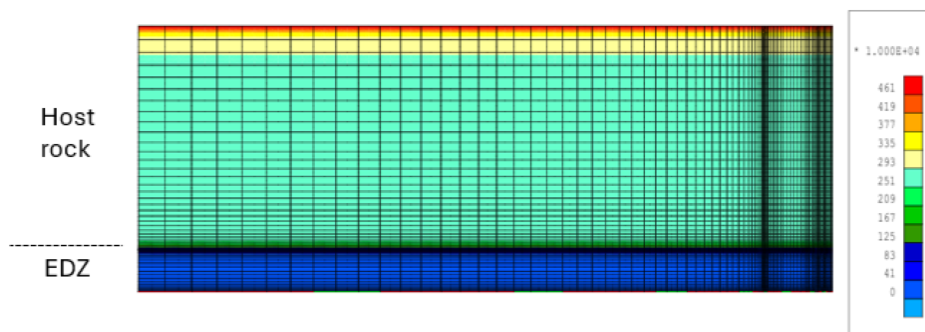


Figure 5.5: Water pressure at 50 days

pressure decreased in the entire sample, due to the bottom boundary condition, which imposes a water pressure that progressively decreases toward 0.1MPa , meaning that the rest of the sample tends to converge toward this value. However, the other boundary condition at the top of the sample imposes a constant head of 5MPa , to simulate an inlet of water from the higher

layers of rocks. Thus, the top boundary induces an increase in water pressure, explaining why higher values of water pressure are still present at the top of the sample. Figure 5.6 presents the temporal evolution of the water pressure at the measurement points of the rock and the EDZ. It can be seen that, initially, the water pressure slightly decreases in the rock, while it

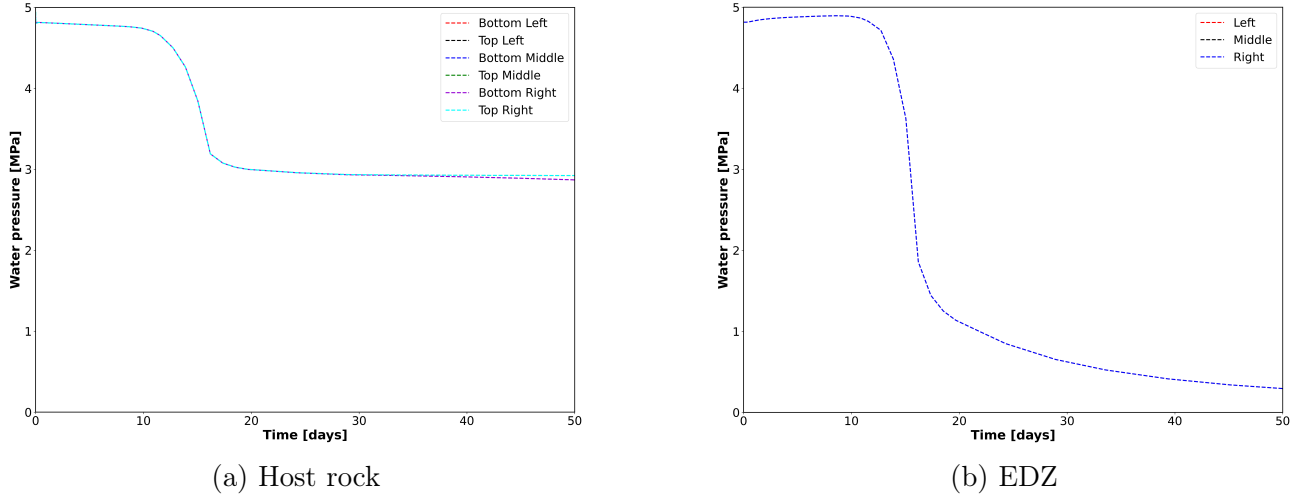


Figure 5.6: Full test case - Temporal evolution of the water pressure

increases in the EDZ. As the water pressure is imposed to decrease due to the imposed boundary condition, this slight increase is probably due to hydro-mechanical coupling. Then, the impact of the excavation starts to be visible after about ten days, where an important decrease occurs, decrease which is much more important in the EDZ, still because of the difference in permeability with the host rock. Of course, the closer the point is to the excavation, the lower its water pressure gets, as can be seen in Figure 5.6a, where the curves corresponding to the top and bottom points split at the end of the first phase. In fact, drainage of the rock above the gallery has already begun.

As the water pressure remains positive, the sample remains completely saturated. Accordingly, the evolution of the mean total stress, shown in Figure 5.7, presents an evolution similar to that of the water pressure, except that it starts at a higher initial value, equal to the initial stress (9.58MPa) plus the initial water pressure (5MPa).

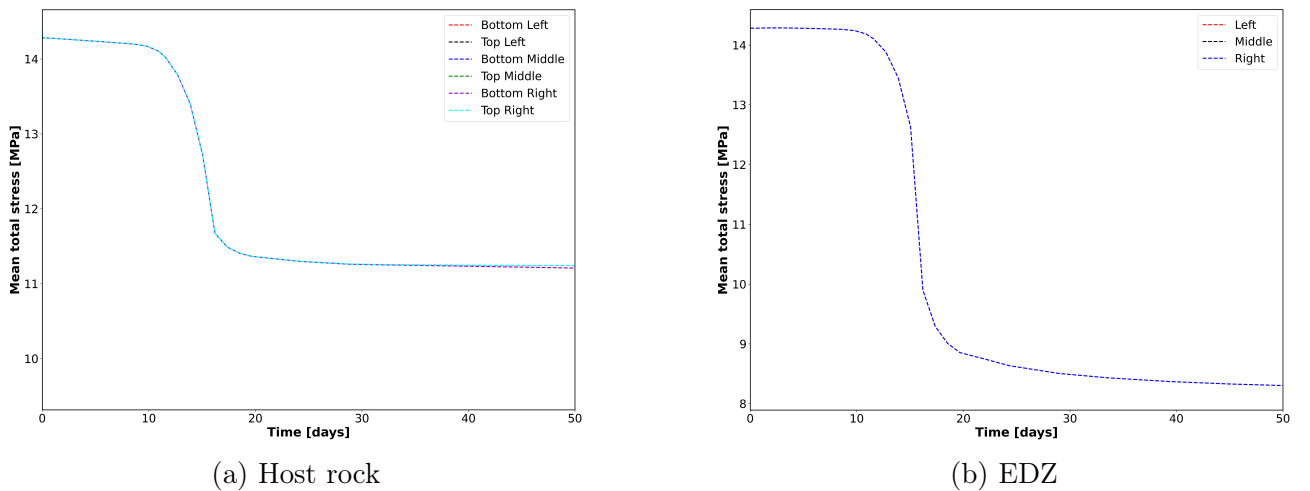


Figure 5.7: Full test case - Evolution of the mean total stress

5.3.2 Drainage phase

Next, the concrete liner is added during the drainage phase (50 days to 100 years). Once more, the water pressure in the entire sample is presented in Figure 5.8, at 100 days and at 100 years. First, it shows that 50 days after the concrete liner has been placed, the situation has not

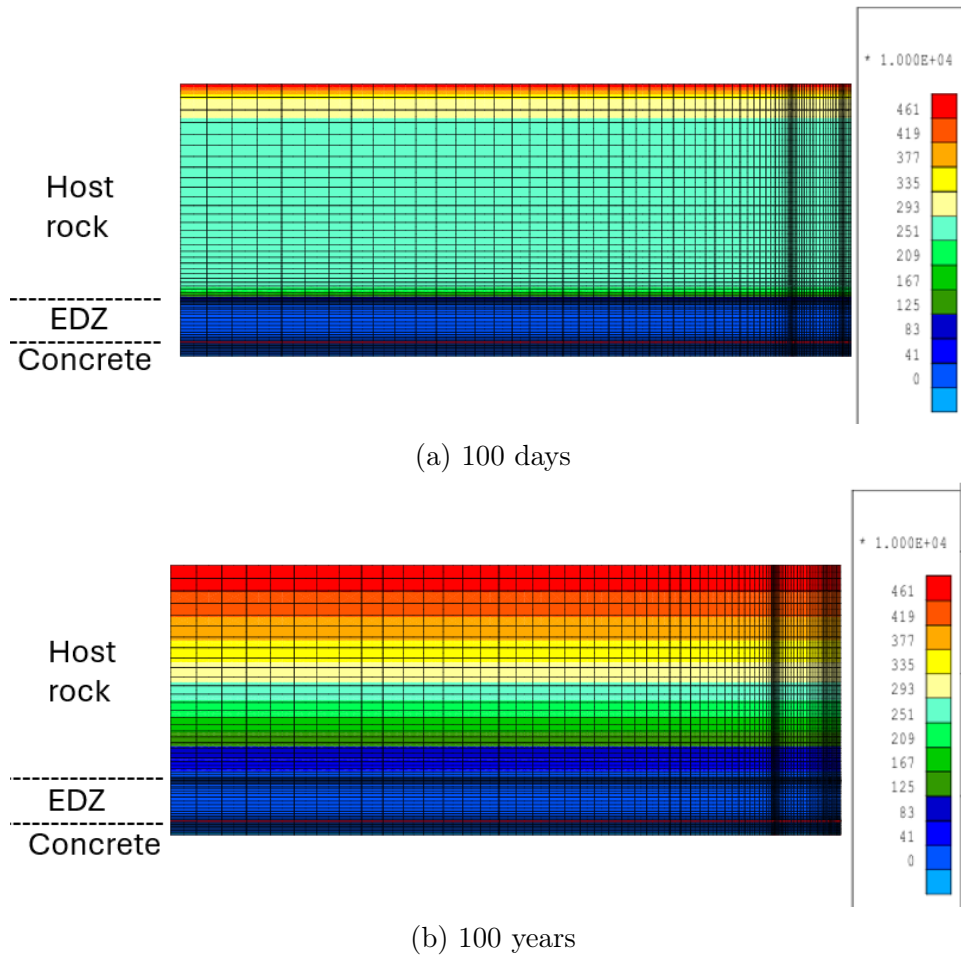


Figure 5.8: Full test case - Water pressure in the sample during phase 2

evolved much. Indeed, compared to Figure 5.5, only small differences are visible, such as the fact that the front coming from the top boundary has propagated slightly further. However, after 100 years, this propagation is much more visible, especially in the host rock. In the EDZ and concrete, the water pressure logically remains close to the value of 0 MPa , which is imposed at the bottom boundary. This can be seen in Figure 5.9, which depicts the temporal evolution of the water pressure in the host rock, EDZ, and concrete. It confirms that the drainage of the gallery continued once the concrete liner was added, which corresponds to the start of the drop in water pressure in the host rock, visible in Figure 5.9a. Moreover, the difference in water pressure based on the radius location of the measurement points is much more visible now that a longer time has elapsed. In the EDZ, the water pressure was already close to 0 MPa at the start of the second phase. Hence, the drop to 0 MPa occurs very early, and is only slightly visible at the very start of Figure 5.9b. In the concrete, the initial value of the water pressure is already equal to 0 MPa and the boundary condition at the bottom of the concrete liner imposes that it remains at that value.

In terms of stresses, the evolution of the mean total stress remains similar to the evolution of the water pressure, as shown in Figure 5.10. Indeed, the mean total stress has continued to decrease compared to the initial situation, and reached only about 7 MPa at the lowest mea-

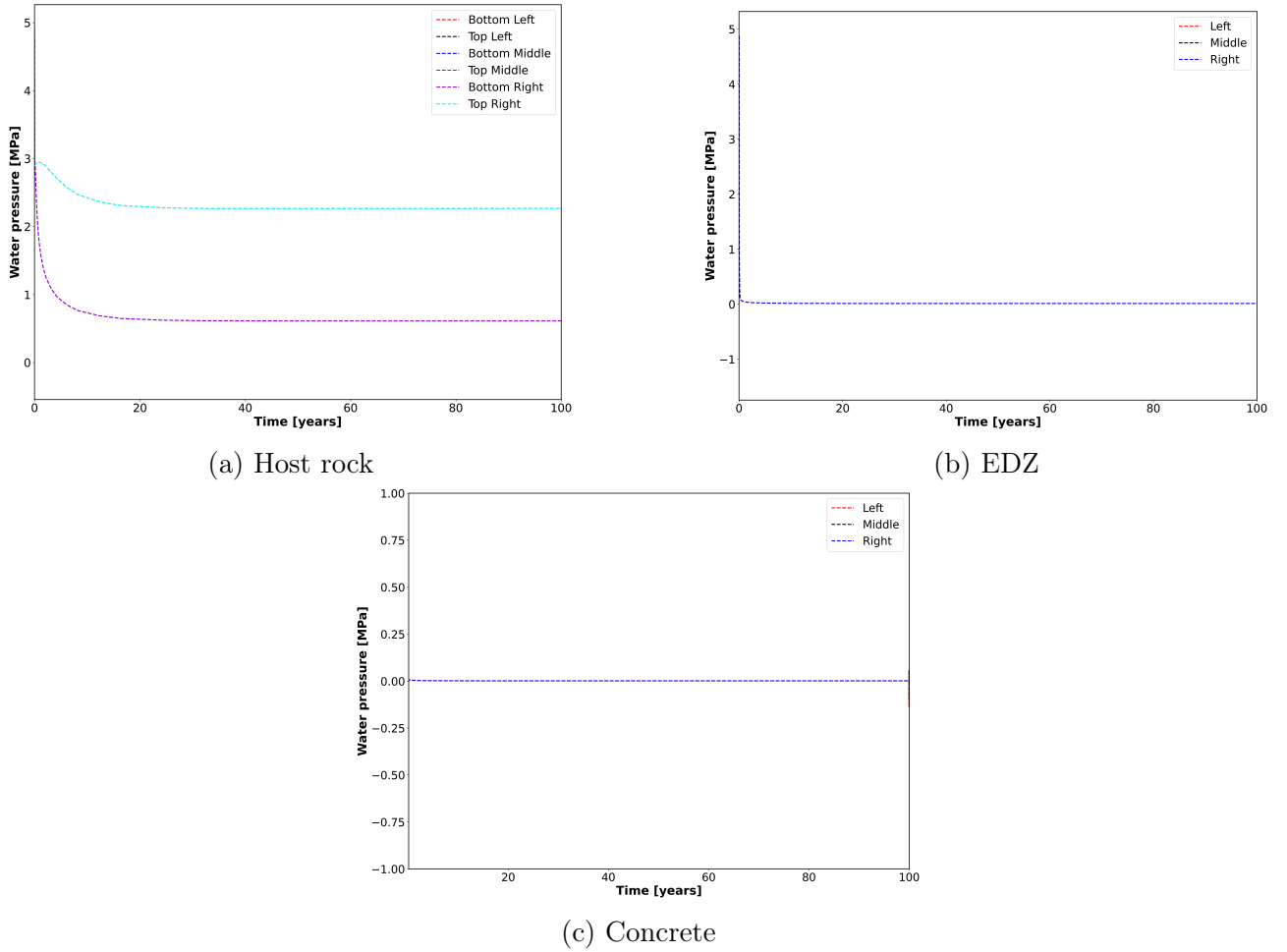


Figure 5.9: Full test case - Temporal evolution of the water pressure during phase 2

surement points, in the concrete.

5.3.3 Saturation phase

Finally, the backfill and the bentonite cores can be added to the simulation, to simulate the saturation phase (100 years to 10,000 years). To begin with, the water pressure in the sample is presented at different times in Figure 5.11. First, Figure 5.11a shows that 198 years after the implementation of backfill and bentonite, the water pressure has decreased in the host rock, because of the saturation process occurring in bentonite and backfill. Of course, as highlighted in the previous chapters, the saturation process is faster in the backfill sample due its higher permeability, inducing the propagation of a second water front in the bentonite core: a first water front comes from the concrete liner and a second comes from the backfill sample. Then, Figures 5.11b, 5.11c, 5.11d, and 5.11e show that the saturation process of the bentonite and backfill cores continues, inducing progressive increases in the water pressure of the entire sample. After 1,000 years, only slight negative water pressures are still present, and the upper part of the host rock is back at its initial value of $u_w = 5 \text{ MPa}$, thanks to the constant water pressure applied at the top boundary. Finally, after 1,747 years, the entire sample, including the bentonite and backfill cores, has stabilised at 5 MPa , reaching equilibrium with the surrounding rock.

More precisely, Figure 5.12 shows the temporal evolution at the measurement points. It confirms that each core evolves toward the initial pressure of the host rock. While the water pressure in the bentonite and backfill cores directly increases at the start of the third phase,

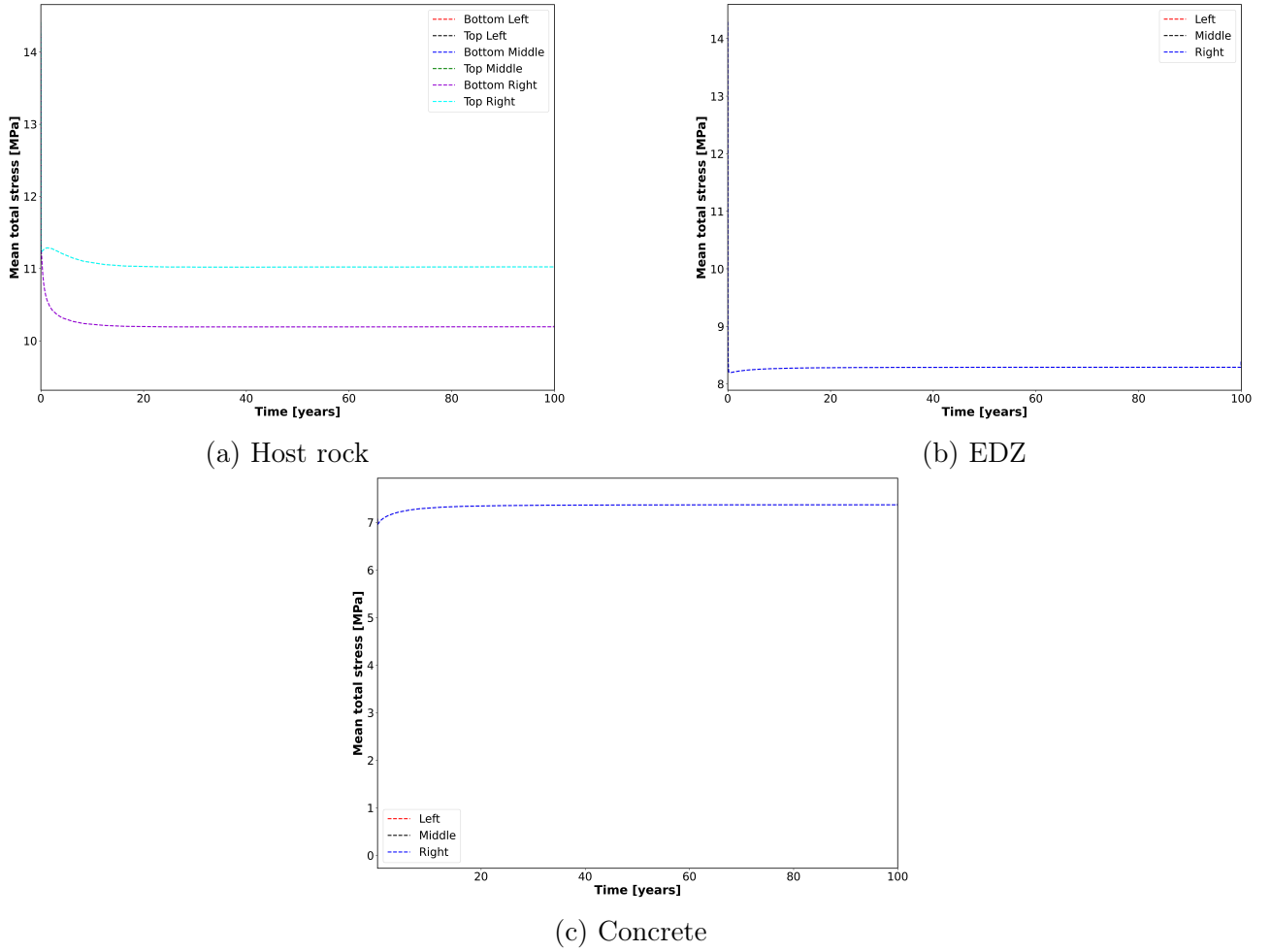
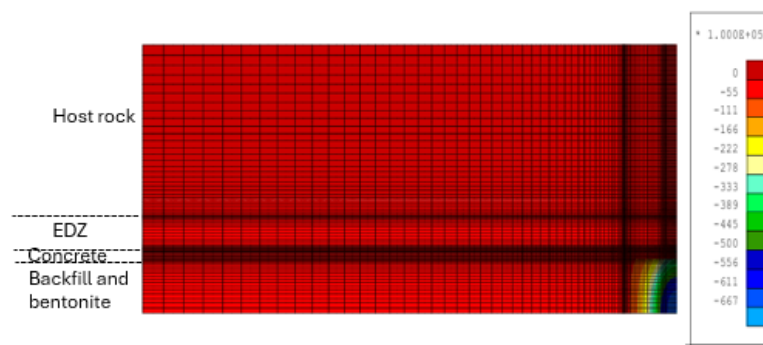


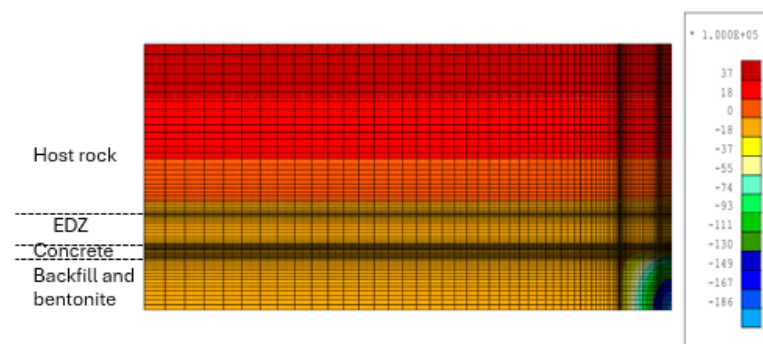
Figure 5.10: Full test case - Temporal evolution of the mean total stress during phase 2

it can be seen that it decreases in the concrete and host rock, because they both lose water to bentonite and backfill. It is also visible in Figure 5.12d that the lower a point is located, the lower its water pressure gets, because of the proximity to bentonite and backfill. Moreover, a small difference is also visible between the points of a same material depending on whether they are located above the bentonite core or the backfill core, especially in the concrete liner which is closer to them, because of the slower re-saturation of bentonite. Accordingly, the water pressure at the right measurement points decreases at least to a lower value than the middle and left points. Additionally, a second smaller increase appears around 1,250 years in Figure 5.12a, corresponding to the point at which saturation is reached in the bentonite sample, as shown in Figure 5.13. Indeed, Figure 5.13a shows that complete saturation of the bentonite sample occurs after around 1,200 years, slightly before the second increase in water pressure. Furthermore, while the host rock remains saturated during the whole process (Figure 5.13d), the degree of saturation of the concrete liner (Figure 5.13c) decreases at the beginning of the phase as it loses water to bentonite and backfill, before re-increasing toward saturation when the bentonite core can no longer receive water. It can also be seen in Figure 5.13b that, despite its higher permeability, backfill does not reach saturation before bentonite because its water propagates toward the bentonite core as long as the latter is not saturated.

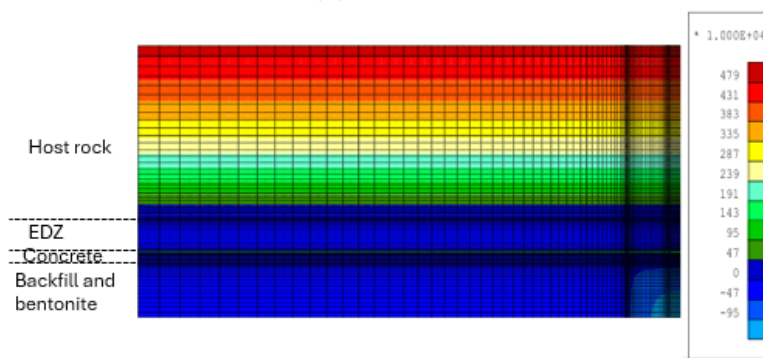
Next, Figure 5.14 shows the evolution of the mean total stress. It shows that, in bentonite and backfill, a first increase occurs during the saturation process. Then, when saturation is reached at about 1,200 years, a more pronounced increase develops, corresponding to the increase in water pressure when the sample converges toward the initial value of $u_w = 5 \text{ MPa}$. At most,



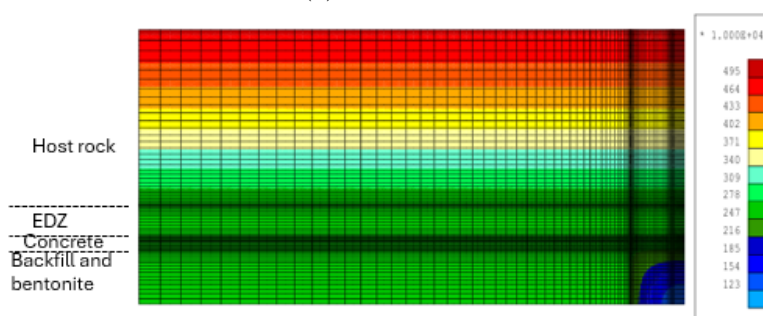
(a) 298 years



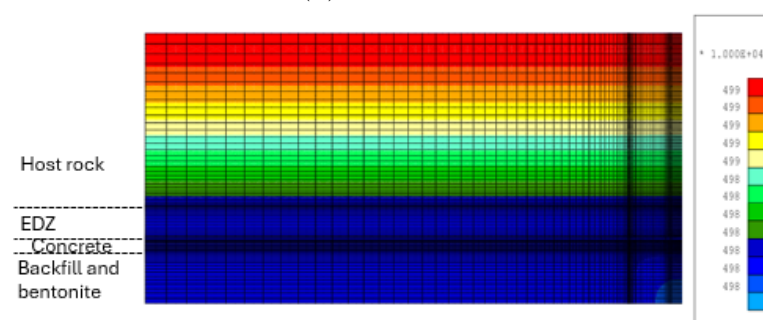
(b) 600 years



(c) 1,000 years

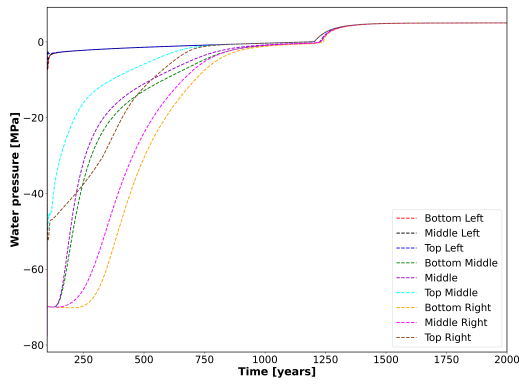


(d) 1,250 years

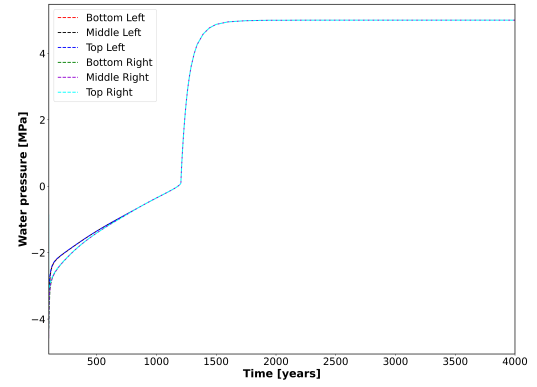


(e) 1747 years

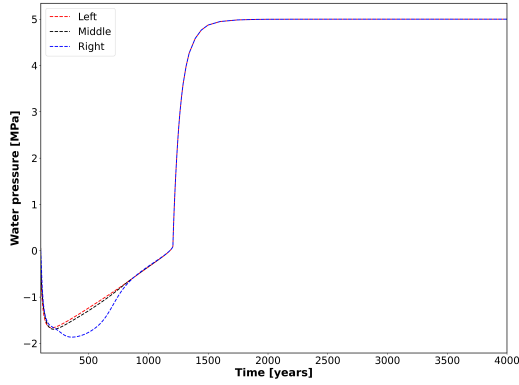
Figure 5.11: Full test case - Water pressure in the sample during phase 3



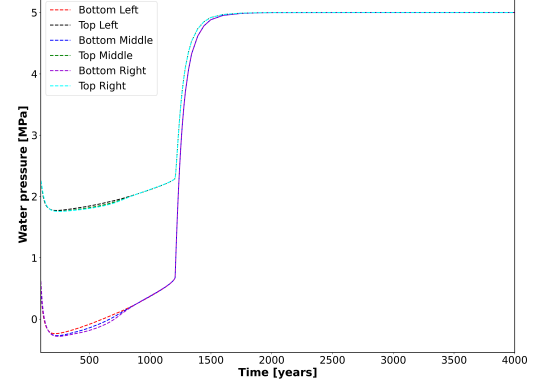
(a) Bentonite



(b) Backfill

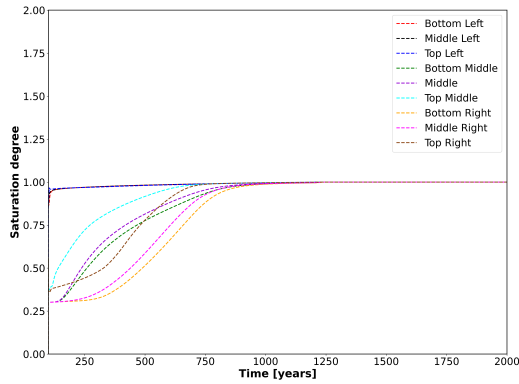


(c) Concrete

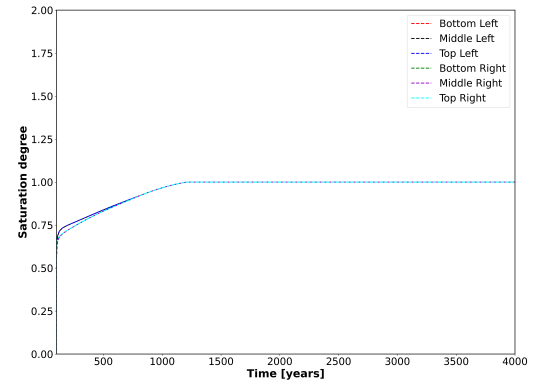


(d) Host rock

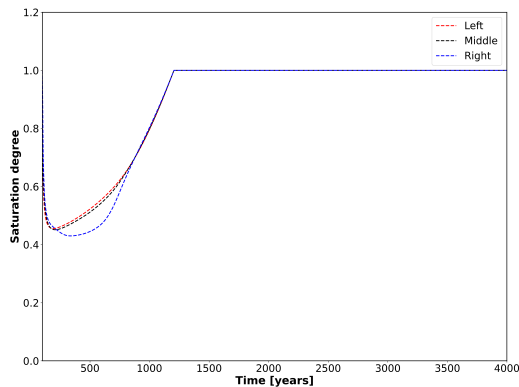
Figure 5.12: Full test case - Temporal evolution of the water pressure during phase 3



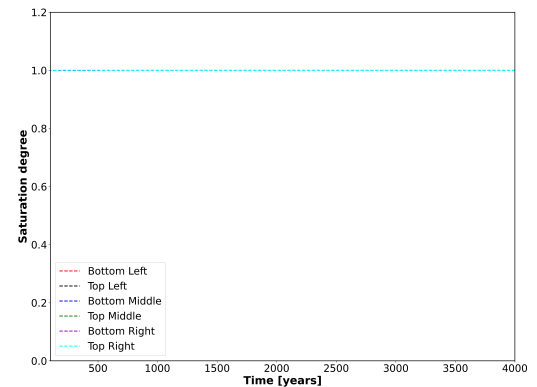
(a) Bentonite



(b) Backfill

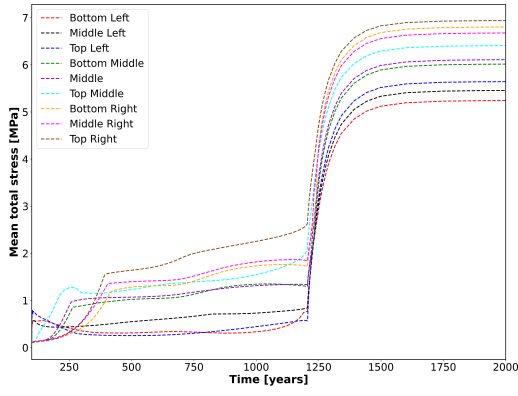


(c) Concrete

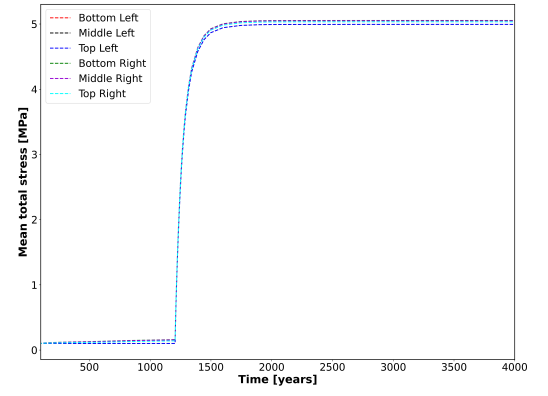


(d) Host rock

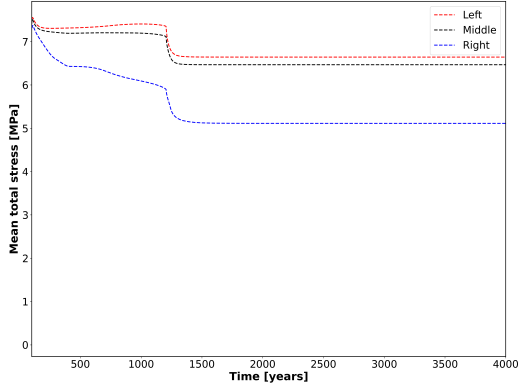
Figure 5.13: Full test case - Temporal evolution of the degree of saturation during phase 3



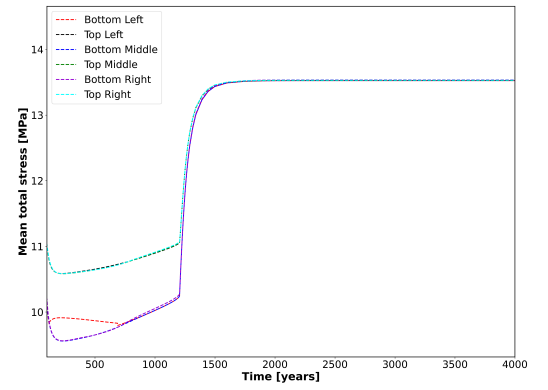
(a) Bentonite



(b) Backfill



(c) Concrete



(d) Host rock

Figure 5.14: Full test case - Temporal evolution of the mean total stress during phase 3

the mean total stress in bentonite is equal to 7MPa , mostly because of the water pressure, which is equal to 5MPa at this stage. In the backfill core, the stress is even lower, reaching only a bit more than 5MPa at most. In the concrete, a different process occurs. At the beginning of phase 3, a small decrease occurs, from 100 to about 500 years, corresponding to the decrease in the degree of saturation as water moves toward the bentonite/backfill core. Then, the concrete liner re-saturates, which leads to an increase in water pressure. However, at the same moment, the mean net stress also decreases in the concrete liner. In total, the mean total stress undergoes a reduction to finally reach a minimum value of about 5MPa .

It can also be seen that, in bentonite, the mean total stress is getting lower when going toward the left part of the sample. This can be explained by the swelling of bentonite, shown in Figure 5.15. Indeed, it shows that bentonite experiences a maximum swelling of 0.35m at the bottom left measurement point. However, among the left measurement points, the top one presents a swelling lower than that of the middle and right points. This can be explained by the fact that the friction between bentonite and the concrete liner acts against the swelling of bentonite, explaining why the three top points present the lowest swelling among the nine measurement points. In terms of swelling pressure, bentonite experiences axial stresses of about 5MPa at the left measurement points, as shown in Figure 5.16. As explained in the previous chapters, the axial stresses are lower at the left points, where the maximum swelling is experienced. Moreover, Figure 5.16b shows that the swelling pressure applied at the interface with the concrete liner is larger, as it is worth about 6MPa . While these pressures are a bit higher than what was found in the second test case, they remain lower than for the initial pure MX-80 bentonite.

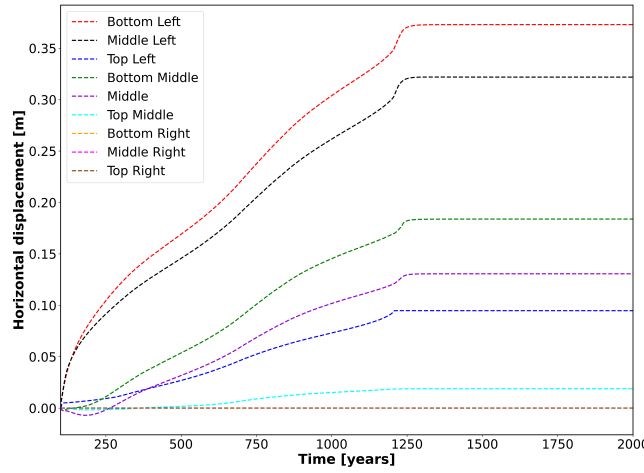
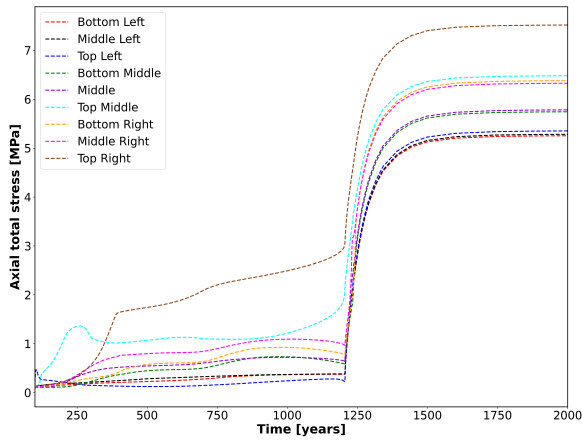
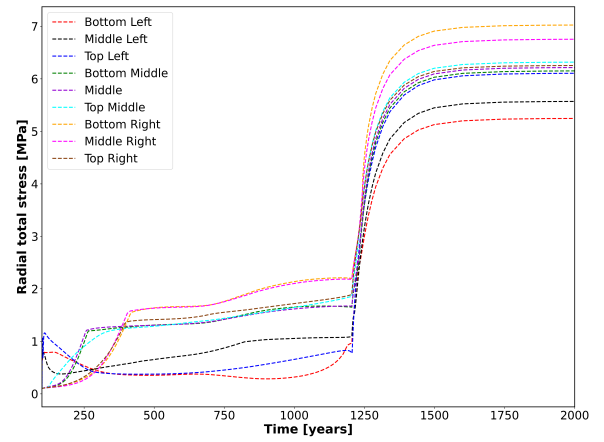


Figure 5.15: Full test case - Horizontal displacement in bentonite



(a) Axial total stress



(b) Radial total stress

Figure 5.16: Full test case - Axial and radial total stresses in the bentonite sample during phase 3

5.4 Preliminary conclusion

In this chapter, each element of the full gallery has been modelled and, for the first time in this work, different phases of the lifetime of the gallery have been simulated. This has made it possible to highlight the different process occurring in terms of water propagation: first, the medium is drained as the excavation works are going on. Then, the drainage continues after the concrete liner has been installed. Finally, after the addition of the engineered materials, re-saturation of the medium occurs, thanks to the surrounding rock, which acts as an inexhaustible source. In total, water pressure returned to its initial state ($u_w = 5MPa$) in about 1,500 years.

Regarding the swelling of the bentonite core, the final values remained in the same order of magnitude to what had previously been found in Chapter 4, with a maximum swelling of $0.35m$. The swelling pressures, applied on the interface with backfill and concrete, also remain in the same order of magnitude as before, with a maximum axial total stress of $5MPa$ and a maximum radial stress of $6MPa$.

Chapter 6

Conclusion

To conclude this work, a review of the different chapters is proposed, followed by a discussion of the study's limitations and directions for future work.

Firstly, Chapter 1 introduced the context of this work, beginning with the growing role of nuclear energy in the global energy transition, the associated production of high level radioactive waste, and the current preferred solution to manage this waste, namely deep geological disposal systems. Then, the general characteristics of these types of systems were presented, followed by a focus on the Belgian and French deep geological disposal concepts. Next, the emphasis was placed on the main difference between the latter, namely the presence of bentonite based materials in the sealing structure. Finally, the objective of this work has been presented, along with the choice of the finite element code **LAGAMINE** as the numerical tool to tackle the complex coupled hydro-mechanical phenomena occurring in bentonite.

Secondly, Chapter 2 provided a review of the existing literature. It started with a presentation of each material of both the natural and engineered barrier systems, with further details about generalities of these materials, but also about their mechanical and hydraulic parameters. Next, numerical models, that would be later used, have been presented, starting with the balance equations of the numerical model and the associated constitutive equations for each type of material. In particular, a further description of the bentonite mechanical model, the Barcelona Basic Model, has been realised in order to better understand the evolution of bentonite upon hydration. Finally, examples of existing numerical simulations have been outlined.

Thirdly, Chapter 3 was the first step toward the modelling of the final sealing structure. As a beginning, the study of a small scale sample of bentonite only was proposed. After a quick presentation of the European partnership EURAD2, which is the source of the exercises employed in this work, the first models were analysed. In particular, the calibrated parameters of the mechanical and hydraulic models used for the MX-80 bentonite were analysed followed by the study of a reference configuration at constant volume, and a configuration with the addition of technological gaps. Lastly, a sensitivity analysis of the BBM parameters have been proposed, enabling a better understanding of the impact of these parameters, especially on the stress path and on the mean total stress development.

Fourthly, Chapter 4 proposed the study of models that are closer to reality. Indeed, the first exercise introduced a larger geometry, with boundary conditions aiming to mimic the actual situation: the presence of a spring at the top boundary, that can be assimilated to the presence of backfill and that allows the bentonite core to swell, and the presence of deposition walls, to reproduce friction with the concrete liner. Then, the second exercise introduced material heterogeneity, with the implementation of backfill on top of the bentonite sample. For both test

cases, the water inflow have also been modified, going from being distributed along the entire height of the sample to being only localised along the height of the bentonite core, to mimic the presence of preferential fluid pathways in the host rock. These exercises led to the modifications of the type of bentonite due to the too high swelling pressures developing. Indeed, the pure MX-80 bentonite was replaced by a mixture of 20% of powder and 80% of pellets. They also led to a first quantification of the development of different variables, such as the suction, the radial and axial stresses, or the swelling experienced by the two materials. In particular, the swelling stresses turned out to remain within targeted values for the URL site of Bure.

Finally, Chapter 5 marked the culmination of the work by modelling the complete sealing structure. The host rock, the EDZ, and the concrete were added to the model. Furthermore, different phases of the lifetime of the gallery were also modelled, starting with an excavation phase, then a drainage phase, and lastly a saturation phase. This chapter highlighted how different variables evolve during the three phases, such as the water pressure, the mean total stress, or the degree of saturation in each material. It also led to the conclusion that the swelling or the axial/radial stresses that develop in the bentonite core remain consistent with what has been found in the previous chapter.

6.1 Limitations and perspectives

Of course, several limitations of this work must be highlighted. For instance, some degrees of freedom were fixed: the temperature and the gas pressure were assumed constant. However, both of them play a key role in the behaviour of sealing structures. Indeed, the introduction of high level waste in the system induces an increase in the surrounding temperature, which has an impact on the behaviour of the materials. Regarding gas pressure, a key aspect of the sealing structure is to allow the dissipation of gas despite its low permeability. Moreover, only 2D axisymmetric models were employed in this work which does not allow to capture the three-dimensional effects that occur in the structure. Another remark can be made on the calibration proposed in Chapter 3. Indeed, more complete laboratory tests could have been done, especially for the stress path which was based on only a few data points. Additionally, this work did not study the impact of the removal of a portion of the concrete liner, that ensures a good contact between the bentonite core and the host rock.

Accordingly, these limitations naturally open up different possibilities for future work, such as the study of the evolution of gas pressures or temperature in the structure, the realisation of a 3D model to tackle three-dimensional effects, more complete laboratory tests to allow for a better comparison with the numerical results, or a model with the removal of portions of the concrete liner. Indeed, all these perspectives would make it possible to better understand the actual behaviour of the sealing structure.

Bibliography

- Alonso, E E, Gens, A, and Josa, A (1990). “A constitutive model for partially saturated soils”. In: *Géotechnique* 40.3, pp. 405–430.
- Alonso, Matías, Vaunat, Jean, Vu, Minh Ngoc, Talandier, Jean, Olivella, Sebastià, and Gens, Antonio (June 2024). “Three-Dimensional Modelling of a Large-Diameter Sealing Concept in a Deep Geological Radioactive Waste Disposal”. In: *Rock Mechanics and Rock Engineering* 57.6, pp. 4133–4158. ISSN: 1434453X. DOI: 10.1007/s00603-024-03813-w.
- ANDRA (2005). The Andra Publication Series Dossier 2005 Evaluation of the feasibility of a geological repository in an argillaceous formation Meuse/Haute-Marne site. Tech. rep. Agence nationale pour la gestion des déchets radioactifs. URL: www.andra.fr.
- ANDRA (2026). Inventaire. URL: <https://www.andra.fr/les-dechets-radioactifs/tout-comprendre-sur-la-radioactivite/inventaire>.
- Armand, G., Leveau, F., Nussbaum, C., De La Vaissiere, R., Noiret, A., Jaeggi, D., Landrein, P., and Righini, C. (Jan. 2014). “Geometry and properties of the excavation-induced fractures at the meuse/haute-marne URL drifts”. In: *Rock Mechanics and Rock Engineering* 47.1, pp. 21–41. ISSN: 07232632. DOI: 10.1007/s00603-012-0339-6.
- Calvin, Katherine, Dasgupta, Dipak, Krinner, Gerhard, Mukherji, Aditi, Thorne, Peter W., Trisos, Christopher, Romero, José, Aldunce, Paulina, Barrett, Ko, Blanco, Gabriel, Cheung, William W.L., Connors, Sarah, Denton, Fatima, Diongue-Niang, Aïda, Dodman, David, Garschagen, Matthias, Geden, Oliver, Hayward, Bronwyn, Jones, Christopher, Jotzo, Frank, Krug, Thelma, Lasco, Rodel, Lee, Yune-Yi, Masson-Delmotte, Valérie, Meinshausen, Malte, Mintenbeck, Katja, Mokssit, Abdalah, Otto, Friederike E.L., Pathak, Minal, Pirani, Anna, Poloczanska, Elvira, Pörtner, Hans-Otto, Revi, Aromar, Roberts, Debra C., Roy, Joyashree, Ruane, Alex C., Skea, Jim, Shukla, Priyadarshi R., Slade, Raphael, Slangen, Aimée, Sokona, Youba, Sörensson, Anna A., Tignor, Melinda, Vuuren, Detlef van, Wei, Yi-Ming, Winkler, Harald, Zhai, Panmao, Zommers, Zinta, Hourcade, Jean-Charles, Johnson, Francis X., Pachauri, Shonali, Simpson, Nicholas P., Singh, Chandni, Thomas, Adelle, Totin, Edmond, Alegría, Andrés, Armour, Kyle, Bednar-Fiedl, Birgit, Blok, Kornelis, Cissé, Guéladio, Dentener, Frank, Eriksen, Siri, Fischer, Erich, Garner, Gregory, Guivarch, Céline, Haasnoot, Marjolijn, Hansen, Gerrit, Hauser, Mathias, Hawkins, Ed, Hermans, Tim, Kopp, Robert, Leprince-Ringuet, Noémie, Lewis, Jared, Ley, Debora, Ludden, Chloé, Niamir, Leila, Nicholls, Zebedee, Some, Shreya, Szopa, Sophie, Trewin, Blair, Wijst, Kaj-Ivar van der, Winter, Gundula, Witting, Maximilian, Birt, Arlene, and Ha, Meeyoung (July 2023). IPCC, 2023: Climate Change 2023: Synthesis Report. Contribution of Working Groups I, II and III to the Sixth Assessment Report of the Intergovernmental Panel on Climate Change [Core Writing Team, H. Lee and J. Romero (eds.)]. IPCC, Geneva, Switzerland. Tech. rep.

- Intergovernmental Panel on Climate Change. DOI: 10.59327/IPCC/AR6-9789291691647. URL: <https://www.ipcc.ch/report/ar6/syr/>.
- Cerfontaine, B., Dieudonné, A. C., Radu, J. P., Collin, F., and Charlier, R. (Sept. 2015). “3D zero-thickness coupled interface finite element: Formulation and application”. In: *Computers and Geotechnics* 69, pp. 124–140. ISSN: 18737633. DOI: 10.1016/j.compgeo.2015.04.016.
- Charlier, Robert, Collin, Frédéric, Pardoën, Benoît, Talandie, Jean, Radu, Jean-Pol, and Gerard, Pierre (2013). “An unsaturated hydro-mechanical modelling of two in-situ experiments in Callovo-Oxfordian argillite”. In: *Engineering Geology* 165, pp. 46–63. DOI: 10.1016/j.enggeo.2013.05.021.
- Chen, Yang, Cui, Yu Jun, and Molinero Guerra, Agustin (2018). “Compression Behavior of Bentonite Powder/Pellet Mixture”. In: *Proceedings of GeoShanghai 2018 International Conference: Multi-p* Springer, pp. 39–46. ISBN: 9789811300943. DOI: 10.1007/978-981-13-0095-0\4.
- Cohen, Kim, Harper, David, Gibbard, Philip, and Car, Nicholas (Mar. 2025). “The ICS international chronostratigraphic chart this decade”. In: *Episodes* 48.1, pp. 105–115. ISSN: 07053797. DOI: 10.18814/epiiugs/2025/025001.
- Conil, N., Talandier, J., Djizanne, H., La Vaissière, R. de, Righini-Waz, C., Auvray, C., Morlot, C., and Armand, G. (Aug. 2018). “How rock samples can be representative of in situ condition: A case study of Callovo-Oxfordian claystones”. In: *Journal of Rock Mechanics and Geotechnical Engineering* 10.4, pp. 613–623. ISSN: 16747755. DOI: 10.1016/j.jrmge.2018.02.004.
- Corman, Gilles (2023). “Hydro-Mechanical Modelling of Gas Transport Processes in Clay Host Rocks in the Context of a Nuclear Waste Repository”. PhD thesis. Université de Liège.
- Corman, Gilles and Collin, Frédéric (2025a). Modelling of transfers in backfills: hydration and performance tests of the REG demonstrator in the GET2 gallery. Tech. rep. University of Liège. URL: <https://www.uee.uliege.be/>.
- Corman, Gilles and Collin, Frédéric (2025b). Modelling the hydro-mechanical behaviour of a sealing structure. Tech. rep. University of Liège.
- Delage, Pierre and Cui, Yu Jun (2001). *L’eau dans les sols non saturés*. Tech. rep. Techniques de l’Ingénieur, C301.
- Dieudonné, Anne-Catherine (2016). “Hydromechanical behaviour of compacted bentonite: from micro-scale analysis to macro-scale modelling”. PhD thesis. Université de Liège.
- Distinguin, Marc and Lavanchy, Jean Marc (2007). “Determination of hydraulic properties of the Callovo-Oxfordian argillite at the bure site: Synthesis of the results obtained in deep boreholes using several in situ investigation techniques”. In: *Physics and Chemistry of the Earth* 32.1-7, pp. 379–392. ISSN: 14747065. DOI: 10.1016/j.pce.2006.02.056.
- Dixon, D. A., Stone, J., Birch, K., and Kim, C. S. (July 2023). “Sealing materials for a deep geological repository: evaluation of swelling pressure and hydraulic conductivity data for bentonite-based sealing materials proposed for use in placement rooms”. In: *Canadian Geotechnical Journal* 60.7, pp. 951–965. ISSN: 12086010. DOI: 10.1139/cgj-2021-0609.

- Dueck, Ann and Nilsson, Ulf (2010). Thermo-Hydro-Mechanical properties of MX-80 Results from advanced laboratory tests. Tech. rep. Svensk Kärnbränslehantering AB. URL: www.skb.se.
- Energy Institute (2025). Statistical Review of World Energy 2025 | 74 th edition. Tech. rep. URL: www.energyinst.org/statistical-review.
- EU (2011). Council Directive 2011/70/Euratom of 19 July 2011 establishing a Community framework for the responsible and safe management of spent fuel and radioactive waste. Tech. rep. Brussels, Belgium: European Union.
- European Union (July 2025). European Partnership on Radioactive Waste Management - 2. URL: <https://cordis.europa.eu/project/id/101166718/fr>.
- Galindo, Andrea (Nov. 2025). What is Nuclear Energy? The Science of Nuclear Power. URL: <https://www.iaea.org/newscenter/news/what-is-nuclear-energy-the-science-of-nuclear-power>.
- Gatabin, Claude, Talandier, Jean, Collin, Frédéric, Charlier, Robert, and Dieudonné, Anne Catherine (Mar. 2016). “Competing effects of volume change and water uptake on the water retention behaviour of a compacted MX-80 bentonite/sand mixture”. In: Applied Clay Science 121-122, pp. 57–62. ISSN: 01691317. DOI: 10.1016/j.clay.2015.12.019.
- Gaucher, Eric C, Robelin, Christian, Matray, J M, Negrel, Gabrielle, Gros, Yves, Heitz, J F, Vinsot, Agnès, Rebours, H, Cassagnabère, Alain, and Bouchet, Alain (2004). “ANDRA underground research laboratory: interpretation of the mineralogical and geochemical data acquired in the Callovian-Oxfordian formation by investigative drilling”. In: Physics and Chemistry of the Earth. Parts A/B/C 29.1, pp. 55–77. DOI: 10.1016/j.pce.2003.11.006. URL: <https://brgm.hal.science/hal-00664755v1>.
- Gens, A., Sánchez, M., Guimarães, L. Do N., Alonso, E. E., Lloret, A., Olivella, S., Villar, M. V., and Huertas, F. (May 2009). “A full-scale in situ heating test for high-level nuclear waste disposal: Observations, analysis and interpretation”. In: Geotechnique 59.4, pp. 377–399. ISSN: 00168505. DOI: 10.1680/geot.2009.59.4.377.
- Genuchten, M Th van (1980). “A Closed-form Equation for Predicting the Hydraulic Conductivity of Unsaturated Soils”. In: Soil science society of America journal. 44.5, pp. 892–898. ISSN: 0361-5995. DOI: 10.2136/sssaj1980.03615995004400050002x.
- Hoffmann, C., Alonso, E. E., and Romero, E. (2007). “Hydro-mechanical behaviour of bentonite pellet mixtures”. In: Physics and Chemistry of the Earth 32.8-14, pp. 832–849. ISSN: 14747065. DOI: 10.1016/j.pce.2006.04.037.
- IAEA (2003). Technical Reports Series No. 413, Scientific and Technical Basis for Geological Disposal of Radioactive Wastes. Tech. rep. Vienna, Austria: International Atomic Energy Agency.
- IAEA (2009). IAEA Classification of Radioactive Waste. General Safety Guide. IAEA Safety Standards GSG-1. Tech. rep. Vienna, Austria: International Atomic Energy Agency. URL: <http://www-ns.iaea.org/standards/>.

- IAEA (Sept. 2020). IAEA Bulletin, Nuclear power and the clean energy transition. Tech. rep. URL: www.iaea.org/bulletin.
- IAEA (2021). Energy, electricity and nuclear power estimates for the period up to 2050, Reference data series NO. 1. Tech. rep. Vienna, Austria.
- IAEA (n.d.). Getting to the Core of Radioactive Waste, Managing the by-products of nuclear technologies to protect people and the environment. Tech. rep. Vienna, Austria: International Atomic Energy Agency. URL: www.iaea.org/NuclearEnergy.
- IEA (2025). World Energy Outlook 2025. Tech. rep. International Energy Agency. URL: www.iea.org/terms.
- Lelieveld, J., Kunkel, D., and Lawrence, M. G. (2012). “Global risk of radioactive fallout after major nuclear reactor accidents”. In: Atmospheric Chemistry and Physics 12.9, pp. 4245–4258. ISSN: 16807316. DOI: 10.5194/acp-12-4245-2012.
- Li, Nan, Duan, Xianzhe, and Peng, Guowen (May 2025). Engineering barriers in deep geological disposal: Implications for radioactive nuclide migration and long-term safety. DOI: 10.1016/j.jenvrad.2025.107670.
- Madsen, F. T. (1998). “Clay mineralogical investigations related to nuclear waste disposal”. In: Clay minerals 33, pp. 109–129. URL: <http://pubs.geoscienceworld.org/minersoc/claymin/article-pdf/33/1/109/3307446/109.pdf>.
- Meunier, Alain (2003). Clays. ISBN: 3-540-21667-7.
- OECD NEA (1995). The Environmental and Ethical Basis of Geological Disposal of Long-Lived Radioactive Wastes. Tech. rep. Nuclear Energy Agency. URL: <http://www.copyright.com/>
..
- Ojovan, Michael I. and Steinmetz, Hans J. (Oct. 2022). Approaches to Disposal of Nuclear Waste. DOI: 10.3390/en15207804.
- ONDRAF (2013). ONDRAF Research, Development and Demonstration Plan for the geological disposal of high-level and/or long-lived radioactive waste including irradiated fuel if considered as waste, State of the art report. Tech. rep. Brussels, Belgium.
- ONDRAF (Dec. 2024). Stockage des déchets radioactifs : quelles quantités ? URL: <https://www.ondraf.be/chiffres/quantites-de-dechets-destinees-au-stockage-definitif>.
- Pardoën, B., Talandier, J., and Collin, F. (May 2016). “Permeability evolution and water transfer in the excavation damaged zone of a ventilated gallery”. In: International Journal of Rock Mechanics and Mining Sciences 85, pp. 192–208. ISSN: 13651609. DOI: 10.1016/j.ijrmmms.2016.03.007.
- Philip, J. R. and De Vries, D. A. (1957). “Moisture movement in porous materials under temperature gradients”. In: Eos, Transactions American Geophysical Union 38.2, pp. 222–232. ISSN: 23249250. DOI: 10.1029/TR038i002p00222.

- Pintado, Xavier., Toprak, Erdem., Mokni, Nadia., and Olivella, Sebastia. (2013). Thermo-hydro-mechanical modelling of buffer : synthesis report. Posiva Oy, p. 108. ISBN: 9789516522299.
- Seiphoori, A., Ferrari, A., and Laloui, L. (July 2014). “Water retention behaviour and microstructural evolution of MX-80 bentonite during wetting and drying cycles”. In: *Geotechnique* 64.9, pp. 721–734. ISSN: 17517656. DOI: 10.1680/geot.14.P.017.
- Sellin, P, Pintado, X, Svoboda, J, Miettinen, A, Simo, E, Olivella, S, Talandier, J, Cuisinier, O, Dueck, A, Fabian, M, Gaboreau, S, Kirby, M, Karastanev, D, Leupin, O, Liu, Z, Mayor, J C, Mecová, M, Métral, M, Middelhoff, M, Mokni, M, Najser, J, Nagel, T, Niskanen, M, Pedrott, i M, Pulkkanen, V-M, Stoppe- Struck, N, Vecernik, P, and Villar, M V (Mar. 2025). EURAD-2 Deliverable 10.1-Initial State-of-the-Art on Hydraulic Mechanical Chemical Evolution of Bentonite for Barriers Optimisation. Tech. rep.
- SPF Economie P.M.E., Classes moyennes et Energie (Feb. 2026). Belgian Energy Data Overview. Tech. rep.
- Takada, Momo, Suzuki, Kaoru, Shirai, Kosuke, Biddinger, Elizabeth J, Shrestha, Sujan, and Durrani, Martin (1998). Nuclear Waste, Management, disposal and governance. Ed. by Klaus-Jürgen Röhlig. ISBN: 978-0-7503-3095-4. DOI: 10.1088/978-0-7503-3095-4.
- Talandier, Jean, Simo, Eric, Pintado, Xavier, and Sellin, Patrick (2025). EURAD-2 Milestone 52: Performance Assessment of Bentonite Barriers: Definition of Assessment Cases, Work Package 10. Tech. rep.
- Tang, Anh Minh, Cui, Yu Jun, and Le, Trung Tinh (Oct. 2008). “A study on the thermal conductivity of compacted bentonites”. In: *Applied Clay Science* 41.3-4, pp. 181–189. ISSN: 01691317. DOI: 10.1016/j.clay.2007.11.001.
- Toprak, Erdem, Olivella, Sebastia, and Pintado, Xavier (Feb. 2019). “Modelling engineered barriers for spent nuclear fuel repository using a double-structure model for pellets”. In: *Environmental Geotechnics* 7.1, pp. 72–94. ISSN: 2051-803X. DOI: 10.1680/JENGE.17.00086. URL: <https://www.sciencedirect.com/org/science/article/abs/pii/S2051803X19000374>.
- Toprak, Erdem, Olivella, Sebastia, Pintado, Xavier, and Niskanen, Mika (2024). “3D THM modelling of Finnish spent nuclear fuel repository”. In: *Frontiers in Built Environment* 10. ISSN: 22973362. DOI: 10.3389/fbuil.2024.1465051.
- Toprak, Erdem, Simo, Eric, Talandier, Jean, Olivella, Sebastia, Vaunat, Jean, Ligurský, Tomáš, Narkūnienė, Asta, Poškas, Povilas, Poškas, Gintautas, Dieudonné, Anne-Catherine, Eiza-guirre, Pablo, Kumar, Vinay, Beese, Stefen, Zdravkovic, Lidija, Potts, David, Lai, Han Ming, Iglesia, Miquel de la, Laviña, Marcelo, Idiart, Andrés, Jenni, Andreas, Pulkkanen, Veli-Matti, Nagel, Thomas, Kristensson, Ola, Malmberg, Daniel, Krejci, Tomáš, Kruis, Jaroslav, Mašín, David, Ferrari, Alessio, Métral, Mathilde, Collin, Frédéric, Corman, Gilles, Navarro, Vicente, Yustres, Ángel, Asensio, Laura, López-Vizcaíno, Rubén, and Pfingsten, Wilfried (2025). EURAD-2 Milestone XX: Theoretical Benchmarking Exercises, Work Package 10. Tech. rep.
- Vaníček, Ivan. (2003). Geotechnical problems with man-made and man influenced grounds : proceedings of the XIIIth European conference on soil mechanics and geotechnical engineering : Prague, Czech Republic, 25-28th August 2003 = Les problèmes géotechniques posés

par les remblais et les sols anthropiques : comptes rendus du XIIIème congrès européen de mécanique des sols et de la géotechnique : Prague, République Tchèque, 25-28 août 2003. Czech Geotechnical Society CICE : Czech Chamber of Certified Engineers and Techniciens. ISBN: 808676902X.

Villar, M. V. (2005). MX-80 Bentonite. Thermo-Hydro-Mechanical Characterisation Performed at CIEMAT in the Context of the Prototype Project. Tech. rep. CIEMAT.

Villar, M. V. (June 2007). “Water retention of two natural compacted bentonites”. In: *Clays and Clay Minerals* 55.3, pp. 311–322. ISSN: 00098604. DOI: 10.1346/CCMN.2007.0550307.

Villar, M. V. and Lloret, Antonio (Apr. 2008). “Influence of dry density and water content on the swelling of a compacted bentonite”. In: *Applied Clay Science* 39.1-2, pp. 38–49. ISSN: 01691317. DOI: 10.1016/j.clay.2007.04.007.

Villar, M.V., Besuelle, P., Cernochova, K., Collin, F., Cuevas, J., Cuss, R., Lesquen, C. de, Dizier, A., El Tabbal, G., Gens, A., Gimeno, N., Graham, C., Grgic, D., Harrington, J., Imbert, C., Kašpar, V., Kaufhold, S., Leupin, O., Levasseur, S., Mašin, D., Najser, J., Narkūnienė, A., Ollin, M., Reijonen, H., Šachlová, Š., Sayenko, S., Simo, E., Svensson, D., Svoboda, J., Tatomir, A., Vettese, G., Yliharju, J., and Zoblenco, B. (2024). Eurad Deliverable 7.2 HITEC. Updated State-of-the-Art on THM behaviour of i) Buffer clay materials and of ii) Host clay formations. Deliverable D7.2 HITEC. EURAD Project, Horizon 2020 No 847593. 129 pp. Tech. rep.

Villar, María Victoria (2000). “Thermo-hydro-mechanical characterisation of a bentonite from Cabo de Gata”. PhD thesis. Universidad Complutense de Madrid. URL: www.enresa.es.

Villar, María Victoria (2004). Thermo-Hydro-Mechanical Characteristics and Processes in the Clay Barrier of a High Level Radioactive Waste Repository. State of the Art Report. Tech. rep. CIEMAT.

Wang, Qiong, Tang, Anh Minh, Cui, Yu Jun, Delage, Pierre, Barnichon, Jean Dominique, and Ye, Wei Min (2013). “The effects of technological voids on the hydro-mechanical behaviour of compacted bentonite-sand mixture”. In: *Soils and Foundations* 53.2, pp. 232–245. ISSN: 00380806. DOI: 10.1016/j.sandf.2013.02.004.

Wypych, Fernando and Freitas, Rilton Alves de (Jan. 2022). “Clay minerals: Classification, structure, and properties”. In: *Developments in Clay Science*. Vol. 10. Elsevier B.V., pp. 3–35. DOI: 10.1016/B978-0-323-91858-9.00004-5.

Zhang, Chun Liang, Armand, Gilles, Conil, Nathalie, and Laurich, Ben (Mar. 2019). “Investigation on anisotropy of mechanical properties of Callovo-Oxfordian claystone”. In: *Engineering Geology* 251, pp. 128–145. ISSN: 00137952. DOI: 10.1016/j.enggeo.2019.02.008.

Appendix A

Graphs for Test case 2

Figure A.1 shows the first 20 days of the evolution of the degree of saturation in the bentonite core for test case 2a.

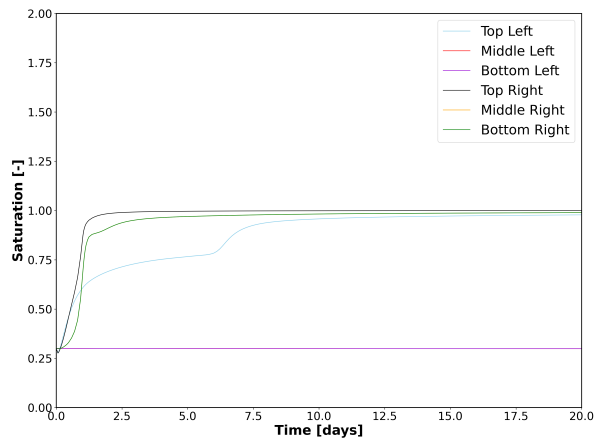


Figure A.1: Case 2a - Bentonite - First 20 days of evolution of saturation

Figure A.2 shows the first 40 days of the evolution of the degree of saturation in the backfill for test case 2b.

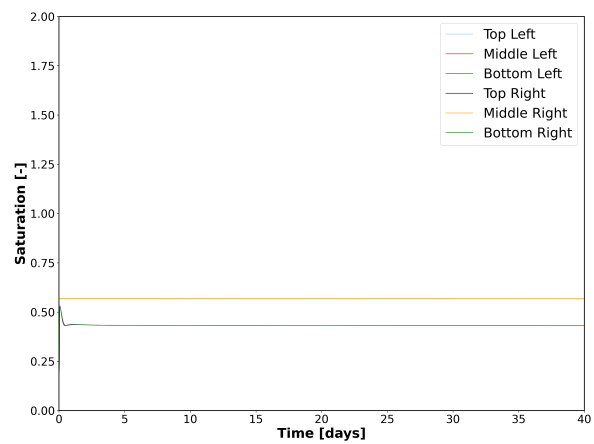


Figure A.2: Case 2b - Backfill - First 40 days of the evolution of the degree of saturation