



A Comparative Analysis of Different Approaches to Large-Scale Modeling for a Radioactive Waste Repository in the Callovo-Oxfordian Claystone and Sensitivity Analysis

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Abstract

This paper aims to present a comparison of several approaches to numerically modeling for a study case of an intermediate-level long-lived radioactive (IL-LLW) repository of the French geological nuclear waste disposal project (Cigéo) within the Callovo-Oxfordian (CO_x) claystone and sensitivity analysis. The IL-LLW repository consists of a tunnel approximately 10 m in diameter, supported by a double liner comprising a 0.5 m concrete support layer and a 0.2 m compressible material layer, ensuring its stability for the operational phase (~100 years). The efforts exerted on the tunnel liner are due to instantaneous response and time-dependent deformation of the host rock, which constitutes the CO_x claystone layer with a thickness of ~150 m and located ~500 m below the ground surface. Deterministic modeling results from seven teams are summarized, wherein the differences between teams consist of mechanical constitutive models representing the behavior of the host rock and the compressible material layer, the multiphysics couplings (e.g., purely mechanical, hydro-mechanical, thermo-hydro-mechanical, two phases hydro-mechanical, etc.), the excavation and installation of structural components stages, the repository configuration, etc. The comparison of teams' results focuses on the evolution of three variables: the maximum absolute (orthoradial) stress in the concrete liner, the maximum absolute radial strain in the compressible layer, and the excavation-induced fractures zone. Several sensitivity analyses were performed to highlight the effects of the time-dependent behavior, the thermal loading, the time of liner installation, the repository configuration, etc.

Highlights

- Comparing the results of several numerical models for an IL-LLW repository from various research teams shows the consistency of different approaches.
- Callovo-Oxfordian creep behavior is the main factor that causes the stress evolution and magnitude in the concrete liner.
- The thermal effect due to the exothermic waste packages is not significant.
- The compressible material layer plays an important role in reducing the stress in the concrete layer.
- The effect of liner installation time depends on the constitutive models describing the CO_x behavior.

Keywords Callovo-Oxfordian · Cigéo project · Deep tunnel · Numerical model · Compressible material · Geological radioactive waste disposal

1 Introduction

The French National Radioactive Waste Management Agency (Andra) is responsible for studying the disposal of high-level and intermediate-level long-lived radioactive

waste (HLW and IL-LLW) in a deep geological repository (Cigéo project), which will be located at the border of the Meuse and Haute-Marne departments, nearly 300 km East of Paris. The host formation consists of the Callovo-Oxfordian (CO_x) claystone, which has a thickness of ~150 m and is situated ~500 m below the ground surface (Plua et al. 2021; Armand et al. 2017a). The CO_x claystone rock exhibits highly favorable conditions for a radioactive waste repository, as it generally processes with very low hydraulic conductivity, low molecular diffusion, and significant retention capacity for radionuclides.

Based on the radioactive waste categories and the excavation method, the Cigéo project is divided into two main zones: HLW and IL-LLW. The IL-LLW zone encompasses large diameter tunnels (~10 m in diameter), also called cells or repositories, which are planned to be dug by TBM with an immediate installation of the liner. The liner of the IL-LLW cells consists of two layers encompassing concrete segments covered by a compressible material layer. The compressible material exhibits a significant deformation (30–70%) under low stress (0.5–2.0 MPa), which allows it to absorb the short-term and time-dependent convergence of the host rock, thereby ensuring the stability of the concrete liner for the operational phase (~100 years, this duration may be adjusted during the project development) (Zghondi et al. 2023). During this period, the IL-LLW repository is subjected to a coupled chemo-thermo-hydro-mechanical (THMC) loading. The excavation first induces the redistribution of the stress and pore pressure, leading to the creation of the fractured zone and the convergence of the tunnel wall (Armand et al. 2013, 2014, 2015, 2023; Vu et al. 2020). The installation of the liner, which may take place before the total unloading of the excavation, leads to an instantaneous increase in stress and strain in the concrete liner and compressive layer (Alonso et al. 2021, 2024; Corman et al. 2022; Camusso et al. 2020, 2022; Tran et al. 2021). The desaturation of the structure and the fractured zone occurs due to the ventilation. Subsequently, the emplacement of the waste packages generates heat emission through the concrete structure to the host rock, inducing thermo-mechanical stresses in the liner and the THM responses in the host rock related to the THM coupling (i.e., thermal pressurization phenomenon due to the low permeability of the CO_x claystone) (Plúa et al. 2023; Conil et al. 2020; Vu et al. 2019; Bumbieler et al. 2021). The chemical effect is negligible before the cessation of ventilation as the cementitious structure remains relatively dry.

To demonstrate the feasibility of constructing and operating the Cigéo project and to enhance its design, Andra is conducting a research program including in situ experiments at the Meuse/Haute-Marne Underground Research Laboratory (MHM URL), laboratory tests at both the sample scale and large-scale structure, as well as theoretical analysis and numerical modeling. The numerical modeling

task involves developing constitutive models for geomaterials (host rock, bentonite, concrete, etc.) and numerical tools to predict the responses of components under the disposal conditions and across various time scales. Based on measurements from laboratory tests and in situ experiments (e.g. Armand et al. 2017b, c; Vaissière et al. 2015; Seyedi et al. 2017), several constitutive models have been proposed to describe the mechanical responses of the CO_x claystone under repository conditions for over 15 years (Souley et al. 2011, 2017a, 2022a, b, 2023a, b; Pardoën and Collin 2017; Yu et al. 2021a, b, 2022; Cuvilliez et al. 2017; Jung et al. 2022; Mánica et al. 2017, 2021a, b; Trivellato et al. 2019; Yao et al. 2017; Zhao et al. 2022). These models can be divided into two main approaches: phenomenological and engineering approaches. The engineering approach consists of introducing the fractured zone into the geometrical model and attributing unfavorable properties to it in comparison to the intact zone to replicate (hydro-) mechanical responses as observed at the MHM URL (Vu et al. 2020; Saitta et al. 2017). The phenomenological approach signifies the development of constitutive rheological models based on laboratory tests and in situ observations (convergence, excavation-induced fractured zone, permeability change, pore pressure, etc.). Three main categories of phenomenological models have been proposed for the CO_x claystone: continuum-based models (Souley et al. 2011, 2017a, 2022a, b, 2023a, b; Pardoën and Collin 2017; Yu et al. 2021a, b, 2022; Cuvilliez et al. 2017; Jung et al. 2022; Mánica et al. 2017, 2021a, b; Trivellato et al. 2019), discrete element modeling (Yao et al. 2017), and micromechanics-based model (Zhao et al. 2022). The first category includes isotropic or anisotropic elasto-visco-plastic models (Souley et al. 2011, 2017a; Pardoën and Collin 2017; Cuvilliez et al. 2017; Jung et al. 2022; Mánica et al. 2017, 2021a, b), elasto-visco-plastic models based on the weakness plane approach (ubiquitous joints) (Souley et al. 2022a), and damage-based and elasto-visco-plastic damage models (Trivellato et al. 2019; Yu et al. 2021a, b, 2022; Souley et al. 2022b, 2023b). These constitutive models have been formulated within the framework of continuum mechanics: purely mechanical conditions (Souley et al. 2011, 2022a; Cuvilliez et al. 2017; Jung et al. 2022; Trivellato et al. 2019; Yu et al. 2022) or of porous media under saturated (Souley et al. 2017a, 2022, 2023b; Yu et al. 2021a, b; Yao et al. 2017) or unsaturated conditions (Mánica et al. 2017, 2021a, b; Pardoën and Collin 2017; Zhao et al. 2022). Some of these models have been used to study the behavior of an IL-LLW repository for the operational phase (Alonso et al. 2021; Camusso et al. 2020, 2022; Tran et al. 2021), even very long-term after the repository closure (Alonso et al. 2024; Corman et al. 2022). It is worth noting that the mentioned models (Souley et al. 2011, 2017a, 2022a, b, 2023a, b; Pardoën and Collin 2017; Yu et al. 2021a, b, 2022; Cuvilliez et al. 2017; Jung et al. 2022; Mánica et al. 2017,

2021a, b; Trivellato et al. 2019; Yao et al. 2017; Zhao et al. 2022) are examples of the corresponding category and not the only ones in existence.

Alonso et al. (2021) studied the behavior of an IL-LLW cell for both the support liner (concrete and compressible material layers) and the host rock under THM loadings (i.e., excavation, seepage, and heat generation from the waste package) for the operational phase. They employed an anisotropic elasto-visco-plastic model proposed by Manica et al. (2017) to describe the behavior of the host rock. Alonso et al. (2024) recently performed 3D and axisymmetric simulations of a sealing system, where the operational phase lasts 100 years, incorporating a prescribed relative humidity on the internal wall of the tunnel. Camusso et al. (2020) used the engineering approach proposed by Saitta et al. (2017) to perform the thermo-mechanical modeling of an IL-LLW repository for 200 years following excavation and the installation of waste packages. Camusso et al. (2022) used a hybrid approach by combining the isotropic elasto-viscoplastic model slightly modified from Souley et al. (2011) with the introduction of the fracture zone within the model to reproduce the measured anisotropic convergence at the MHM URL. This approach was then used to determine the evolution of stress and strain within the liner of the IL-LLW repository under both mechanical and thermal loads. Corman et al. (2022) studied the THM-gas coupling around an IL-LLW repository over a duration of 100,000 years considering successive loadings: excavation, ventilation, and gas production due to the corrosion of metallic material within the disposal. In their study, the CO_x claystone was represented by the second gradient anisotropic elasto-viscoplastic model proposed by Pardoen and Collin (2017) and hydro-mechanical coupling with the multiphase flow was considered. Tran et al. (2021) envisaged the reliability analysis of the IL-LLW cell, where the host rock is modeled by the Lemaitre creep model, and the uncertainty of its parameters was quantified based on laboratory triaxial creep tests.

This study aims to compare the numerical models' results from the various studies mentioned in the previous paragraph (Alonso et al. 2021, 2024; Corman et al. 2022; Camusso et al. 2020, 2022; Tran et al. 2021) and other results not yet published, as well as to present the sensitivity analysis. The results have been provided by seven teams: National Institute for Industrial Environment and Risks (Ineris), Universitat Politècnica de Catalunya (UPC), University of Lille-Laboratoire de Mécanique, Multiphysique, Multiéchelle (LaMcube), University of Liège (ULg), ITASCA France (Itasca), Orleans University (UO), and Andra. Unpublished results are provided by Ineris, LaMcube, and UO. Ineris introduced weakness planes into the elasto-visco-plastic matrix to describe the induced anisotropic behavior of Callovo-Oxfordian (Souley et al. 2022a, 2023a). LaMcube used a double phase-field model coupled with elasto-visco-plastic

strain for replicating the CO_x responses. Andra modeled the IL-LLW cell using the isotropic elasto-visco-plastic model proposed by Souley et al. (2011) for the CO_x claystone. UO considered the engineering approach with the Mohr–Coulomb criterion for short-term behavior (i.e., a perfectly elastoplastic model) and the Lemaitre model for the long-term behavior of the host rock (Nguyen et al. 2023). First, the comparison of results between teams focuses on the evolution of three variables: the maximum absolute hoop (orthoradial) stress in the concrete liner, the maximum absolute radial strain in the compressible layer, and the excavation-induced fracture zone (EFZ). These variables are the main indicators to assess the stability and safety of the repository for the operational phase. The current design for the stability of the IL-LLW cell relies on two criteria: the maximum absolute minor principal stress in the concrete layer must be lower than a prescribed stress threshold (uniaxial compressive strength multiplied by a safety factor), and the maximum absolute minor principal strain in the compressive layer must be lower than two-thirds of the rigidification threshold of the compressible material (Zghondi et al. 2023). Moreover, the EFZ evolution defines the remaining intact host rock, which serves as the natural barrier for the repository. The long-term evolution of stresses and strains in the liner is controlled by the time-dependent response of the host rock, which is the combination of the creep strain, the hydro-mechanical coupling, and the thermal effect (if it is considered). Second, sensitivity analyses were carried out to evaluate different aspects, e.g., the time-dependent behavior, the thermal loading, the installation time of the support, the repository configuration, etc., to comfort the design robustness. The role of the compressible layer is discussed based on both deterministic and probabilistic analysis.

2 IL-LLW Zone and Cell

2.1 Concept of IL-LLW Zone and Cell

The Industrial Centre for Geological Disposal (Cigéo) consists of surface facilities and underground structures connected to each other by ramps and shafts. A possible architecture of Cigéo is illustrated in Fig. 1. The surface facilities are divided into two domains: the nuclear activity zone and the construction activity zone. Vertical shafts are designated for construction and support activities, ventilation, and the transport of workers. The ramps transport waste packages from the surface to the main level of the disposal (~500 m in depth). The disposal area at the main level is divided into two areas based on the waste types and the excavation size: HLW zone (micro-tunnel of about 0.7–1 m in diameter) and IL-LLW zone (tunnels of ~10 m in diameter). This separation of these two zones provides independence in terms

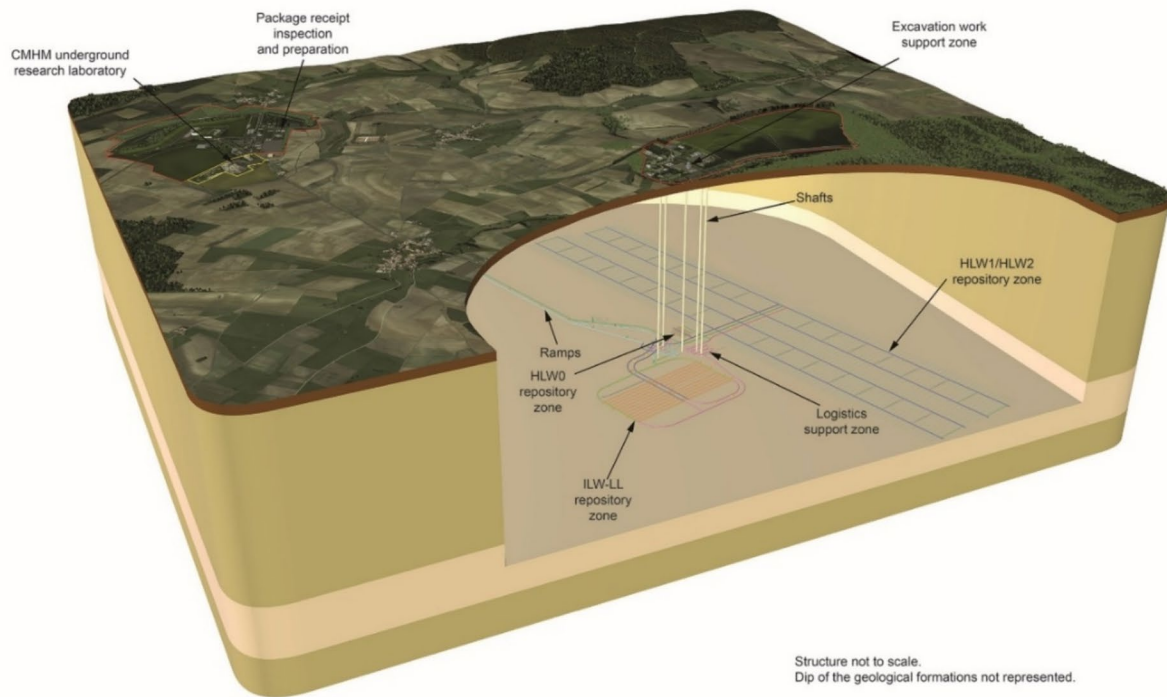


Fig. 1 Illustration of a possible architecture of the Cigéo project

of the management of different radioactive wastes and the physico-chemistry of each zone regarding the specific characteristics of the waste that they dispose of.

The IL-LLW zone consists of IL-LLW cells, connecting galleries, access galleries, and return air galleries (see Fig. 2). IL-LLW cells are parallel to each other and follow the direction of the minor principal horizontal stress. They are distributed on either side of two central connecting galleries. The other extremity of IL-LLW cells is connected to the return air galleries.

As displayed in Fig. 3, an IL-LLW cell includes a chamber for handling waste packages, a docking port for waste package transport, a disposal part with a length of 500 m for the waste packages, and a return air junction. Each cell accommodates one or several compatible waste families from a physico-chemical and geometrical point of view.

The secondary civil engineering in the disposal part consists of a tunnel floor, concrete infills on the right and left sides, and vault infill (see Fig. 4 for an illustration). The secondary civil engineering geometry varies from one cell to another depending on the waste package families and their arrangement. The secondary civil engineering size is designed to minimize the residual voids after the cell closure.

Most of the galleries and cells within the IL-LLW zone are planned to be excavated by TBM and supported by a double layer: 50 cm precast concrete segments covered by

20 cm of compressible material. This support liner design ensures the stability of the IL-LLW cell for approximately a hundred years for the placement and possible removal of waste packages.

The measurement estimates that the in situ stress field of the CO_x claystone formation is constituted by three principal stresses, including vertical stress (σ_v) corresponding to the ground weight, a minor horizontal principal stress ($\sigma_H \approx 1.3\sigma_h$), and horizontal principal stress close to the vertical stress ($\sigma_h \approx \sigma_v$) (Wileveau et al. 2007). For the sake of clarity, the minor principal stress is the most compressive stress, i.e., the highest in absolute value. Therefore, the minor principal stress is usually called by the major horizontal stress and the other principal horizontal stress is called by the minor horizontal stress in previous studies. In this study, we use the convention of continuum mechanic (i.e., tension is positive, and compression is negative). All the IL-LLW cells are drilled following the minor principal stress direction (σ_H).

2.2 Main Processes Governing the Mechanical Behavior of IL-LLW Cell

This section summarizes the main THMC processes governing the mechanical evolution of an IL-LLW cell from its construction to its closure. It starts with the excavation and the liner installation. The excavation (i.e. the stress

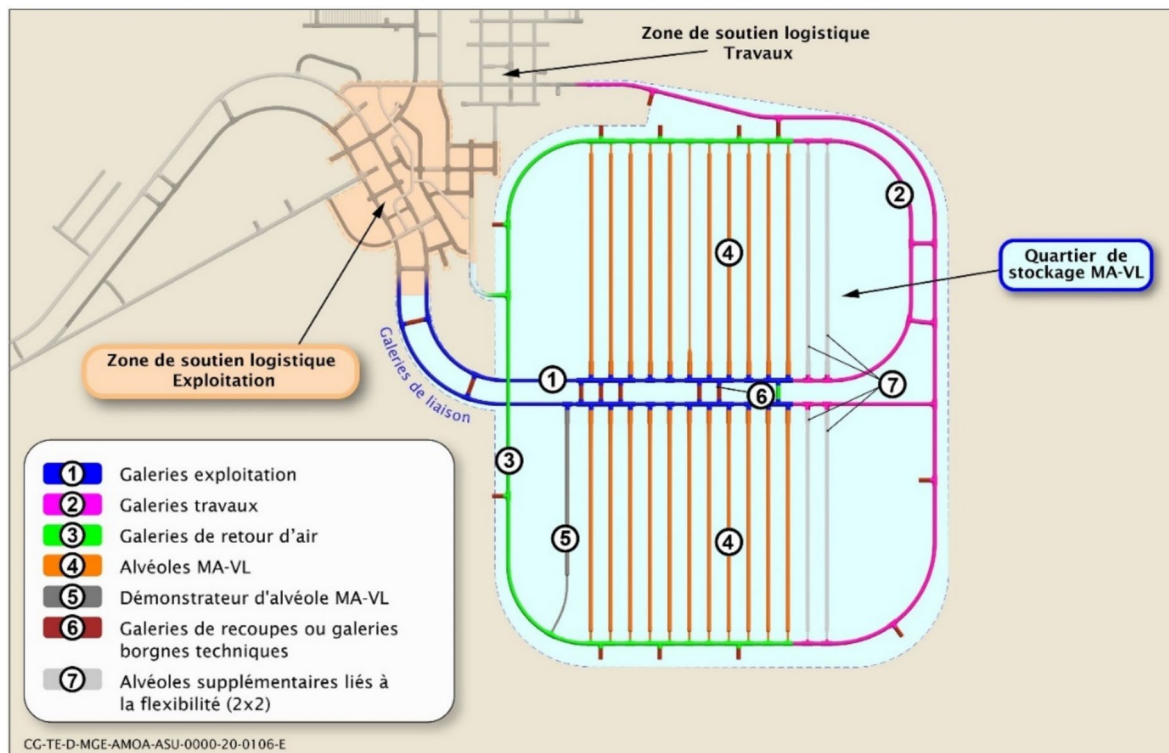


Fig. 2 An illustration for the plan of the IL-LLW zone in the Cigéo project

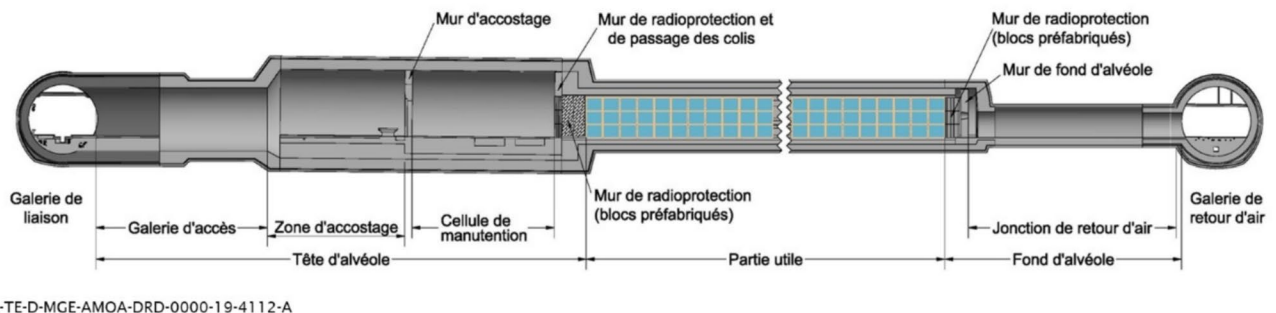


Fig. 3 An example of a longitudinal profile of an IL-LLW cell (for illustration)

unloading on the tunnel wall from the in-situ stress field to zero stresses) induces the convergence of the tunnel wall, the stresses and pore pressure redistribution, and the EFZ (Armand et al. 2013, 2014, 2015, 2023; Vu et al. 2020). As a reminder, IL-LLW cells are parallel to the orientation of the minor principal horizontal stress (σ_H). This implies that the in situ stress state in the cross-section of the tunnel is almost isotropic ($\sigma_h \approx \sigma_v$). However, according to the measurements and observations around the drifts at the MHM URL, the EFZ shape and the convergence are anisotropic. The EFZ exhibits a large extension in the drift sides (i.e., in the horizontal direction) and a limited extension on the roof and

floor of the drift (i.e., in the vertical direction) [see Armand et al. (2014) for more details]. The excavation-induced fractured zone leads to an anisotropic convergence evolution of drifts with horizontal convergence about two times larger than vertical convergence (Armand et al. 2013). Moreover, the distribution of pore pressure after the excavation is also non-uniform on a circumference within the rock with the same center of the excavation, on which over-pore pressures occur at the drift sides, while a drop in pore pressure is observed above the drifting roof (Armand et al. 2015). This anisotropic hydro-mechanical response occurs under quasi-undrained conditions due to the low permeability of

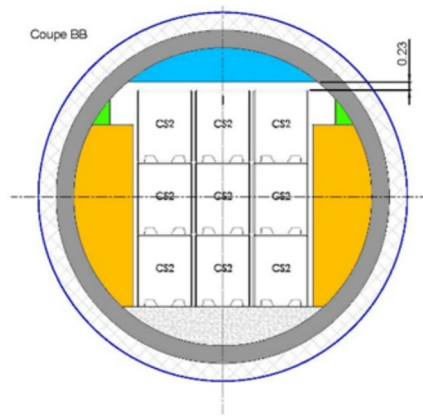


Fig. 4 An example of an IL-LLW cell section (for illustration)

the CO_x . This phenomenon is explained by two main factors: stiffness anisotropy and the EFZ shape (Vu et al. 2020). The overpressure magnitude depends on the excavation rate (Armand et al. 2023). Then the installation of the liner limits the instantaneous and time-dependent convergence of the drift wall, thereby increasing the stress and strain in both the concrete support and the compressible layer.

Afterward, the period from the construction of the secondary civil engineering to the beginning of waste package installation varies from a few years to several decades, depending on the chronology of each cell. The ventilation and the subsequent progress of waste package deposition lead to the desaturation of the host rock near the wall. According to the monitoring within MHM URL, the desaturation due to the ventilation takes place a few meters from the rock wall, primarily within the fractures in the EFZ, while the intact blocks between fractures remain almost saturated. The installation of exothermic waste packages results in an increase in temperature within cementitious materials and the host rock. The waste package delivery ensures the temperature does not exceed 65 °C in cementitious materials. Due to this low temperature, the thermal transient in the cell structures and surrounding host rock persists for a period of ~ a few hundred years. The temperature rise induces thermal strain, possible thermal effects on the creep behavior, and possible thermal compaction within the host rock in the near field (Braun et al. 2020). The time-dependent mechanisms including the thermal effects, the competition between the desaturation due to the ventilation and the seepage towards the repository due to the pressure gradient, and the creep strain of the host rock might increase the EFZ. However, as shown later in this paper, this increase should be very limited. The heat emitted from waste packages varies from one cell to another. Some cells might not contain any exothermic waste packages.

Recall that the concrete structure is still dry through the operational phase of the IL-LLW repository, and thus the

chemical effect is negligible. Moreover, gas production is also very limited, as reported by Corman et al. (2022). Therefore, these two aspects are not considered in the numerical simulations shown below.

3 Numerical Models

3.1 Model Geometry, Initial and Boundary Conditions

As a reminder, the current design of the IL-LLW zone includes several parallel cells with a 50 m (~5D) distance between the axes of two adjacent cells. The construction chronology, waste package disposal, and the quantity of heat emission vary from one cell to another. To simplify, two extreme configurations can be considered: either a single cell within an infinite formation reproduced by far-field conditions (Fig. 5a), or an IL-LLW zone encompassing an infinite number of parallel cells with identical characteristics (geometry, chronology, etc.). In the second configuration, only a single cell is modeled due to vertical symmetry (Fig. 5b). Then the whole, half, or a quarter of the model was modeled by different teams. A half or a quarter (if the gravity is negligible) is usually considered to reduce the computation resources. However, the representation of the whole domain might avoid the symmetry condition affecting the initiation of strain shear bands around the IL-LLW cell and the constrained boundary conditions along the symmetry axes. The model dimension in the vertical direction is more than 5D from the tunnel center. In THM and TM simulation, real geological layers from the ground surface to a depth of 1000 m are considered (Alonso et al. 2021; Camusso et al. 2020).

Most tunnels within the IL-LLW zone are planned to be excavated by TBM and have identical excavation diameter D and inner diameter $D_i = 8.7$ m. The current design only envisages the double-layer liner (0.5 m precast concrete segments and 0.2 m compressible material). However, the future excavation might also have an overcut. In this case, the overcut thickness is assumed to be 0.15 m and filled by either cement grout or compressible grout (like compressible material). Hence, the excavation diameter is 10.1 m for the case without the overcut and 10.4 m with the overcut.

The natural stress determines the initial state of the mechanical model. At the depth of MHM URL (−490 m), the natural stress tensor is characterized by the vertical stress $\sigma_v = -12.7$ MPa (intermediate principal stress), the major principal horizontal stress $\sigma_h = -12.4$ MPa, and the minor principal horizontal stress $\sigma_H = -16.1$ MPa (Seyed et al. 2017). The cell depth is not identical for all the simulations. A linear extrapolation is assumed to determine the initial stress state if the cell is not located at a depth of 490 m.

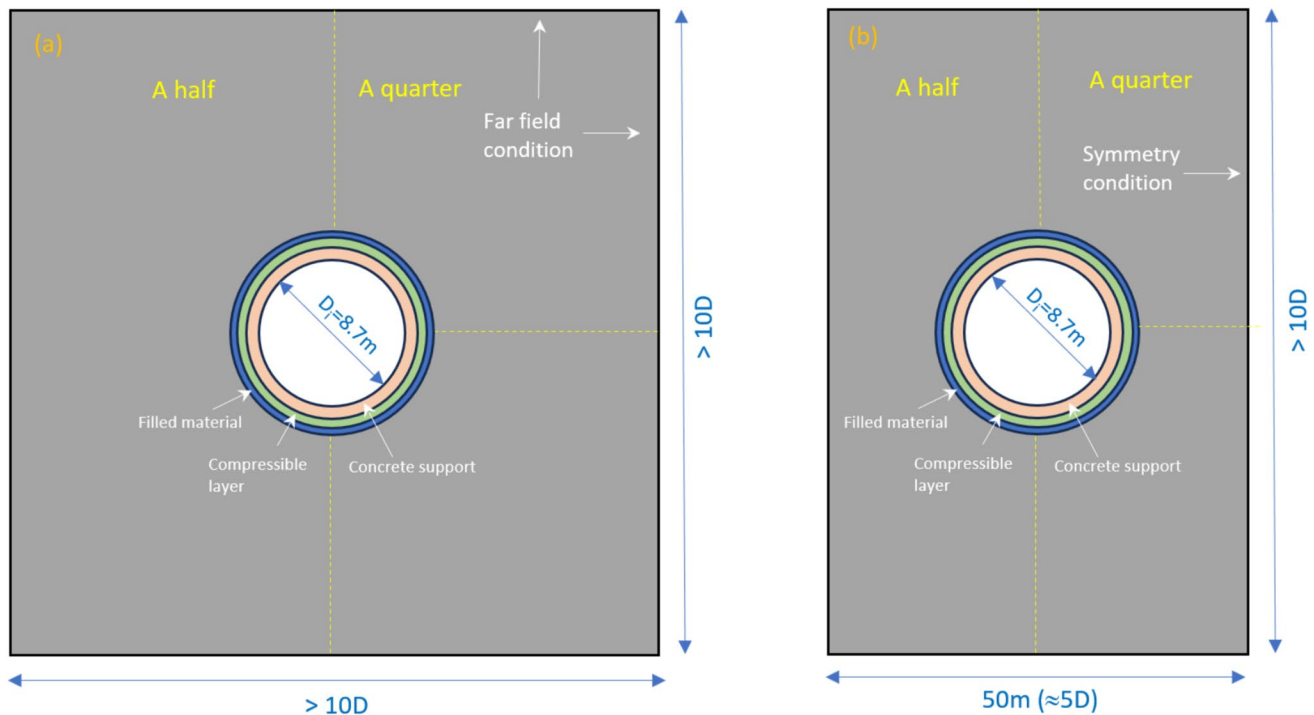


Fig. 5 Model geometry: **a** a single cell within an infinite medium, **b** parallel cells with symmetrical condition (scale is not true)

For hydro-mechanical simulations, the pore pressure of 4.7 MPa (measured at the URL depth) is prescribed. The initial stress and pore pressure are assumed to be uniform over the domain if only the CO_x is considered in the model. For TM and THM models, where all geological layers from the ground surface to a depth of 1000 m are considered, hydrostatic pore pressure gradient, hydrostatic stress gradient, and geothermal gradient are taken into account (see Alonso et al. (2021) for more details).

3.2 Materials

3.2.1 Callovo-Oxfordian Claystone

This paragraph summarizes from Armand et al. (2017b) the main experimental observations of the mechanical behavior of the CO_x investigated by classical laboratory tests. Triaxial tests have evidenced that the CO_x claystone exhibits non-linear (hardening/softening/residual) plastic and/or damage responses when subjected to triaxial loading. There is a transition from brittle to ductile behavior when the confining pressure increases. The host rock behavior also depends on the stress path, for instance, the triaxial load paths: axial compression with constant confining pressure, axial extension with constant confining pressure, axial extension with constant mean stress, lateral unloading with constant confining pressure, etc. The CO_x claystone seems to exhibit anisotropy in strength and stiffness. However, the difference

in peak strength and elastic moduli in different directions might be lower than the variability of these properties in a considered direction. The variability of properties is naturally due to the heterogeneity and spatial variability of CO_x claystone. Moreover, creep behavior is also observed from triaxial creep and relaxation tests. Armand et al. (2017b) presented an overview of the mechanical behavior of the CO_x claystone observed from classical laboratory tests.

The increase in permeability of CO_x claystone due to fracture occurrence is demonstrated by both laboratory tests (Zhang 2016) and field measurements in the EFZ over several years at the MHM URL (Vaissière et al. 2015). Unsaturated fractures exhibit self-sealing in relation to resaturation and mechanical compression, as evidenced by laboratory tests and the CDZ experiment in URL (Vaissière et al. 2015; Zhang et al. 2020; Wang et al. 2022a). However, the host rock near the drift wall will be subjected to desaturation due to ventilation over almost the operational phase. Full saturation will not be reached at the end of this phase. This is confirmed by numerical simulations performed by Alonso et al. (2024) and Rima et al. (2023). Thus, the possible swelling and self-sealing of unsaturated host rock near the wall due to resaturation will not occur. Nevertheless, the presence of fractures and confining pressure induces a significant decrease in the swelling capacity of the CO_x claystone (Zhang et al. 2021; Wang et al. 2021, 2022b). Therefore, the swelling and self-sealing phenomena are not considered in this work.

Parallel to the extensive laboratory and in situ characterization of the THM behavior of CO_x claystone, which has been partially shown in numerous published papers for more than 2 decades, the development of constitutive models (i.e., rheological models) for this rock has been undertaken. A review of constitutive models describing the mechanical behavior of CO_x claystone was presented in Seyedi et al. (2017). This effort continues to integrate more physical phenomena and coupling processes into mathematical models and to develop advanced models that better describe the behavior of this material [e.g., (Mánica et al., 2021a, b; Pardoen and Collin 2017; Souley et al. 2022a, 2023a, b; Yu et al. 2021a; Zhao et al. 2022)]. Consequently, constitutive models used by seven teams in this study to model the IL-LLW repository consider all or a part of the characteristics of CO_x claystone identified from the in situ and laboratory characterizations of this rock. Andra, Ineris, UPC, ULg, and LamCube teams used phenomenological models to describe the CO_x claystone behavior. Table 1 recapitulates the main features of the phenomenological models used in their numerical models. Engineering and hybrid approaches have also been used to replicate the mechanical responses of the host rock to the excavation (Camusso et al. 2020, 2022; Tran et al. 2021; Nguyen et al. 2023). In the engineering approach, the short-term behavior of the CO_x claystone is elastic and perfectly plastic based on the Mohr–Coulomb criterion. In contrast, the long-term behavior for both intact and damaged CO_x claystone is represented

by the well-known creep models: the Norton law is used by Itasca team (Camusso et al. 2020), and the Lemaitre law is employed by UO team (Nguyen et al. 2023). UO team also used only the Lemaitre model when performing the reliability analysis of the IL-LLW cell (Tran et al. 2021). As the model is quite simple, it is also considered as an engineering approach even if without the predefinition of a fractured zone. The hybrid approach proposed by Itasca team (see Camusso et al. (2022) is the combination of an isotropic constitutive model slightly modified from Souley et al. (2011) and an introduction of a predefined fracture zone to model the induced anisotropy. The host rock reaches the yield criterion faster (i.e., at lower strain) than the intact rock. Each team performed one or several simulations, the details of which are outlined in Sect. 4.1. Constitutive equations and associated parameters of these models are given in the origin publications (Alonso et al. 2021, 2024; Corman et al. 2022; Camusso et al. 2020, 2022; Tran et al. 2021; Souley et al. 2011, 2023a, Pardoen and Collin 2017; Mánica et al. 2017; Yu et al. 2021a). It should be noted that all these models are validated and calibrated by reproducing observations from both laboratory tests and in-situ experiments.

3.2.2 Support Materials

As a reminder, the IL-LLW cell liner consists of two layers: concrete and compressible materials, which are immediately installed after the excavation. Future construction

Table 1 Main features of the phenomenological models used to represent the CO_x behavior in this work

	Souley et al. (2011)	Souley et al. (2023)	Manica et al. (2017)	Pardoen and Collin (2017)	Yu et al. (2021a, 2021b, 2022)
Stiffness anisotropy	No	Yes	Yes	Yes	Yes
Strength anisotropy	No	No	Yes	Yes	No
Strain hardening /softening/residual	Yes	Yes	Yes	Yes	Yes
Brittle/ductile transition	Yes	Yes	No	No	No
Tensile plasticity	No	Rankine	Intersection	Intersection	No
Shear plasticity	Hoek–Brown	Hoek–Brown	Mohr–Coulomb	Van Eekelen	Drucker–Prager
Tensile damage	No	No	No	No	Yes
Shear damage	No	No	No	No	Yes
Lode's angle	Yes	Yes	Yes	Yes	-
Viscoplasticity	Lemaître	Lemaître	Modified Lemaître	Perzyna	Modified Lemaître
Viscoplasticity vs plasticity/damage	Yes	Yes	No	No	No
Permeability vs damage/plasticity	No	No	Yes	Yes	Yes
Biot coefficient vs damage/plasticity	No	No	No	No	Yes
Weakness plane	No	Yes	No	No	No
Regularization	No	No	No	Second gradient	Phase-field
Used by team	Andra, Ineris	Ineris	UPC	ULg	LamCube

of the repository may also have grout injected into the void between the external surface of the compressible layer and the excavated boundary. Grout material (also called filling material) can be either cementitious or compressible grout. The concrete layer assembles the precast concrete segments with the concrete grade B60, which remains in the elastic regime over the operational phase. Since the grout material layer is thin, it becomes plastic/damage due to the convergence of the host rock. Indeed, the host rock convergence increases the radial stress exerted on the external surface of the grout layer. Using an elastic model for the grout layer leads to important orthoradial stress exceeding the uniaxial compressive strength of the cementitious grout. Hence, the Mohr–Coulomb criterion is used to represent the plastic behavior of this material. The compressible grout and the compressible layer have similar behavior. Figure 6 shows the stress–strain curve under the oedometer load path of a compressible material, expected for the current section of the IL-LLW repository in the Cigeo project (Zghondi et al. 2023). This curve exhibits three stages. The first consists in an elastic response until ~ 1.5 MPa of stress. This elastic stress threshold allows confining the excavated wall and thus restrict the EFZ extension. The second stage exhibits a large deformation of $\sim 50\%$ under a low stress increase from ~ 1.5 to 3 MPa. This stage corresponds to the pore collapse mechanism. The third stage corresponds to the densification mechanism. Such behavior is typical for highly porous geomaterials, metals, pharmaceutical powders, etc. The purpose of the compressible material (particularly with its mechanical behavior in the second stage) is to absorb the convergence of the host rock and then limit the transmission of loading to the concrete liner. The current design aims at a response of the compressible layer within the second stage during the operational phase of the IL-LLW repository.

Simple constitutive models were used by different teams to describe the behavior of the compressible material, such as linear elasticity, trilinear elasticity, Cam–Clay

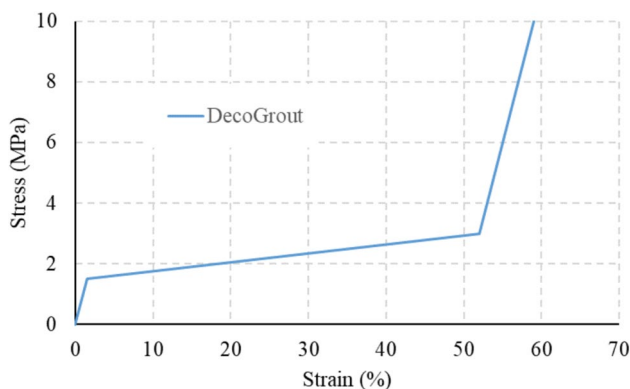


Fig. 6 Stress–strain curve of compressible material under oedometric loading (Zghondi et al. 2023)

(Roscoe et al. 1963), or double-yield (DY) models. The DY model is implemented in Itasca code FLAC3D (<https://www.itasca.fr/software/flag3d>). Ineris proposed a sophisticated model for compressible material, called LMC (Souley et al. 2017), which integrates different mechanisms: elasticity, the Drucker–Prager criterion for the shear failure mechanism, the pore collapse mechanism (volumetric cap yield) based on the Gurson criterion, the densification mechanism also represented by Gurson model, and a cut-off for the tensile failure mechanism. Constitutive equations and associated parameters for support materials are referred to in origin publications (Alonso et al. 2021, 2024; Corman et al. 2022; Camusso et al. 2020, 2022; Tran et al. 2021; Souley et al. 2011, 2017b, 2023a; Pardoen and Collin 2017; Mánica et al. 2017; Yu et al. 2021a).

3.3 Loading

Depending on the team, numerical models can be purely mechanical or coupled multi-physics (e.g., HM under saturated conditions, HM under unsaturated conditions, HM-gas, THM). Applied loading on the tunnel wall (i.e. excavation boundary before the support installation) and then on the internal concrete liner surface after the support installation depends on the couplings involved in the numerical models.

For purely mechanical simulation, only the chronology of excavation and liner construction relates to the mechanical loading. All the simulations assume 2D plane strain, except for the axisymmetric and 3D simulations considered by Alonso et al. (2024). The excavation simulation is based on the convergence-confining method (Panet and Sulem 2022), in which the excavation rate is 18 m/day and the deconfining curve provided by Seyedi et al. (2017). The deconfining ratio when installing the support may not be identical between different simulations (see Table 2 in Sect. 4.1). After that, the free displacement is prescribed on the internal boundary of the concrete liner.

In HM simulations under saturated conditions, the pore pressure reduction (from the initial value to atmospheric pressure) on the drift wall is also identical to the reduction curve given by Seyedi et al. (2017). When considering unsaturated conditions, the pore pressure reduces to a negative value corresponding to the relative humidity (RH) controlled by the ventilation, as proposed by Corman et al. (2022).

In the models with thermal loading, the heat flux on the internal surface of the concrete liner is calculated from the heat power of waste packages and their arrangement. The heat flux varies from one cell to another. UPC and Itasca teams considered the thermal loading in their THM and TM simulations for the sensitivity analysis (see Sect. 4.2.2). They used three thermal loads, as shown in Fig. 14.

Table 2 Differences between teams for their reference models

Teams	Case name	Code	Model	CO _x claystone behavior	Compressible material behavior	Coupling	Filled material	Symmetry condition	DR (%)
Andra	Andra	FLAC3D	2D	Isotropic elasto-visco-plastic (phenomenological model) (Souley et al. 2011)	Double Yield	Mechanic	No	Yes	96
Ineris	Ineris1	FLAC3D	2D	Isotropic elasto-visco-plastic + weakness plan (phenomenological model) (Souley et al. 2023)	LMC	Mechanic	Yes	Yes	95
	Ineris2	FLAC3D	2D	Isotropic elasto-visco-plastic (phenomenological model) (Souley et al. 2011)	Double Yield	Mechanic	No	Yes	95
UPC	UPC1	Code_Bright	2D	Anisotropic elasto-visco-plastic (phenomenological model) (Manica et al. 2017)	Tri-linear elastic	HM saturated	No	Yes	96
	UPC2	Code_Bright	3D	Anisotropic elasto-visco-plastic (phenomenological model) (Manica et al. 2017)	Tri-linear elastic	HM unsaturated	No	No	100
	UPC3	Code_Bright	Axisymetry	Isotropic elasto-visco-plastic (phenomenological model) (Manica et al. 2017)	Tri-linear elastic	HM unsaturated	No	No	100
ULg	ULg	Lagamine	2D	Second gradient anisotropic elasto-visco-plastic (phenomenological model) (Pardoen and Collin 2017)	Linear elastic	HM-gas	Yes	No	96
LamCube	LamCube	In house code	2D	Damage+plastic+viscoplastic (Yu et al. 2021a, 2021b, 2022)	Tri-linear elastic	HM	No	Yes	95
Itasca	Itasca1	FLAC3D	2D	Isotropic elasto-visco-plastic + explicite fractured zone (engineering approach) (Camusso et al. 2020)	Double Yield	Mechanic	No	Yes	95
	Itasca2	FLAC3D	2D	Isotropic elasto-visco-plastic + explicite fractured zone (hybrid approach) (Camusso et al. 2022)	Double Yield	Mechanic	No	Yes	95
UO	UO1	Code_Aster	2D	Isotropic creep (Lemaitre) (engineering approach) (Tran et al. 2021)	Tri-linear elastic	Mechanic	No	No	95
	UO2	Code_Aster	2D	Isotropic elasto-visco-plastic + explicite fractured zone (engineering approach) (Nguyen et al. 2023)	CamClay	Mechanic	No	No	95

4 Numerical Results

This section is first dedicated to comparing results obtained by different numerical models regarding maximum absolute stress in the concrete liner, and maximum absolute strain in the compressible layer. Second, results of sensitivity analysis conducted by one or some teams are presented to highlight the effect of different factors on the stress in the concrete liner: thermal loading, time-dependent mechanism, deconfining ratio (DR) at the support installation time, secondary civil engineering and filling material (grout). The evolution of EFZ over the operational phase is also highlighted.

4.1 Comparative Analysis

The comparison between teams is carried out on the results obtained from basic numerical models (also called reference

models or reference simulations). Each team performed one or more reference simulations and may also consider the sensitivity analysis. Reference simulations are not identical. The common points between all reference simulations consist of numerical models without thermal loading and secondary civil engineering. It is noted that Ineris, Itasca, and UO performed two different reference simulations (named Ineris1, Ineris2, Itasca1, Itasca2, UO1, UO2) and UPC considered three reference models (named UPC1, UPC2, UPC3). Reference models assume liner installation at a deconfining ratio $DR = 0.95$, except for three models Andra, UPC1, and ULg which adopted a value $DR = 0.96$, and $DR = 1$ is used in UPC2 and UPC3 models. Only ULg and Ineris1 models integrate the cementitious grout as filling material, while the gap between the excavated rock wall and the compressible layer is excluded in the other models. The configuration of a single cell within an infinite domain

is used in ULg, UO, UPC2, and UPC3 models, while other teams consider the configuration of an infinite number of parallel cells. Table 2 recapitulates the main characteristics of the reference simulations.

Since UPC and LamCube teams carried out a parameter calibration for the CO_x material under HM coupling with saturated conditions, they also considered this coupling for the simulation of the IL-LLW repository. This is also the case for ULg with coupled HM under unsaturated conditions. Other models only envisage the equivalent purely mechanical simulation. For the sake of clarity, results from Ineris1, Ineris2, LamCube, and UO2 have not been published yet. Whereas, Andra, UPC1, UPC2, UPC3, ULg, and UO1 models' results were, respectively, reported by Zghondi et al. (2023), Alonso et al. (2021, 2024), Corman et al. (2022), Camusso et al. (2020, 2022), and Tran et al. (2021).

Figure 7 plots the evolution of maximum absolute stress in the concrete support from all reference models in Table 2. The uniaxial compressive strength ($R_c = 60$ MPa) and the design criterion of 34.2 MPa are also plotted for comparison. The design criterion of 34.2 MPa corresponds to $R_c \times F_s$ (F_s is the safety coefficient) based on the Eurocode standard. It is worth mentioning that F_s already includes the creep strain of the host rock in Eurocode. Due to the EFZ shape and the anisotropy in convergence (horizontal convergence is higher than vertical convergence), the maximum hoop stress normally occurs at the top and on the inner surface of the concrete liner. However, the appearance of localized strain bands (for instance, in ULg simulation) may change the position of maximum orthoradial stress within the concrete liner. As observed, all curves exhibit a similar trend with an important increase rate at the beginning due to the short-term unloading of the drift excavation and the important initial creep strain rate of the host rock after the excavation according to the in situ measurements (see Armand et al. 2013). ULg obtained the highest stress (~ 31 MPa) at 100 years since the

compressible material is assumed to be elastic with Young modulus determined from the first slope in Fig. 6. Without the second stage representing the pore collapse (in Fig. 6), the compressible layer cannot limit the effort transmission from the CO_x claystone to the support layer. On the contrary, UO1 only obtained maximum absolute stress of 18 MPa in the concrete liner at 100 years since it considered only an isotropic creep behavior of the host rock, without non-linear short-term behavior. Therefore, even if different characteristics are considered in all reference models (see Table 2), the variability in maximum absolute stress in the concrete liner is insignificant, ranging from 18 to 31 MPa, and they are all lower than the design criterion. If we only consider the models with the anisotropic behavior of CO_x and the compressible material behavior with three stages illustrated in Fig. 6, the variability of maximum absolute stress in the concrete support reduces to a range of 22.5–27.5 MPa. This also emphasizes the role of the compressible layer, which absorbs the uncertainties in host rock behavior and properties, coupling processes and numerical model assumptions.

Moreover, the isotropic behavior of CO_x , considered in Andra, Ineris2, UPC3, and UO1 models, leads to low stresses (from 18 to 23 MPa) compared to anisotropic models. It is interesting to notice the difference between the two models (Andra and Ineris2), even if they used similar models to describe the behavior of the CO_x claystone and the compressible layer. This difference is due to two reasons: with the presence of filling material and $\text{DR} = 95\%$ in Ineris2 vs. without filling material and $\text{DR} = 96\%$ in Andra. The constitutive model describing the CO_x claystone behavior (see Souley et al. 2011) includes the coupling between the plastic and visco-plastic strain. Concretely, plasticity makes accelerated creep strains. The development of the plastic zone is more pronounced when DR exceeds 95%. Therefore, a larger extent in the plastic zone, in particular post-peak and residual zones in Andra model with $\text{DR} = 96\%$ leads to an increase in visco-plastic strain rate compared to the Ineris2 model with $\text{DR} = 95\%$. The effect of DR will be further discussed in Sect. 4.2.3. Moreover, the presence of filling material also helps to reduce the stress within the concrete support.

It is also interesting to compare 3D and axisymmetric simulations (UPC2 and UPC3) (see Fig. 7). A 3D simulation allows considering the inherent anisotropy of the CO_x claystone (stiffness, strength, and permeability), while all properties must be isotropic in an axisymmetric simulation. As shown by Manical et al. (2017), the inherent anisotropy generates the induced anisotropic HM responses (fractured zone and convergence) to the excavation. 3D and axisymmetric simulations yield 23.2 MPa and 20.3 MPa of maximum absolute stress in the concrete support at 100 years, which results in a relative difference of $\sim 13\%$.

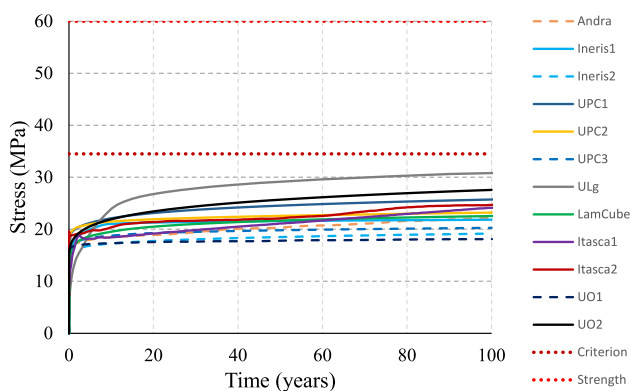


Fig. 7 Comparison of the evolution of maximum absolute stress in the concrete liner resulting from 12 reference models (dashed lines for isotropic simulations, continuous lines for anisotropic simulations)

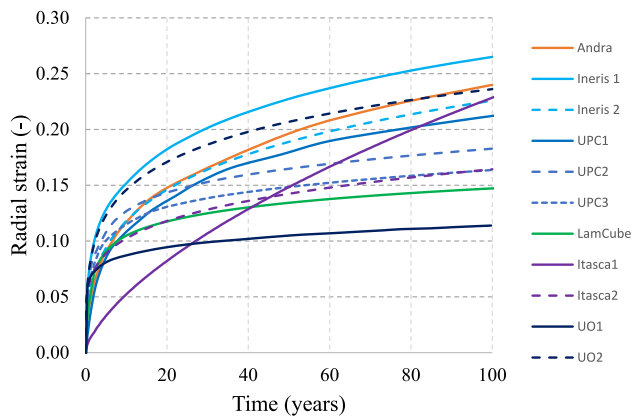


Fig. 8 Comparison of the evolution of the maximum absolute radial strain in the compressible layer for 11 reference models (excluding ULg)

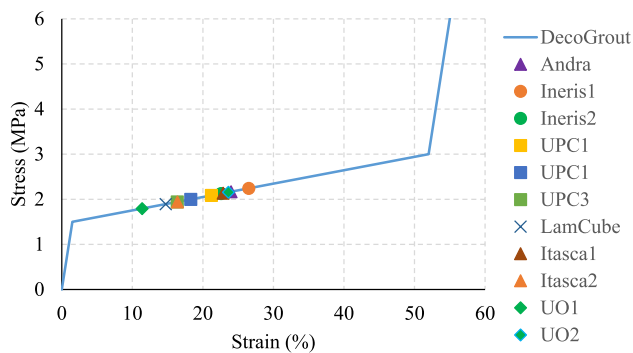


Fig. 9 Maximum radial strain in the compressible layer at 100 years for 11 reference models (excluding ULg)

Figure 8 displays the evolution of maximum absolute radial strain in the compressible layer for 11 reference models (excluding ULg model). As seen in Fig. 9, the maximum radial stress within the compressive layer at 100 years varies from 11 to 27% with a mean value of $\sim 20\%$, which corresponds to the collapse stage (second slope in Fig. 6) and far from the second plastic threshold (52%). As seen in Fig. 8, only Itasca1 model exhibits a different trend from other models. Indeed, Itasca1 model used the Norton law, with a constant strain rate vs. time, for the creep behavior of the host rock. This is the reason why the maximum absolute radial strain increases almost in a linear manner.

If we exclude the ULg model, where the compressible material does not follow the response in Fig. 6, the maximum absolute stress in the concrete layer at 100 years from 11 models is characterized by a mean value of ~ 22.9 MPa, a standard deviation of 2.2 MPa, a minimum value of 18.0 MPa, and a maximum value of 27.6 MPa (~ 1.5 times the minimum value). Regarding the strain in the compressive layer, the statistical characteristics of 11 models are: a

Table 3 Different inherent anisotropy considered for the implicit reproduction of EFZ

Anisotropy	Ineris1	UPC1	UPC2	ULg	LamCube
Stiffness	X	X	X	X	X
Strength		X	X	X	
Permeability		X	X	X	X
Biot coefficient				X	

mean value of 19.9%, a standard deviation of 4%, a minimum value of 11.4%, and a maximum value of 26.5% (~ 2.1 times the minimum value). This statistical analysis again shows the compressible layer's role in reducing the dispersion of the maximum absolute stress in the concrete layer over the operational phase.

For the implicit reproduction of the EFZ, Ineris1, UPC1, UPC2, ULg, and LamCube models considered different inherent anisotropy (see Table 3). It is interesting to note that Ineris1 and LamCube do not consider the inherent strength anisotropy. As shown by Souley et al. (2023a), using the ubiquitous joint, considered in Ineris1 model, allows for generating the excavation-induced anisotropy in convergence and slight anisotropy in EFZ. While the double phase-field model used in LamCube model leads to anisotropy in both convergence and EFZ shape (Yu et al. 2021a, b). As a reminder, the EFZ is defined as the plastic or damage zone around the drift in the continuum approaches used in all the simulations of this work. Figures 10, 11, 12 show the extension of EFZ from the end of excavation to 100 years resulting from three models: ULg, LamCube, and UPC2. It is observed that the EFZ does not change in ULg model. LamCube model shows a slight over-extension of the shear damage zone, but not any over-extension of the tensile damage zone. EFZ's over-extension means the additional extension of EFZ at 100 years to that at the end of the excavation. UPC2 does not show any over-extension

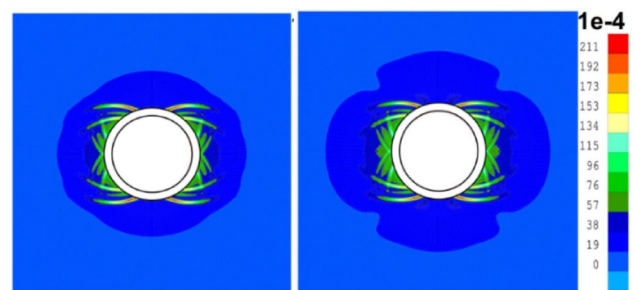


Fig. 10 ULg simulation: EFZ extension—total deviatoric strain at the end of excavation DR=100% (left) and 100 years (right) (Corman et al. 2022)

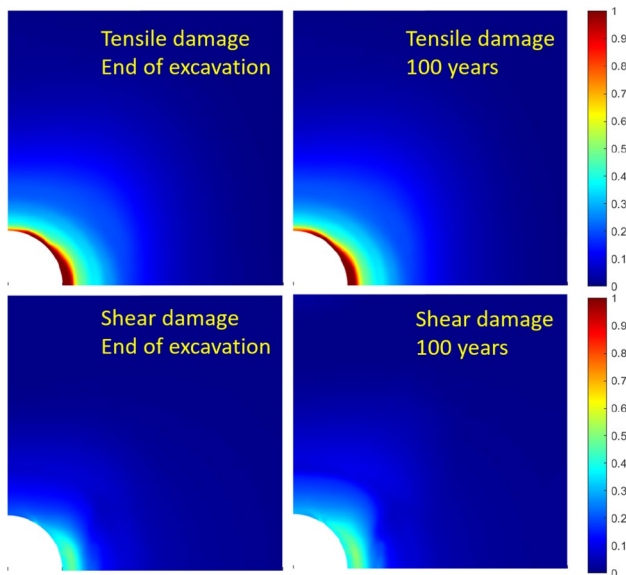


Fig. 11 LamCube simulation: EFZ extension

of the plastic zone. However, the magnitude of plastic strain increases over time in this zone. This is also seen in the UPC1 model with thermal loading, which will be shown in Sect. 4.2.2. EFZ evolution is also considered after the operation phase in Alonso et al. (2024) (over 10 000 years) and Corman et al. (2022) (over 100 000 years). They showed that the over-extension of the EFZ is limited.

4.2 Sensitivity Analysis

The aim of this section is multiple. First, the main factor controlling the stress evolution in the concrete liner is

highlighted. Second, the effect of thermal loading due to the heat released from the waste package is studied. Third, sensitivity analysis of the installation timing for the liner is performed via the DR. Fourth, the comparison between two models with and without internal civil engineering is shown. Fifth, the effect of filling material in the void between the drift wall and the external surface of the compressible layer is studied. Finally, the role of the compressible layer is brought out.

4.2.1 Time-Dependent Behavior Effect

Without thermal loading, the time-dependent behavior of the host rock is a combination of visco-plastic strain and HM coupling. This section only considers HM coupling under saturated conditions, where the transient flow takes place from the rock massif towards the tunnel due to the hydraulic gradient. This flow changes the stress state in the rock and the support. Figure 13 displays the evolution of the maximum absolute stress in the concrete liner resulting from four simulations: UPC1 and LamCube models with and without creep behavior. The comparison between models with and without creep shows that the effect of HM coupling is negligible in LamCube model and is not significant in UPC1 model. The visco-plastic strain of the host rock is the main factor that controls the magnitude and the evolution of stress in the concrete liner. This result is coherent with the conclusion made by Pardoen and Collin (2017) when modeling the convergence of a drift excavated within the CO_x formation following the orientation of minor horizontal principal stress, with and without creep behavior. Moreover, Coarita-Tintaya et al. (2020) showed that creep strain does not affect the pore pressure around the drift.

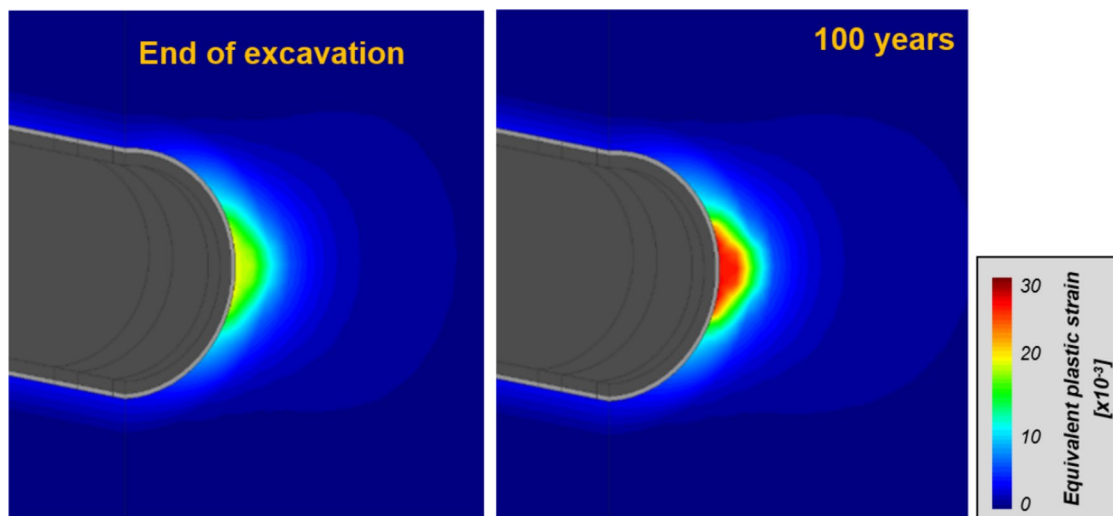


Fig. 12 UPC2 simulation: EFZ extension (Alonso et al. 2024)

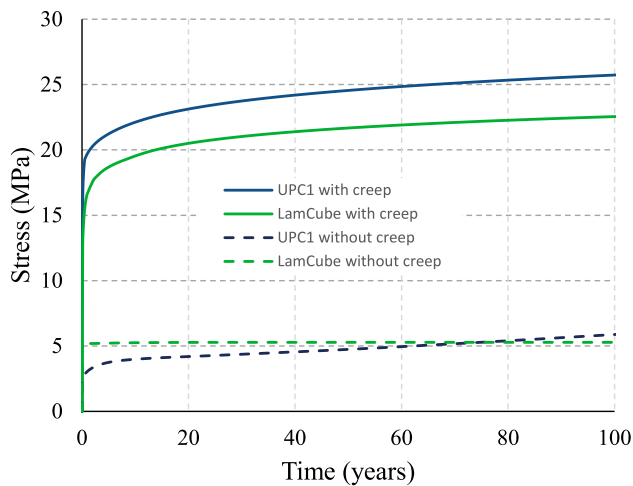


Fig. 13 Comparison between numerical models with and without creep behavior of the host rock

4.2.2 Thermal Load Effect

Almost all nuclear waste packages stored in the IL-LLW repositories are exothermic. The thermal design consists of determining the waste age at the beginning of the disposal and their arrangement to respect design criteria. One design criterion is maintaining a temperature lower than 65 °C within the cementitious materials. The heat released from the waste package increases the temperature within the support and the host rock, and thus thermal strain and THM coupling (thermal pressurization, thermal compaction, etc.). We consider UPC1, Itasca1, and Itasca2 models for studying the thermal effect. In Sect. 3, UPC1 model considers HM coupling under saturated conditions, while Itasca1 and Itasca2 models are purely mechanical simulations. By adding thermal loading, THM processes (thermal expansion and thermal pressurization) occur in UPC1 model, while only TM coupling (i.e. thermal expansion) takes place in Itasca 1 and Itasca2 models.

Two heat powers are considered for UPC1 simulation, called UPC1_THM1 and UPC1_THM2 models. The thermal load in the UPC1_THM1 model is identical to the Itasca1 model (called Itasca1_TM model). Another heat power is used for Itasca2 simulation, called Itasca2_TM. Figure 14 shows these three thermal loads (UPC1_THM1 = Itasca1_TM, UPC1_THM2, Itasca2_THM) represented by the heat flow on the inner concrete liner surface. UPC1_THM1 and UPC2_THM2 curves represent the models where all the waste packages are assumed to be instantaneously installed 4 years after the repository construction (excavation and support installation). In contrast, Itasca2_TM model considers a progressive installation of the waste packages in four stages at 4, 14, 24, and 34 years after the tunnel excavation, corresponding to four peaks in the curve for Itasca2_TM in

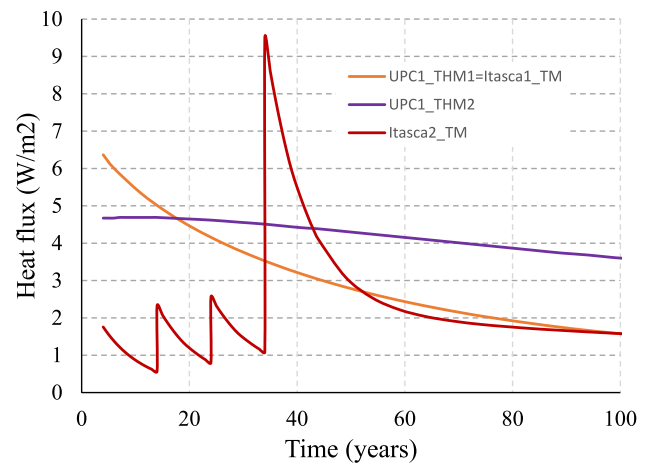


Fig. 14 Thermal loads on the tunnel wall

Fig. 14. These peaks, followed by a decrease in temperature, are due to ventilation. A significant increase in heat flux at 34 years corresponds to the ventilation stoppage.

Figure 15 displays the evolution of maximum temperature in the concrete liner. As expected, UPC1_THM1 and Itasca1_TM yield almost identical results regarding maximum temperature evolution in the concrete liner. For all four simulations with thermal loading, the peak temperature is reached before the end of the operation phase (< 100 years) and is lower than 65 °C. In an instant, the location of the maximum temperature is obviously on the inner boundary of the concrete liner. Moreover, a uniform heat flux applied on the inner concrete liner surface leads to a uniform temperature on this surface at an instant.

Figure 16 shows the comparison of the evolution of the maximum absolute stress within the concrete liner for models with and without thermal loading: UPC1_HM vs UPC1_THM1 vs UPC1_THM2; Itasca1_M vs Itasca1_TM; Itasca2_M vs Itasca2_TM, in which UPC1_HM, Itasca1_M,

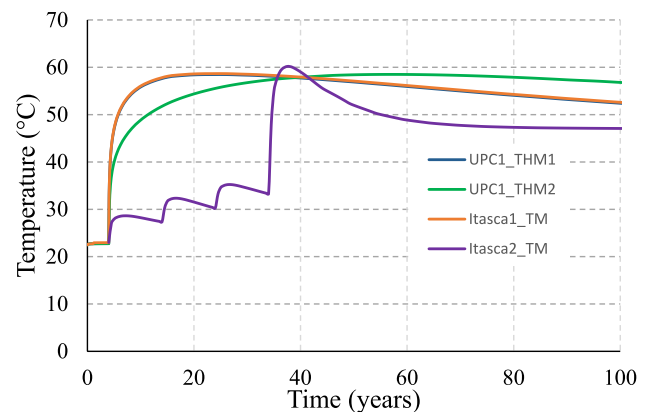


Fig. 15 Evolution of maximum temperature in the concrete support

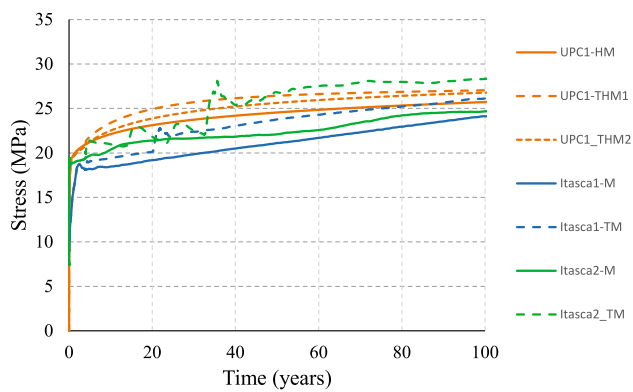


Fig. 16 Effect of thermal loading on the evolution in the maximum absolute stress in the concrete support

and Itasca2_M are UPC1, Itasca1 and Itasca2 models in Sect. 4.1 (e.g., in Fig. 7). It is observed that the thermal loading leads to a higher stress in the concrete compared to models without thermal loading. However, the effect of thermal loading is moderate. Indeed, the differences in stress at 100 years between models with and without thermal loading are: $\text{UPC1_THM1} - \text{UPC1_HM} = 1.1 \text{ MPa}$; $\text{UPC1_THM2} - \text{UPC1_HM} = 0.9 \text{ MPa}$; $\text{Itasca1_TM} - \text{Itasca1_M} = 2 \text{ MPa}$; $\text{Itasca2_TM} - \text{Itasca2_M} = 3.6 \text{ MPa}$. In Itasca1_TM and Itasca2_TM models, the creep rate of the host rock is assumed to increase linearly from the initial value at the initial temperature (22 °C) to three times higher

at 80 °C. Whereas UPC1_THM1 and UPC1_THM2 do not consider the thermal effect on the creep behavior of CO_x claystone. This is one reason why the difference in stress between UPC1 models is lower than that between Itasca1 and Itasca2 models. It is also essential to note that considering thermal loading results in maximum absolute stress in the concrete support consistently lower than the design criterion.

It is also interesting to investigate the thermal effect on the EFZ. UPC1_THM1 model is extended to 1000 years, and its EFZ evolution is shown in Figs. 17 and 18. As observed, the plastic zone does not change over time (see Fig. 17), but the plastic intensity represented by the plastic multiplier increases with time (see Fig. 18). This result is also coherent with the UPC2 model shown in Fig. 12.

4.2.3 DR Effect

This section investigates the effect of the liner installation time during tunnel construction for both phenomenological and engineering approaches. The excavation is modeled via the convergence-deconfining method (Panet and Sulem 2022). In such a method, the installation time is represented by a deconfining ratio (DR). Three models, UPC1_THM1 (in Sect. 4.2.2), Itasca2 and UO2 (in Sect. 4.1), are considered. Traditional engineering design of a tunnel usually uses a simple model for the rock, which has shown that a late installation of the support (i.e., a higher

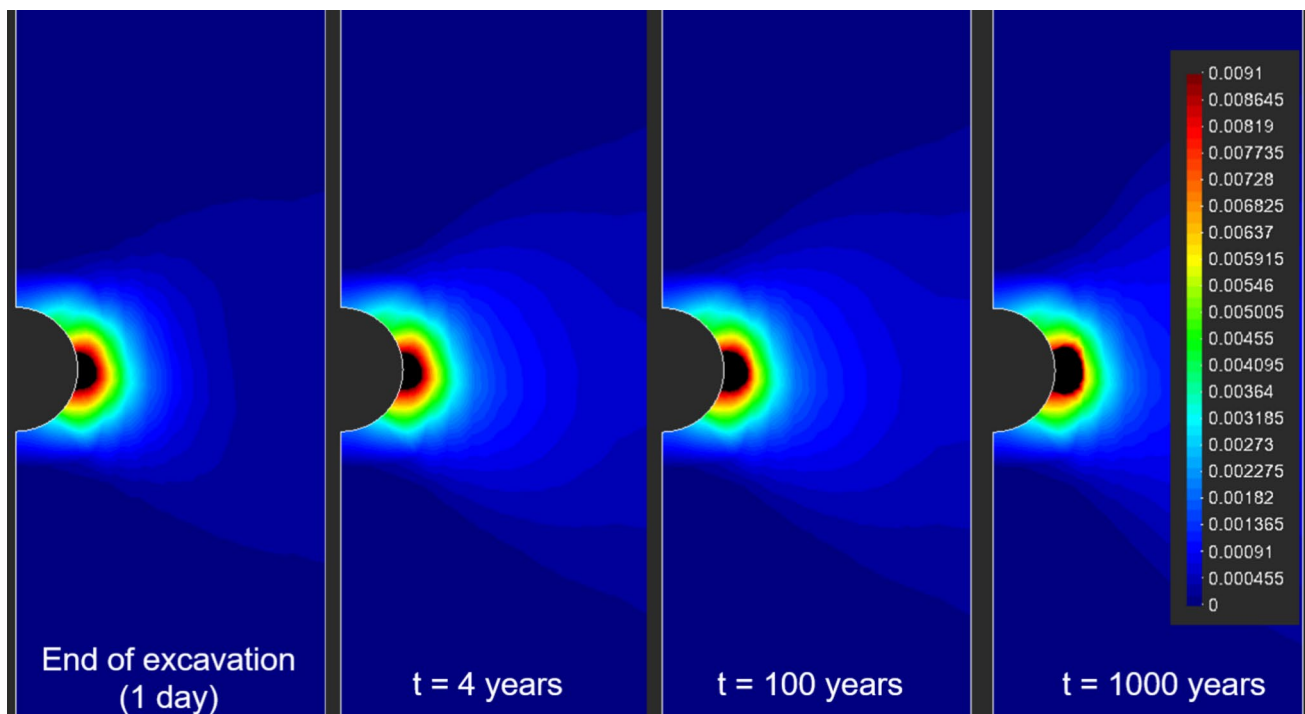


Fig. 17 Extension of EFZ–UPC1_THM1 model

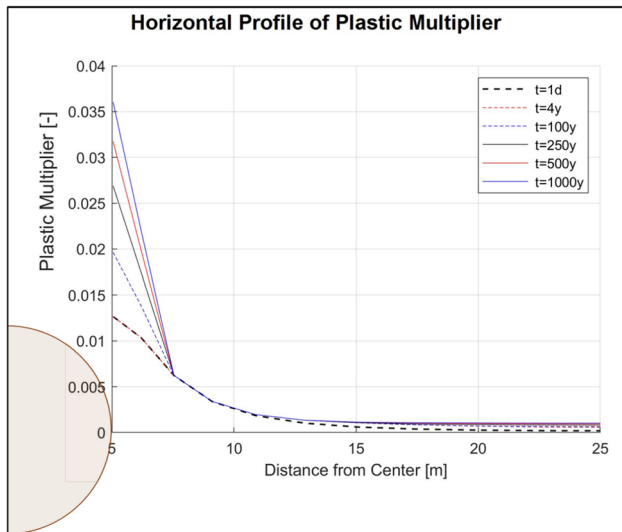


Fig. 18 Evolution of plastic multiplier within the plastic zone—UPC1_THM1 model

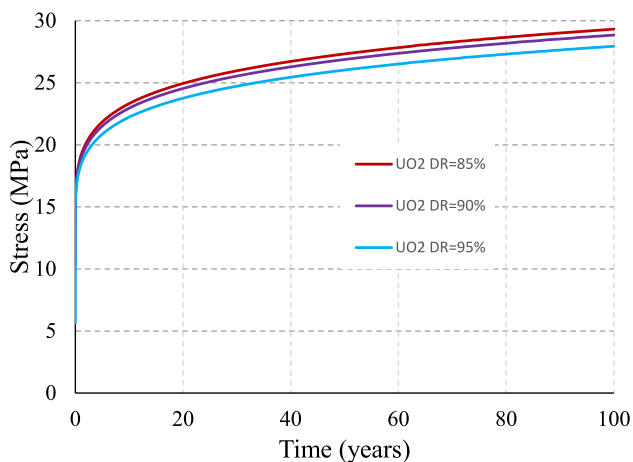


Fig. 19 DR effect: UO2 model

DR) results in smaller stress within the liner. Indeed, a higher DR leads to higher stress release at the tunnel wall before support installation, thus lower instantaneous stress applied to the support. This may not be the case if the host rock behavior is complex. Figure 19 shows the evolution of the maximum absolute stress in the concrete liner for different DR, varying from 85 to 95% for UO2 model. As a reminder, UO2 only uses a simple model for the host rock: elasto-perfectly plastic Mohr–Coulomb and Lemaitre laws for its instantaneous and time-dependent behavior. As expected in classical engineering design, the stress decreases when DR increases. However, this is not the case for Itasca2 and UPC1_THM1 models, where complex

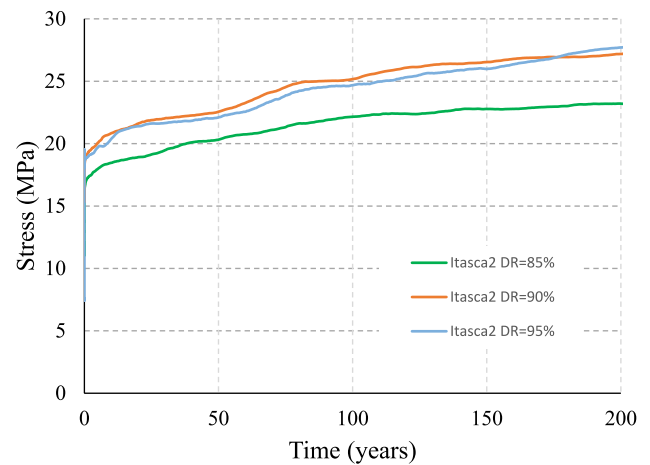


Fig. 20 DR effect: Itasca2 model

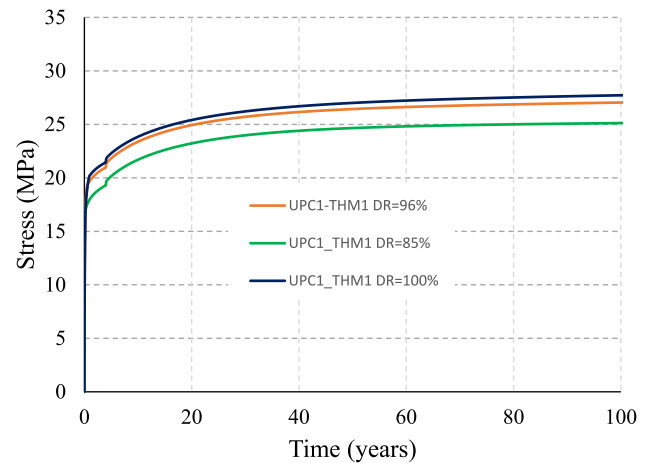


Fig. 21 DR effect: UPC1_THM1 model

constitutive models are used to describe the behavior of the host rock as realistically as possible. Indeed, Figs. 20 and 21 show that an increase in DR results in higher stress in the concrete. Itasca2 models were extended to 200 years to examine this trend between DR = 90% and 95%. Itasca2 used the constitutive model proposed by Souley et al. (2011), where the plastic strain contributes to accelerating the visco-plastic strain. Therefore, an increase in DR leads to an increase in the plastic zone and hence an increase in the visco-plastic strain of the host rock, which transmits more effort to the support liner. Although there is no coupling between plasticity and creep in UPC2_THM1 model, a DR increase also induces an increase in stress within the concrete liner (Fig. 21).

4.2.4 Internal Civil Engineering Effect

As shown in Fig. 4, the IL-LLW cell is not an empty tunnel. There is also the internal civil engineering part to place the waste packages. The internal civil engineering may reinforce some parts of the support liner. However, its repartition is not uniform on the internal surface of the concrete support, which might lead to stress concentration, particularly in the ears' zone of the repository. The effect of internal civil engineering is considered in this section via the Itasca2 simulation (see Fig. 22). Figure 23 shows that internal civil engineering causes higher stress in the concrete support, ~5 MPa higher than the model without internal civil engineering. This difference occurs in the very short term and then the two models with and without internal civil engineering exhibit similar evolution of the maximum absolute stress.

Figure 24 presents the absolute minor principal stress at 100 years in the concrete layer for models without and with internal civil engineering (Fig. 24a, b). As expected, the stress concentration is found in the ears' zone of the repository, precisely at two intersections between the bottom boundary of the secondary civil engineering of the vault and the concrete liner. If we exclude two elements around each intersection as seen in Fig. 24c, the maximum absolute stress reduces from 31.5 to 28 MPa (Fig. 24b vs Fig. 24c). Four elements on each ear zone should be excluded to decrease the maximum absolute stress from 31.5 to 24.7 MPa (maximum absolute stress in model without civil engineering) (see Fig. 24d). This result is significant since it shows where the reinforcement should be done to reduce the stress

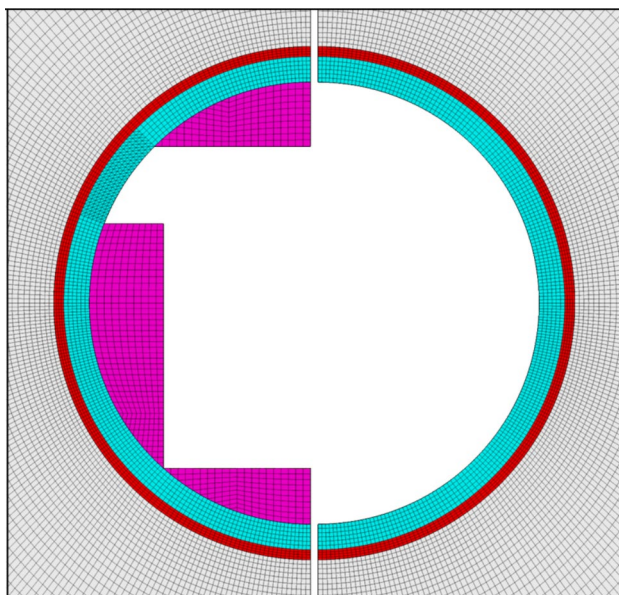


Fig. 22 Cell configurations with and without secondary civil engineering (color code: gray—CO_x; red—compressible layer; blue—concrete liner; pink—2nd civil engineering) (Color figure online)

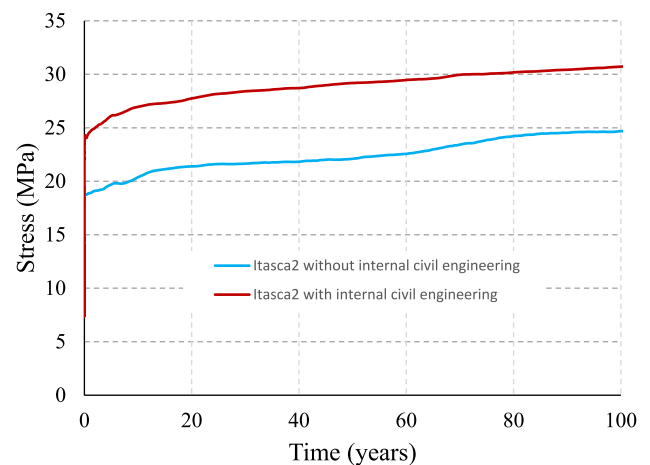


Fig. 23 Evolution of the maximum absolute stress in numerical models without and with secondary civil engineering (Itasca2)

concentration in the concrete liner. For instance, a thin layer should be added to the internal surface of the concrete liner at the ear zones to reduce stress on the support.

4.2.5 Filling Material Effect

Among reference models, Ineris1 and ULg consider filling material in the 0.15 m gap between the drift wall with $D = 10.4$ m and the external diameter of the compressible layer with $D_e = 10.1$ m. To investigate the filling material role, UO2 geometry model was modified to introduce the filling grout. Two filling grout types are considered. The first one is cementitious grout described by the elasto-perfectly plastic Mohr–Coulomb model and its properties are given by Corman et al. (2022). The second one is compressible grout with an identical behavior as the compressible material layer (see Fig. 6). Figure 25 plots the maximum absolute stress in the concrete support for three models: UO2 (without filling material), UO2 with compressible filling material, UO2 with cementitious filling material. The result brings out the benefit of the filling material, which helps to reduce the maximum absolute stress in the concrete liner. The stress reduction result from using compressible filling material (12.5% at 100 years) is more pronounced than with cementitious material (4.7% at 100 years). This result is coherent with field observations in the MHM URL.

4.2.6 Compressible Material Layer Thickness

The concept of the precast concrete segment covered by a compressible layer is an innovative design for drift support of the Cigéo project (Zghondi et al. 2023). The main mechanical behavior of this material is described in Sect. 3.2.2. Andra model in Table 2 is adapted to study the effect of the compressible layer thickness on the maximum

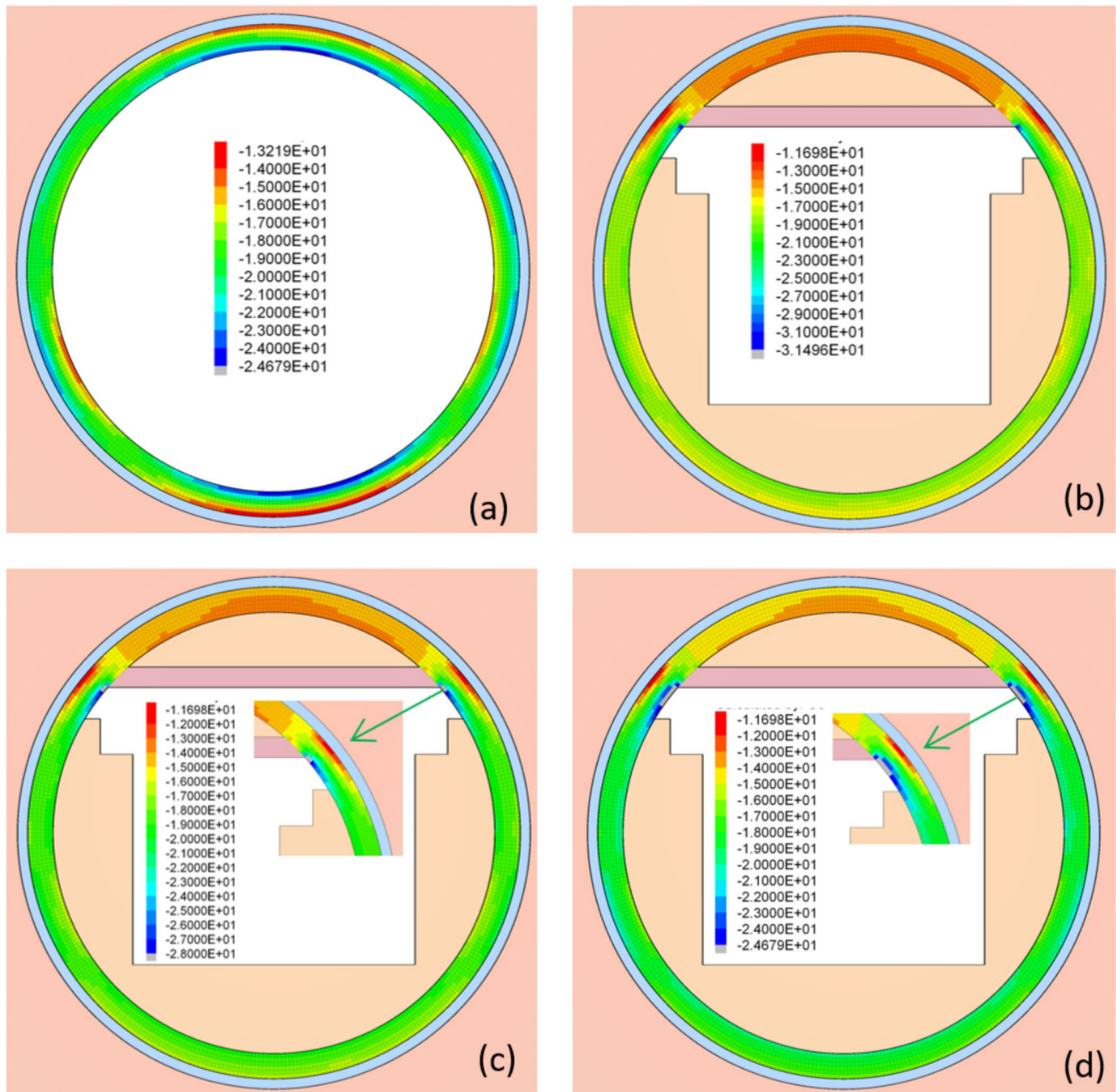


Fig. 24 Minor principal stress contour in the concrete liner at 100 years (compression is negative): **a** empty repository, **b** without internal civil engineering, **c** exclusion of two elements in the ear zone, **d** exclusion of four elements in the ear zone

absolute stress in the concrete layer. The inner diameter of the concrete liner ($D_i = 8.7$ m) and its thickness of 0.5 m are unchanged. Different simulations with different thicknesses of the compressible layer varying from 0 to 0.3 m were carried out. This means that the excavated diameter varies. It should be noted that an increase in the thickness of the compressible layer leads to a slight increase in the excavated diameter and thus a slight increase in drift convergence and stress in the concrete liner and conversely. Figure 26 shows the gain regarding the maximum absolute stress in

the concrete layer at 100 years brought by the compressible layer. A decrease in the maximum absolute stress in the lining vs. the thickness of the compressible layer is evidenced. The decrease rate decreases when the compressible layer increases. It is interesting to note that an asymptotic trend is observed. There is no further gain beyond a thickness of 20 cm for the Andra model (using the constitutive model proposed by Souley et al. (2011) for describing the mechanical responses of the CO_x claystone), although the drift convergence slightly increases due to a slight increase in the

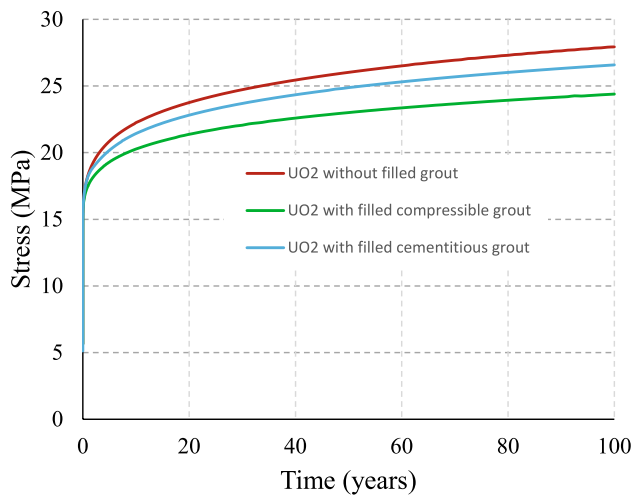


Fig. 25 Effect of filling material: UO2 model

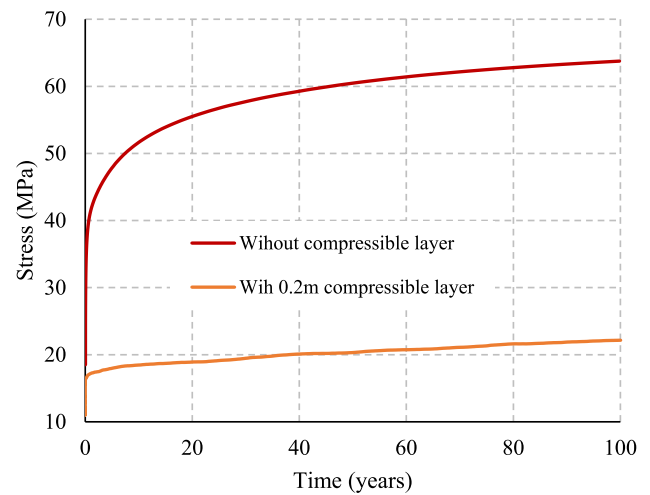


Fig. 27 Cell configurations with and without compressible layer (Zghondi et al. 2023)

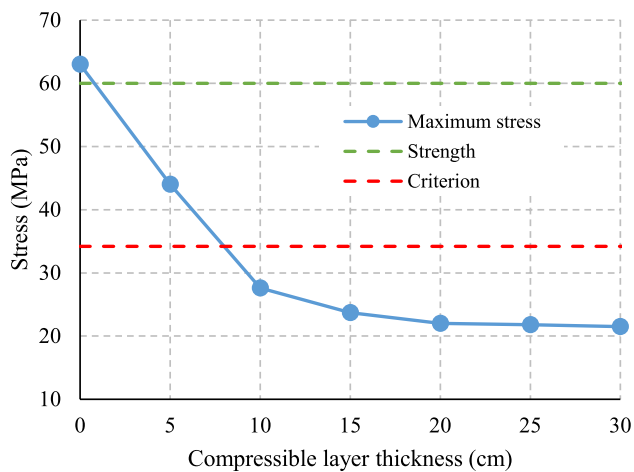


Fig. 26 Maximum of absolute stress within the concrete liner vs. the compressible layer thickness (Zghondi et al. 2023)

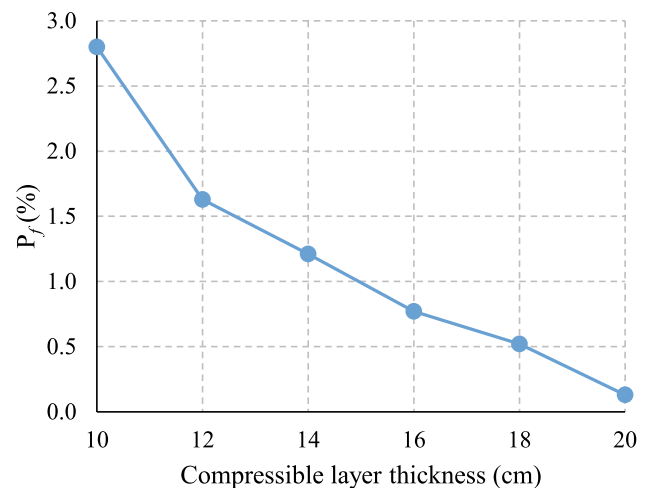


Fig. 28 Exceedance probability of the design criterion for the concrete liner as a function of compressible layer thickness (Tran et al. 2021)

excavated diameter. It is worth noting that the compressive layer thickness threshold of the asymptotic trend depends on the constitutive models of both the compressible material itself and the host rock. Indeed, UO2 model shows a benefit of ~3.5 MPa when the thickness of the compressible material layer increases from 20 to 35 cm (see Fig. 25).

Figure 27 illustrates the evolution of the maximum absolute stress in the concrete support over time for two models without and with 20 cm of compressible layer. It highlights a very significant stress reduction (from ~64 to ~22 MPa at 100 years) in the structure when adding the compressible material. The difference in stress between these two models also increases over time.

Tran et al. (2021) also showed the benefit of the compressible layer via a reliability analysis of the IL-LLW

cell. They assessed the effect of uncertainty of time-dependent parameters of CO_x claystone on the exceedance probability of acceptable stress in the concrete liner. The deterministic simulation with mean values of parameters is UO1 model in Table 2. Figure 28 shows a decrease in the exceedance probability of admissible stress in the concrete support when the thickness of the compressible layer increases, with a probability of 2.8% for 10 cm and 0.13% for 20 cm of compressible layer thickness. This result is also coherent with the reliability simulation shown by Do et al. (2021). They also studied a tunnel covered by a double liner: concrete and compressible layers. However, they considered a nonspecific rock with Burger creep behavior.

5 Conclusion

This study summarizes the numerical simulations of an IL-LLW repository within the CO_x claystone in Cigeo project over the operational phase from seven teams (Ineris, UPC, ULg, LamCube, Itasca, UO, and Andra). One team carried out one or several basic simulations and sensitivity analysis. The differences between teams consist of the constitutive models describing the behavior of the CO_x claystone and the compressible layer, the geometrical configurations (2D, 3D or axisymmetry; a single cell or several cells; with or without filling material in the gap between the liner and the rock; with and without internal civil engineering), the multiphysics coupling (M, HM, HM-gas, THM, under saturated or unsaturated conditions), and the excavation phase (timing of the liner installation). Several sensitivity simulations were performed and analyzed to comfort the findings. The presentation of results focuses on three variables: maximum absolute stress (absolute orthoradial stress) in the concrete layer, maximum absolute strain (absolute radial strain) in the compressible layer, and EFZ over-extension. The following significant conclusions can be made:

- All basis models yield a maximum absolute stress in the concrete liner lower than the design criterion and a maximum absolute strain in the compressible material layer lower than 60% of the rigidification threshold. The over-extension of the excavation-induced fracture zone is negligible during the operational phase. This result shows the robustness of the current design for the IL-LLW repository of the Cigéo project.
- Variability of the maximum absolute stress in the concrete liner at the end of the operational phase resulting from all basic models is insignificant (with a mean value of ~ 22.9 MPa, and a standard deviation of 2.2 MPa). At the same time, the variability of the maximum absolute strain in the compressible material layer is more pronounced (with a mean value of 19.9%, and a standard deviation of 4%). The low dispersion in the maximum absolute stress in the concrete support may be explained by two reasons: (1) constitutive models (both phenomenological and engineering approaches) closely describe the behavior of the host rock; (2) the presence of the compressible material may absorb the uncertainty related to the behavior of the host rock, the multiphysics coupling and the simulation assumptions.
- The host rock creep strain is the main factor controlling the stress magnitude and evolution in the concrete liner, while the effect of hydro-mechanical coupling is not significant.
- The effect of heating released from exothermic waste packages within different cells results in a slight

increase in stress within the concrete liner. However, a conservative assumption about the thermal effect on the host rock creep strain is considered.

- The effect of liner installation time, described by the deconfining ratio value, depends on the constitutive model adopted to describe the behavior of the host rock.
- The internal civil engineering causes an increase in stress within the concrete liner due to geometric corners. However, this overstress only concentrates in a small zone around these singular points, which should be easily handled by slightly modifying of the internal civil engineering geometry.
- The filling grout material presents a benefit for the stress within the concrete liner. The filling compressive grout exhibits a more pronounced benefit than the filling cementitious grout.
- The compressive material layer presents a significant gain in terms of stress within the concrete support. The stress decreases with an increase in the compressive material layer. The stress decrease rate decreases and may exhibit an asymptotic trend when the compressive material reaches a threshold, beyond which the gain brought by the compressive material is negligible.

Enriching the approaches to large-scale modeling of the IL-LLW repository is an ongoing study. First, the constitutive models describing the CO_x behavior should be added by physical processes observed from laboratory tests and in situ experiments, such as swelling, sealing, thermal compaction, closure of fractures in the EFZ due to the installation of the liner, temperature-dependent strength and creep, etc. Second, the evolution of THM parameters with the deformation of the compressive material will be considered. Third, mathematical models for concrete behavior will also integrate more phenomena, such as creep strain, reinforcement effect, and the interface between concrete segments, etc. Fourth, thermo-hydro-chemo-mechanical simulations of this repository over a highly long period ($\sim 10^6$ years) are in progress. Moreover, the inter-comparison exercise presented in this work will be carried out for other structures of the Cigéo project such as tunnels excavated along the major horizontal principal stress, the intersection between tunnels, HLW repository, the closure system, etc.

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Data Availability The data used during the current study are available from the first author on reasonable request.

Declarations

Conflict of interest The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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