Contribution of HADES URL to the development of the Cigéo project, the French industrial centre for geological disposal of high-level and long-lived intermediate-level radioactive waste in a deep clay formation

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Abstract: In the 1980s, HADES (High-Activity Disposal Experimental Site) was the first underground research laboratory (URL) dedicated to the study of the geological disposal of radioactive waste in a deep clay formation, the Boom Clay. It was not until the early 2000s, after a siting process, that ANDRA implemented the Meuse/Haute-Marne URL, in the Callovo-Oxfordian formation at a depth of about 500 m in order to develop the Cigéo project (French industrial centre for geological disposal). ANDRA therefore relied heavily on the work carried out in HADES, through numerous co-operation projects (participation in *in situ* experiments) both between ANDRA and ONDRAF/NIRAS and SCK CEN (EURIDICE) and/or with Mont Terri consortium, and within European projects (CLIPEX, RESEAL, etc.). This was driven by a dual objective: (1) to prepare its own experimental programmes in the Meuse/Haute-Marne underground laboratory (methodology, experimental devices and protocols, etc.); and (2) to acquire general knowledge on the behaviour of argillaceous rocks, in particular in terms of similarity and differences between the various argillaceous rocks. This paper illustrates the contribution of HADES to the ANDRA programme. This concerns the characterization of the claystone behaviour, host rock and swelling clay-based seals (hydromechanical, thermo-hydromechanical, excavation damaged zone, etc.), and the design and the behaviour of underground structures and seals in deep clay formation (constructability, lining/support, etc.).

The French Parliament passed two laws, in 1991 and 2006, concerning the management of radioactive waste in France. Dedicated to long-lived intermediate-level waste and high-level waste (HLW), the law of 30 December 1991 provided for 15 years of research into three potential solutions for the long-term management of these wastes (long-term storage, deep geological disposal and separation/transmutation for HLW). ANDRA, in charge of studies on geological disposal, undertook a preliminary study of several candidate sites. Then, following a government decision in 1998, ANDRA focused on a Callovo-Oxfordian claystone formation envisaged as the host rock for the repository. It started to build in 2000 the Meuse/Haute-Marne underground research laboratory (referred to as the CMHM in the following text) at a depth of 500 m in the middle of the Callovo-Oxfordian claystone layer (Delay *et al.* 2007).

At the end of this 15-year research period, the French Parliament adopted the 2006 law on the implementation of deep geological disposal for the long-term management of high-level and low-level long-lived waste, adding a requirement of reversibility over at least 100 years for this disposal. ANDRA then initiated the French project of the industrial geological disposal centre, also known as the Cigéo project, in order to carry out conceptual design studies, then preliminary design studies and finally detailed design studies for the application for a licence to build the facility, which was submitted in January 2023.

In addition to the design studies, the construction of the Meuse/Haute-Marne Underground Research Laboratory (URL) and the conducting of scientific and technological experiments (Armand *et al.* 2017) demonstrated the feasibility of the Cigéo project. The demonstration experiments also show the robustness (functioning from the point of view of

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processes, operation, etc.) of the various elements of the Cigéo project and are a key contributor to project optimization (in terms of safety and cost).

In the world of the geological disposal of radioactive waste, the experiments conducted in URLs meet two types of needs:

(1) characterization, i.e. acquiring knowledge about the geological, hydrogeological, geochemical and mechanical properties of the host rock and its response to disturbances (heat, radiation, geochemical interaction between the different elements (rock, concrete, waste, etc.)); and
(2) construction and operation, i.e. developing equipment to acquire know-how about the construction of all components of a disposal facility until its closure, as well as the emplacement and/or retrieval of the waste. In this phase, the optimization of the design will also be considered, whether it is for technical–economic optimization, or operational or long-term safety.

As pointed out by Delay *et al.* (2014), both the type and the importance of the work performed in URLs have evolved over the years. In the early days, some 25–40 years ago, the objective was primarily to identify the main scientific and technical issues and develop methods to manage them. The priorities were then (1) to define a programme related to the study on the confinement by the host rock or by the engineered barriers, (2) to develop experimental equipment and methods, (3) to conduct basic technical feasibility studies and (4) to collect fundamental geological data. Since the early 2000s and even more in recent years, all of the URLs have turned towards the development of demonstrators. Demonstration experiments are intended to represent, test and optimize actual potential disposal systems and repository components. The CMHM follows this pattern, with a first research phase up to 2006 during which the main goals were to characterize the confining properties of the Callovo-Oxfordian claystone through *in situ* hydrogeological tests, chemical measurements and diffusion experiments (Delay *et al.* 2007), followed by a programme focusing more on technological improvements and demonstration issues for different disposal systems like gallery support (Armand *et al.* 2015*a*), launched according to the 'Technology Readiness Level' Scale.

In the 1980s, HADES (High-Activity Disposal Experimental Site) was the first URL dedicated to the study of the geological disposal of radioactive waste in a deep clay formation, the Boom Clay located under the Mol–Dessel nuclear site. Accordingly, ANDRA relied heavily on the work carried out in HADES, through numerous co-operation projects (participation in *in situ* experiments) both directly between ANDRA and ONDRAF/NIRAS and SCK CEN (EURIDICE), and within European projects (CLIPEX, NF – PRO, RESEAL, amongst others). This was driven by a dual objective: (1) to prepare its experimental programmes in its own URL at Bure (methodology, experimental devices and protocols, etc.); and (2) to gain general knowledge in terms of similarity and differences between the various argillaceous rocks (indurated or non-indurated clays). The much longer duration of monitoring of the behaviour of the structures in HADES than at the CMHM is also a point of interest for ANDRA, as it enables longer-term trends to be confirmed by comparison, in particular concerning hydromechanical long-term behaviour. It must also be emphasized that ANDRA has been closely involved in the experiments carried out in the Mont Terri URL in Switzerland (Canton of Jura) on Opalinus claystone since 1996 (Bossart *et al.* 2017*a*). There is direct collaboration between ANDRA and SCK CEN at Mont Terri as partners of the Mont Terri consortium, mainly in geochemistry experiments. The outcomes/feedback from HADES and Mont Terri URL cannot be easily separated but played a significant role in the development of the CMHM.

Sealing experiments, a key point for closure of the geological disposal and long-term safety, were taken into account early in HADES and Mont Terri (Bossart *et al.* 2017*b*). The hydromechanical behaviour of the seals is related to the behaviour of the bentonite-based clay core, which is independent of the host rock. The *in situ* tests and associated studies carried out on this subject in HADES contribute to the improvement of knowledge of the behaviour and to the demonstration of the capacity of implementation and operation of seals.

The paper aims to illustrate the contributions of HADES to the ANDRA programme in the framework of the Cigéo project, with a specific focus on geomechanics in the broad sense. This concerns the characterization of the argillaceous media behaviour, host rock and swelling clay-based seals (hydromechanical, excavation damaged zone, etc.), and the design and the behaviour of underground structures and of seals in a deep clay formation (constructability, lining/support, etc.)

Underground research laboratories in clay formations and feedback for ANDRA

Clay formations in their natural state exhibit very favourable confining conditions for a repository of radioactive waste because they generally have a very low hydraulic conductivity, small molecular diffusion and significant retention capacity for radionuclides. That is why countries like Belgium, France and Switzerland study clay formations for deep geological repositories of radioactive waste and started to develop URLs to first study the feasibility of such a repository. HADES was the first one to be developed early in the 1980s followed in the 1990s by the Mont Terri URL and the CMHM in 2000.

The Meuse/Haute-Marne URL

Sediments of the Callovian–Oxfordian unit consist of a dominant clay fraction associated with carbonate, quartz with minor feldspars, and accessory minerals (Lerouge *et al.* 2011). The Callovo-Oxfordian is divided vertically into three litho-stratigraphic units from the base:

- The clay unit (UA) is the richest in clay minerals (over 40% on average), covering two-thirds of the total thickness of the layer. The mineralogical variations within it are weak and gradual. At the CMHM main level (490 m depth), the clay fraction is high (40–60%), and clay minerals consist of illite, ordered illite/smectite mixed layers, kaolinite, tri-octahedral, iron-rich chlorite and minor biotite.
- The transition unit is the term of passage between the UA and the silty carbonate unit (USC) with the highest levels of carbonates (40–90%).
- The USC, 20–30 m thick, shows a large vertical petro-physical variability associated with lithological alternations (marl and carbonate siltstone). It has a mineral composition which is more varied and heterogeneous, with a progressive enrichment in carbonates marking the existence of the carbonate platform of higher Oxfordian.

The Callovo-Oxfordian clay-rich rock porosity lies between 14 and 20% at the CMHM and is close to 18% at the URL main level. The natural water content in weight ranges between 5 and 8%. Owing to it having a very small mean pore diameter (about 0.02 µm), the claystone has a very low permeability (5×10^{-20} to 5×10^{-21} m^2).

In the clay unit, the unconfined compressive strength is about 21 MPa and larger than the 30 MPa in the silty carbonate unit above. The mechanical behaviour is slightly anisotropic, as with much sedimentary rock, and exhibits initial elastic behaviour followed by hardening and brittle behaviour after reaching the peak strength. Under deviatoric stress, Callovo-Oxfordian claystone (COx) shows creep deformation. A more detailed description can be found in Armand *et al.* (2017*a*).

Figure 1 shows the actual network of galleries and shafts at the CMHM. Two shafts provide access to two levels of drifts at depths of 445 and 490 m. The shafts with the connecting drift at −490 m (in between the two shafts) also ensure ventilation and safety. At the depth of 445 m, it is in the upper part of the COx layer where the carbonate content is higher. This niche at that depth was used to perform the first *in situ* experiments in the COx in the URL.

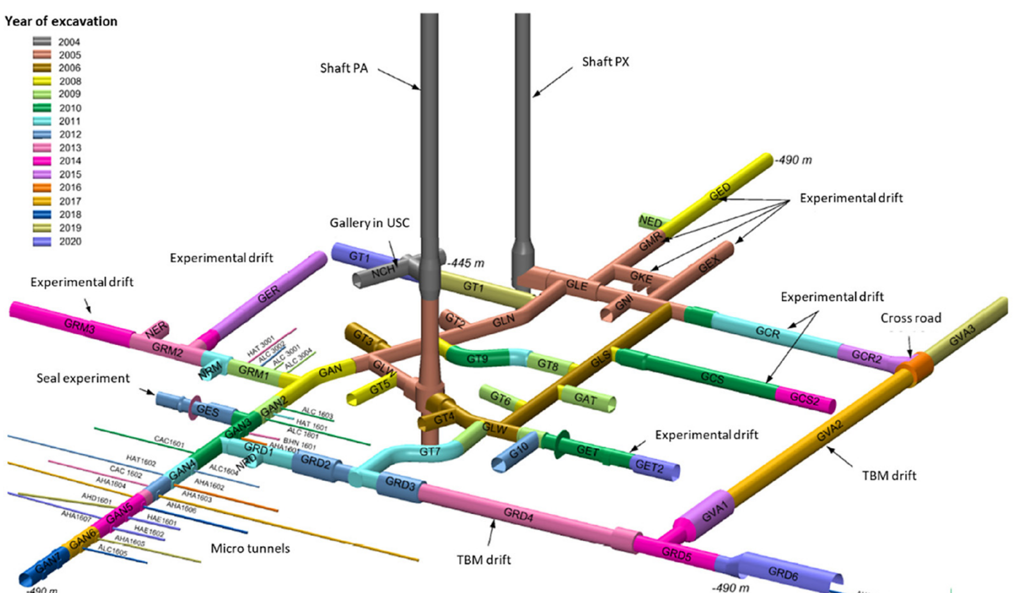

Fig. 1. The Meuse/Haute-Marne underground research laboratory (CMHM). Most abbreviations are the names of the galleries or micro-tunnels; TBM drift is a gallery excavated with a tunnel boring machine; Shaft PA is the access shaft; Shaft PX is the auxiliary shaft; and Gallery in USC is a gallery at 445m in the silto carbonate facies.

Among the experiments performed, the first geomechanical experiment (REP experiment) was carried out during the excavation of the main access shaft. It is a vertical mine-by-test designed to follow in real time the mechanical and hydromechanical responses of the Callovo-Oxfordian claystone formation to shaft sinking (Armand and Su 2006).

The depth of 490 m is the middle of the Callovo-Oxfordian layer, at the URL, which is the most representative of the potential location of the waste repository. At the main level of the URL (Fig. 1), the orientation of the experimental drifts was determined according to the orientation of the *in situ* stress field, which also corresponds to the major orientation of the drift in the repository. Wileveau *et al.* (2007) discuss the anisotropy of the stress state; the major principal stress σ_1 is horizontally oriented NE155°. *In situ* stresses in the Callovo-Oxfordian layer at the URL level are: $\sigma_v = \rho.g.Z$; $\sigma_h \approx \sigma_v$ and the ratio σ_H/σ_h is close to 1.3, where σ_v is the vertical stress, ρ is the density, g is the acceleration, Z is the depth, σ_h is the minor horizontal stress and σ_H is the major horizontal stress.

HADES URL

In Belgium, a tertiary clay formation, the Boom Clay, present under the Mol–Dessel nuclear site at a depth between 185 and 287 m below ground, was selected as a potential host formation for the disposal of HLW and long lived intermediate-level radioactive waste. The Boom Clay layer is almost horizontal (it dips 1–2% towards the NE) and water-bearing sand layers are situated above and below it. The Boom Clay Formation is characterized by a rather constant chemical and mineralogical composition over the formation (Vandenberghe *et al.* 2014).

To study the properties and behaviour of this poorly indurated clay at several hundred metres depth, a URL was constructed in the Boom Clay in the 1980s at about 225 m deep (Fig. 2). The construction of HADES started in 1980 and during the first phase, the first shaft and first gallery were constructed. In 1987, the second gallery (also known as the test drift) was constructed, followed by a major extension in 1997 with the construction of the second shaft and in 2002 with the construction of the Connecting Gallery. The last phase was the construction of the PRACLAY gallery in 2007 which is perpendicular to the Connecting Gallery (Fig. 2). The history of the construction of the HADES URL is described in Li *et al.* (2023).

At that depth, the porosity of the Boom Clay is about 39% with a low permeability between 2 and $4 \times 10^{-19}\,m^2$. The over-consolidation ratio is about 2.4 and the unconfined compressive strength is about 2–2.5 MPa. The Boom Clay behaviour is characterized by a highly non-linear stress–strain response and a ductile behaviour post peak. The total vertical stress and porewater pressure at HADES level are respectively 4.5 and 2.25 MPa, and the coefficient of earth pressure at rest is about 0.7 (Coll 2005; Bernier *et al.* 2007).

Feedback from HADES for construction and monitoring

A URL aims to demonstrate that the construction and operation of a geological disposal will not introduce unacceptable perturbations for radionuclide migration (Delay *et al.* 2010). One of the first concerns is the ability to excavate and support the galleries at great depth. Even before having its own URL, ANDRA joined other partners to test methods of

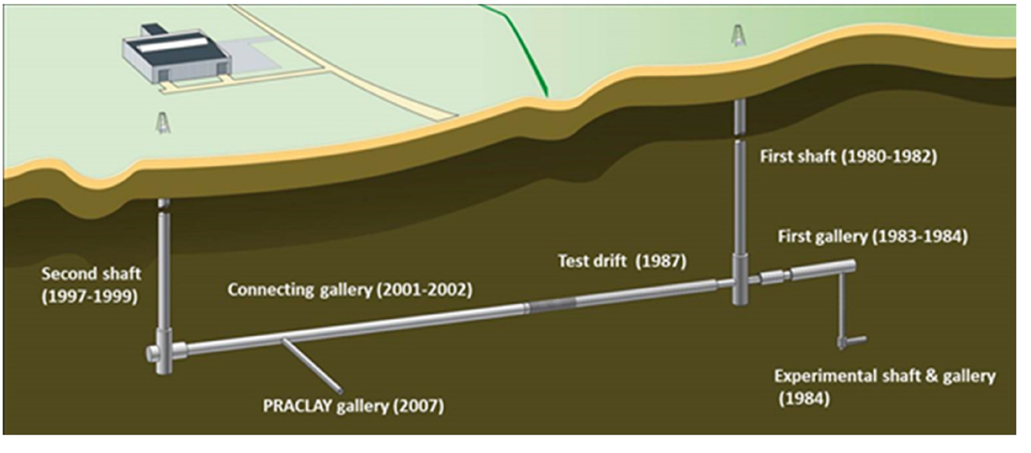

Fig. 2. The HADES underground research laboratory and its construction phases.

support in clay formations. The first test was performed in particular in HADES where a 12 m long gallery was lined with metallic sliding rings composed of four sliding elements to study this specific lining in view of the future construction of their laboratory in the CMHM.

A mine-by experiment is a state-of-the-art project to characterize excavation-induced damage and determine relationships governing the behaviour of a rock mass around an underground opening. Bernier *et al.* (2002) and Martin *et al.* (2002) show that mine-by-test measurements (i.e. emplacement of hydromechanical measurements in boreholes prior to the excavation of the gallery, supplemented by measurement during the excavation work) is an accurate way to obtain reliable hydromechanical data around a drift, giving insight into time-dependent processes for understanding hydromechanical behaviour and conceptual models of the excavation damage zone. In HADES, the EU CLIPEX project (Bernier *et al.* 2002) was realized as part of the extension of the underground research facility, in order to demonstrate that galleries can be constructed industrially while keeping the disturbance of the host-rock at an acceptable level for the long-term safety of the disposal facility. The extension consisted of the realization of a second shaft and a Connecting Gallery of 90 m length, which connects the existing test drift.

As partner of this EU CLIPEX project (Bernier *et al.* 2002), ANDRA received feedback for the construction of its own URL and used the excavation worksite in the Callovo-Oxfordian layer as a scientific experiment to characterize the impacts of excavations, understand the hydromechanical behaviour of the claystone and study the excavation damaged zone. As in HADES where different types of lining methods were used (test drift lined with concrete blocks and wooden plate between the segments as in Fig. 3; Connecting Gallery lined with concrete segmental lining (wedge-blocks technique in the CLIPEX project)), the experimental strategy developed by ANDRA at the main level of the CMHM was the study of the HM behaviour of structures built with different methods for different directions with respect to the on-site stress conditions. In a sense, it reflects previous work performed in HADES but with a more systematic approach. It is based on sequential development of the network of galleries in order to compare and monitor the hydromechanical behaviour of parallel galleries (Table 1). In galleries running in the same direction, sections with increasingly rigid supports and linings were built in order to compare their behaviour in the short to medium term (Table 1). The first supports used were so-called 'flexible' supports; these consist of sliding arches and radial bolting and sprayed concrete (Martin *et al.* 2010) and afterwards radial bolting with sprayed concrete with compressible shims (Bonnet-Eymard *et al.* 2011) in line with rock convergence values. More rigid supports (thicker and with linings poured in place several months after excavation) were also used in order to minimize soil deformation (Table 1). In addition, prefabricated arch segments were installed as works progressed with a road header, to simulate TBM excavation in

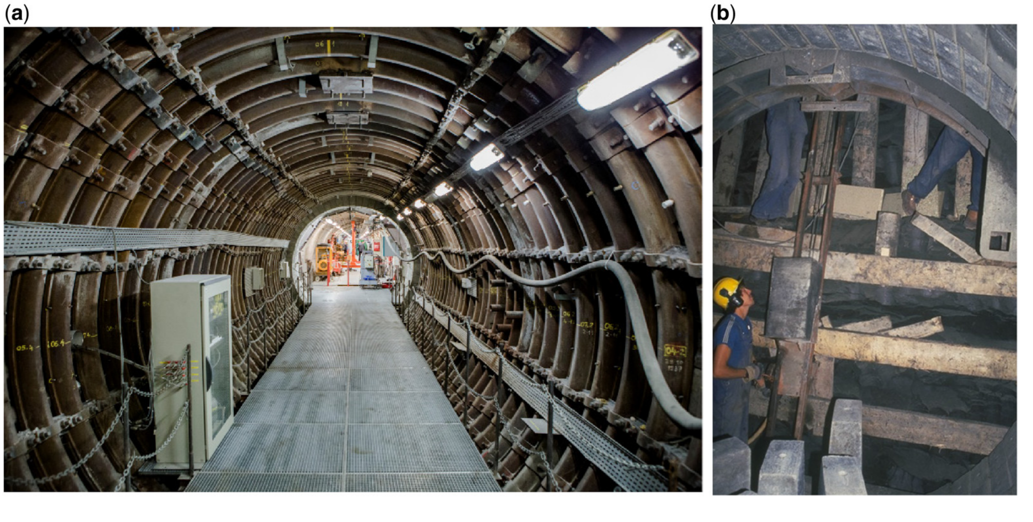

Fig. 3. (**a**) View of the ANDRA section in HADES with sliding steel arches. (**b**) Construction of the test drift with semi-manual techniques. Each ring is composed of 64 concrete blocks (60 cm thick) with wooden plates between the segments. Source: ©EIG EURIDICE.

Table 1. *Different geometries and supports used at the CMHM*

Name of gallery		GCS	GCR	GRD2 (BPE)	GRD4 (TPV)	GVA2	GRD3	GAT	GET	GRM3* Pilot	GRM3* Final	GED	GAN	GER
Diameter (m)		5.2	5.4	6.3	6.27	6.27	7.8	5.2	5.2	3.8	5.7	4.6	5.2	5.2/5.4
Section	Circular	+	+	+	+	+				+				+
	Semi-circular with vault						+	+	+		+	+	+	
Radial bolting		+	+	+			+	+	+	+	+	+	+	+
Shotcrete (18 + 3 cm) with compressible wedges		+	+											+
Sliding arches/shotcrete (5 cm)							+	+	+		+	+	+	
Shotcrete 45 cm (in four layers)				+										
Concrete segments 45 cm (C60/75)/classical or compressible grouting					+	+								
Concrete lining poured in place (30 cm) at 7 months after excavation	C30/37		+											+
	C60/75		+											+

*Gallery GRM3 was excavated in two stages: first with a ‘pilot’ gallery of 3.8 m diameter and then over-excavated at a diameter of 5.7 m.
The locations of the different galleries are shown in Figure 1.

gallery GRD4 and GVA2. Figure 4 illustrates some of the supports used at the CMHM (Armand *et al.* 2015*a*).

The results obtained at the CMHM show that the traditional and mechanized excavation methods are adequate for the excavation of the Callovo-Oxfordian (i.e. for the Cigéo underground structures), but that whatever the method, fracturing induced by the excavation appears and the fracture pattern depends on the orientation of the galleries with respect to the orientation of the major horizontal stress. The demonstration of gallery constructability in a less indurated clay at HADES is confirmed by the results provided by the CMHM for the Callovo-Oxfordian, which exhibits more brittle behaviour.

Even in the 1990s, only a few experimental data existed for tunnels (Panet 1995). The mine-by experiment aims to provide *in situ* data to assess the short- and long-term hydromechanical behaviour of the gallery. In the CLIPEX instrumentation, water pore pressure, total stress and displacement were used to cover three parts: the first part consists of eight instrumented boreholes (Fig. 5a) drilled from the test drift (Bernier *et al.* 2002); the second part consists of two instrumented boreholes drilled from the second shaft; and the third part consists of the instrumented concrete segment.

A similar approach was used at the CMHM to monitor the hydromechanical response during excavation. The architecture of the laboratory with parallel galleries made it possible to set up measurements of pore pressure, relative displacement (extensometer, inclinometer) from the first gallery to monitor the behaviour of the second gallery and so on (Fig. 5b). Armand *et al.* (2013) shows the advantage of this monitoring system, as it makes it possible in particular to measure the radial displacements before the passage of the front and thus better estimate the rate of deconfinement at the passage of the front if it is analysed with the convergence/confinement method. If the parallel galleries are too far apart (Fig. 5c), the location of the measurement boreholes is similar to that used in CLIPEX (Fig. 5a).

Particular attention was paid to the monitoring of pore pressures with the installation, where possible, of a network of piezometers, even at a large distance from the excavated gallery. In fact, in these clayey rocks, which have a very low permeability, a small volumetric deformation is instantly translated into a variation in pore pressure. Bernier *et al.* (2007) show very distant pressure variations in the case of the Boom Clay in HADES. Taking into account this observation and in order to try to reveal the extent of the hydromechanical impact of the excavation, a large-scale pore pressure survey was set up at the CMHM. Figure 5b illustrates this for the GCS gallery excavation with all the boreholes for pressure

Fig. 4. Example of different gallery supports: (**a**) GCS gallery emplacement of the compressible concrete wedge; (**b**) GCS gallery support with compressible concrete wedge and shotcrete; (**c**) GCR gallery with concrete lining grouted in place (thickness ∼27 cm); (**d**) GVA3 gallery with rock bolt sliding arches and a thick layer of shotcrete; (**e**) segment emplacement under the shield in the GRD gallery; and (**f**) view of the final support in the GRD gallery.

and displacement measurements carried out before the excavation with measurements in the near field of the future gallery, but also measurements at several tens of metres' distance. Fifteen instrumented long boreholes (30–50 m long) were drilled from the surrounding drifts (GAT and GLS). Nine boreholes of the 15 are devoted to measurement of pore pressure with a multi-packer system (five intervals

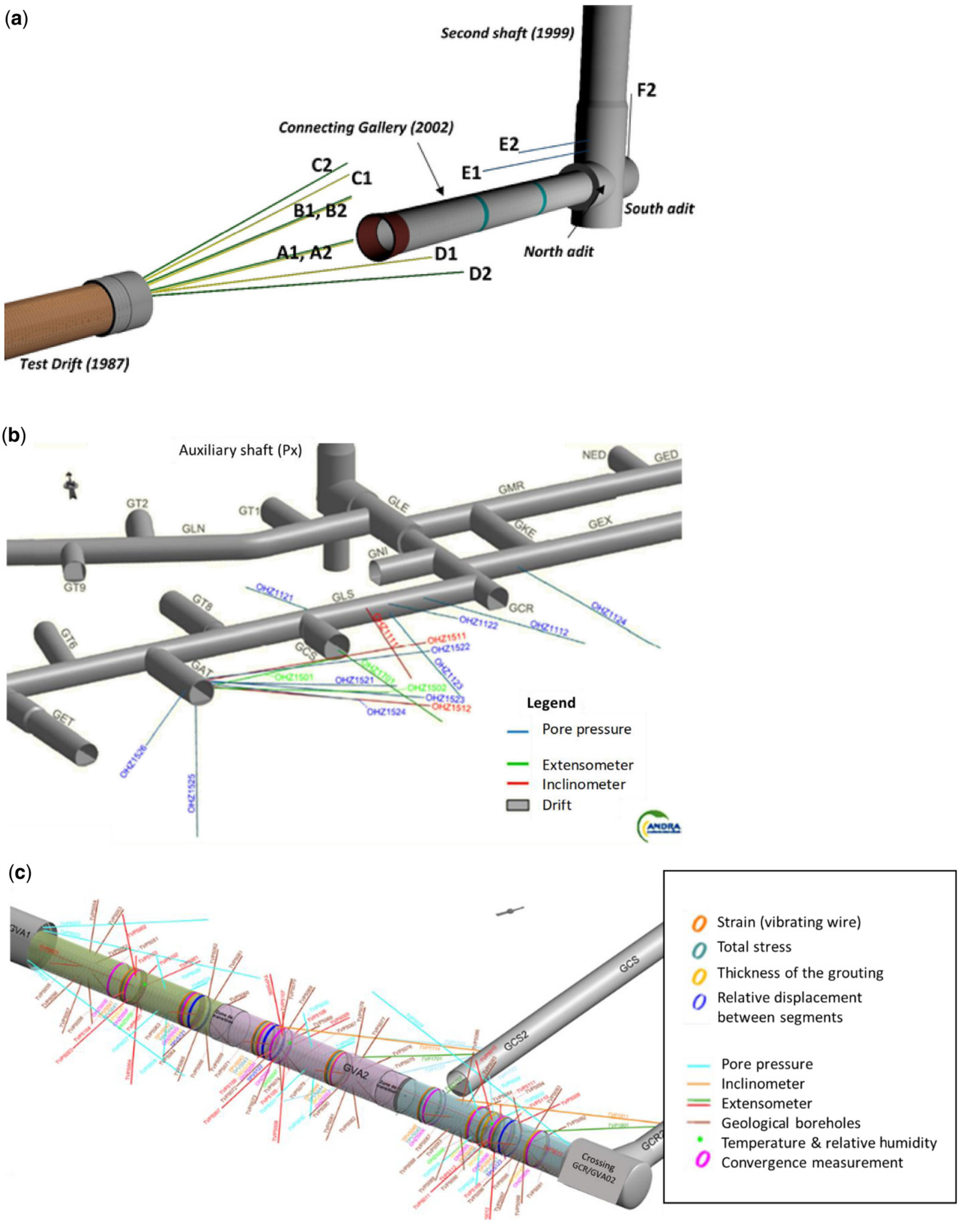

Fig. 5. Mine-by experiment: (**a**) CLIPEX project (HADES) – general schematic CLIPEX instrumentation plan during the Connecting Gallery excavation phase (Verstricht *et al.* 2022); (**b**) GCS gallery in the CMHM; and (**c**) GVA2 gallery in the CMHM.

in which pore pressure is measured) and cover an area from the GCS wall up to 50 m away. Three extensometers measure radial and axial displacements in the horizontal plane, and three inclinometers measure vertical displacements. The main data and insights gained from this monitoring survey can be found in the "Hydromechanical behaviour of galleries" section.

Hydromechanical behaviour of galleries

During excavation

At the CMHM, *in situ* measurements showed an anisotropic deformation (Armand *et al.* 2013) and pore pressure distribution around the gallery (Armand *et al.* 2015*b*), which depends on the gallery orientations. This anisotropic behaviour is observed from the beginning of the excavation and persists over time. For both directions of excavation ($\|\sigma_H$ and $\|\sigma_h$), lateral overpressures (i.e. in the horizontal direction) were measured before the front passage, with higher values observed for drifts excavated parallel to the direction of σ_H.

Figure 6 illustrates the evolution of pore pressure measured on the side-wall axis and above the gallery (GCS) parallel to the major horizontal stresses. All the measurement intervals are stabilized around 47 bar before the beginning of the excavation work, except the interval 5 (chamber 5) which is not far enough from the GAT gallery. The last measurement interval (chamber 1, Fig. 6a) in the borehole is located ahead of the face and was destroyed during the excavation. The amplitudes of the overpressure and of the pressure drop depend on the distance to the wall and to the face (along the axis of the drift).

The distance of influence on the pressure field is more than 20 m. For the interval ahead of the face (chamber 1), pore pressure increases and reaches a peak when the face is at the 4.8 m interval. Then pressure drops to 0 at 2.7 m, probably owing to the fact that the face hit the borehole.

At the wall in near field, the peak of pore pressure was observed ahead of the face. A pore pressure drop was observed when the face reached the plane of the measurement interval. A drastic drop was observed down to the atmospheric pressure in the zone where fracture appears (see "The excavation damage zone" section). Further into the rock, an overpressure is generated during the excavation of the gallery and dissipates slowly. The front of the overpressure propagates into the rock mass. Ten years after the excavation a small overpressure (0.3 bar) still remains noticeable at 15 m from the wall.

Above the gallery, a small overpressure is observed before the front crosses the section of measurement; on reaching the section, a drop of pressure is observed with the magnitude depending on the distance to the wall. It is noticeable that pore pressure reaches atmospheric pressure and/or suction in areas without any excavation-induced fracture (see "The excavation damage zone" section), meaning that volumetric deformation is large enough to create pore pressure drop which cannot be sustained owing to a quasi-undrained condition. The overall evolution of pore pressure is similar for different-sized excavations at the CMHM (Armand *et al.* 2015*b*), as shown in Figure 7 for a micro-tunnel (ALC1605) 0.8 m in diameter.

The explanation of this evolution could be the following: the excavation is performed under a quasi-undrained condition owing to the low permeability of Callovo-Oxfordian claystone and the evolution of pore pressure is essentially controlled by the volumetric strain. An increment of the compressive volumetric strain generates an overpressure while the extensional volumetric strain causes a pressure drop (negative pressure increment). The inherent anisotropic stiffness of the rock mass is one of the factors triggering this response but seems to be insufficient to explain the pore pressure evolution as

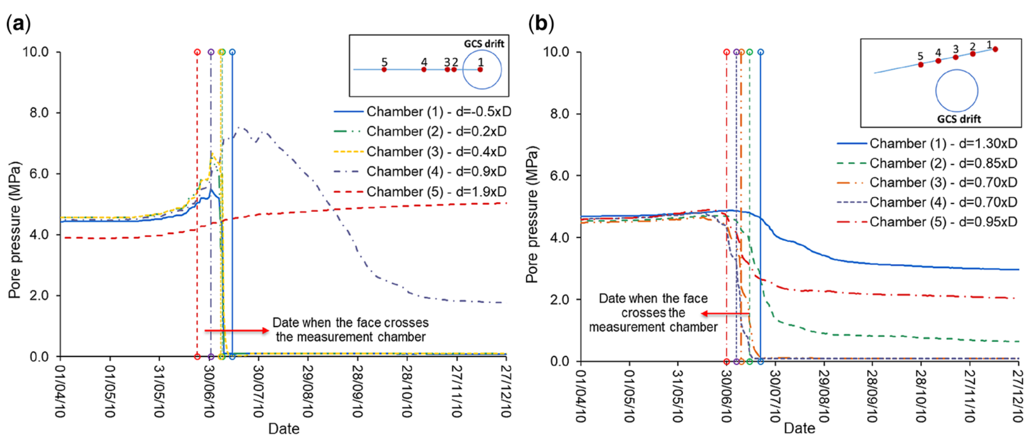

Fig. 6. Pore pressure evolution during excavation in the horizontal (**a**) and vertical (**b**) directions as a function of time around a gallery (GCS) parallel to the major horizontal stress (*d*, distance to the wall; *D*, drift diameter; chambers 1–5 are the locations of the intervals in which the pore pressure is measured at a distance, *d*, from the wall). Source: Vu *et al.* (2020).

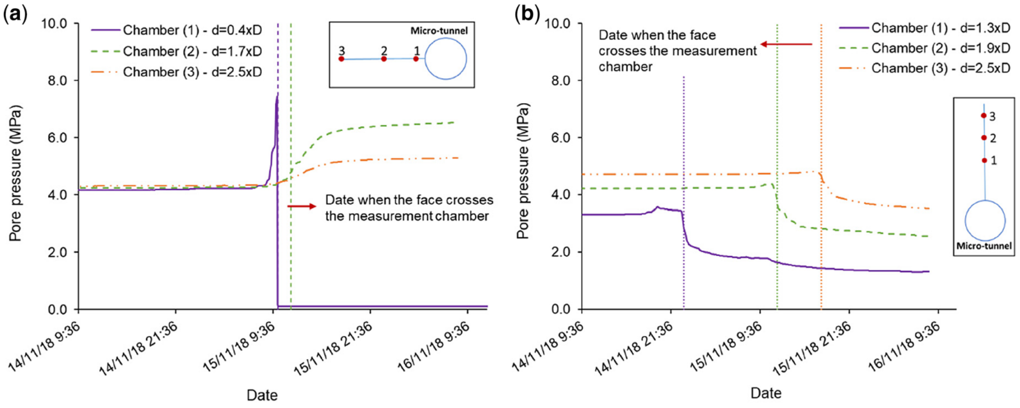

Fig. 7. Pore pressure evolution in the horizontal (**a**) and vertical (**b**) directions during and after excavation of a micro-tunnel parallel to the major horizontal stress (ALC1605) 0.8 m in diameter (D) (d, distance to the wall; D, drift diameter; chambers 1–5 are the locations of the intervals in which the pore pressure is measured at a distance, d, from the wall). Source: Vu *et al.* (2020).

shown by Vu *et al.* (2020) through theoretical and numerical modelling. In particular, *in situ* observations suggest that the effects of non-linear behaviour of the rock mass and of the anisotropic fracture network around the excavation might significantly influence the evolution of the pore pressure field.

In HADES, Bernier *et al.* (2002) show that during the excavation of the Connecting Gallery (CLIPEX project), a progressive increase in the porewater pressure was observed ahead of the excavation front followed by a sharp drop as the excavation front approached very closely the measuring section (Fig. 8a). The pressure response and mechanical displacement are strongly coupled. The increase in the pore pressure corresponds to the undrained contractant behaviour of the clay. The drop phenomenon is linked to the dilatation of the rock, and also to the fracture ahead of the excavation front. The high decompression of the formation near to the excavation face generates porewater suction (negative pore pressure). One important finding was the occurrence of measurable hydraulic effects at a distance of about 60 m (12.5 tunnel diameter) ahead of the tunnel excavation.

Figure 8b shows the pore pressure evolution during the PRACLAY gallery excavation (Li *et al.* 2010; Van Marcke *et al.* 2013). As in the Callovo-Oxfordian claystone, overpressures can be seen at the wall in the bedding plane and a drop of pressure under and above the gallery. This shows compression in the bedding plane related to the mechanical anisotropic behaviour of the Boom Clay.

Wileveau and Bernier (2008) show some comparison on convergence between HADES and CMHM. The observations show that convergence continued over time during the experiments and did not stop during the experiments. The evolution of convergence follows an evolution in $(1/t^n)$ (with t time and n a positive number) as shown in Guayacán-Carillo *et al.* (2016). The consequence of this convergence evolution is the loading of the concrete support lining installed by traditional or tunnel boring machine methods. Dizier *et al.* (2023) show a globally similar loading of the lining in the Boom Clay and the Callovo-Oxfordian claystone, often with anisotropic stresses evolving slightly but continuously in the long term (over a period of 20 years at HADES).

The fact that two different laboratories in two different clay formations using similar but different instrumentation devices show similar hydromechanical processes reinforces our confidence in the reliability of our results and the understanding of these results.

The excavation damage zone

In Callovo-Oxfordian claystone as in Boom Clay, no tectonic fractures were observed in the full print of the URL and even in a larger area. However, fractures were systematically observed at the front (Fig. 9) and sidewalls of the galleries at the main level of the CMHM and in HADES, meaning that these are induced by the excavation work during the construction of the galleries. At the CMHM with the conventional method, the front face is reinforced with 12 m length glass fibre bolts which stabilize the chevron fracture network and limit the fall of the rock block, forming a corner delimited by the chevrons in the centre of the gallery. In Figure 9a, the front was not reinforced.

The change in stress around the opening induced by excavation leads to a stress larger than the strength of the claystone (here the uniaxial

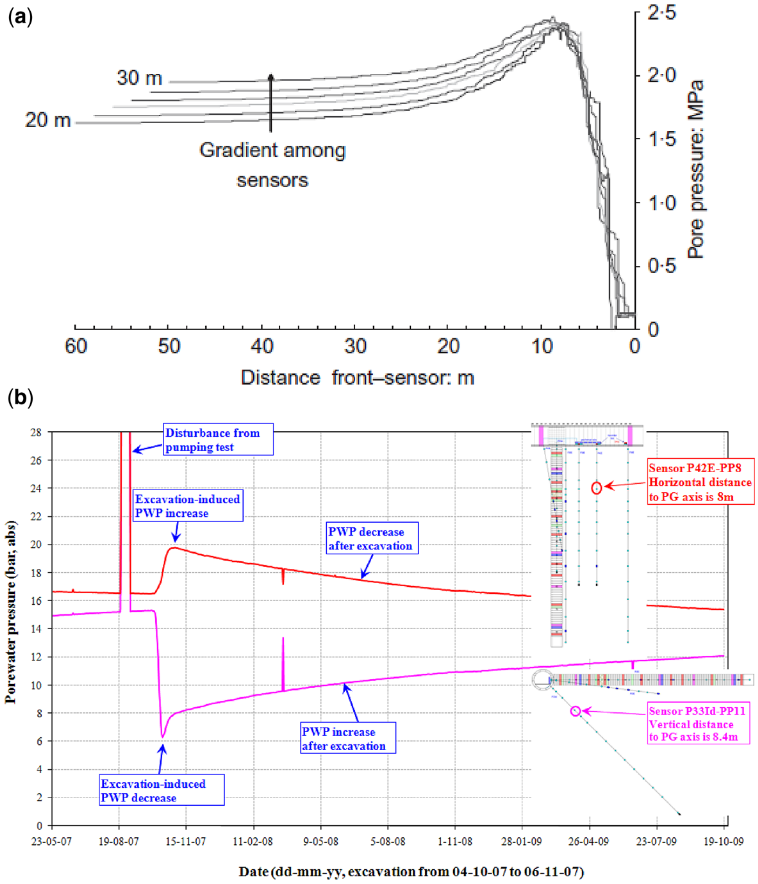

Fig. 8. Pore pressure evolution ahead of the front face in CLIPEX (Bernier *et al.* 2007; distance to the front face of the test drift) and in the section of the PRACLAY gallery (PG) at around 8 m on the side wall and underneath (Van Marcke *et al.* 2013). PWP, porewater pressure.

compressive strength $\sigma_c \approx$ USC) and then fractures appear. At the CMHM considering isotropic linear elasticity, the ratio $r = \sigma_\theta/\sigma_c$ can be determined with the maximum orthoradial stress (σ_θ) at the tunnel wall calculated by $\sigma_\theta = 2\sigma_v$ (where σ_v is the vertical stress) for an isotropic stress state, and

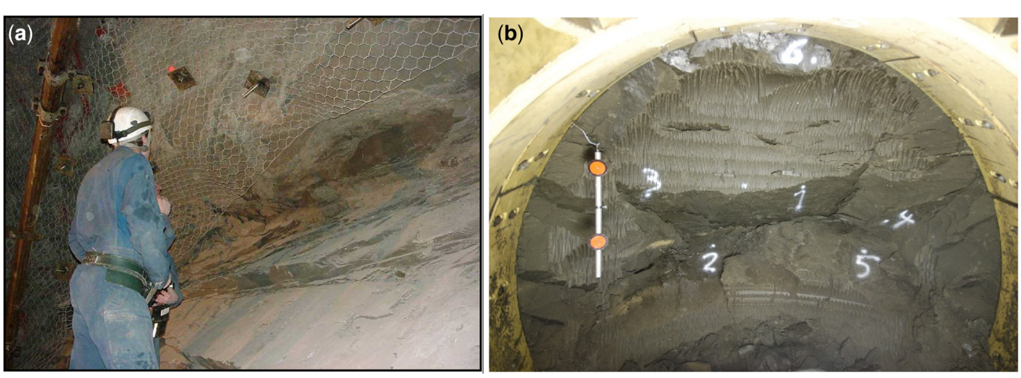

Fig. 9. (**a**) View of a front with chevron fractures at the main level of the CMHM (Armand *et al.* 2014) and (**b**) view of a front in HADES. Source: ©EIG EURIDICE.

$\sigma_\theta = \max\,((3\sigma_V - \sigma_h); (3\sigma_h - \sigma_V))$ for an anisotropic stress state, with σ_c being the uniaxial compressive strength. Looking at the total stresses, the ratio r is less than 1 in the USC facies (carbonate-rich unit), whereas in the UA facies (clay unit) it is greater than 1.1 (with $r \approx 1.7$ for the galleries oriented along σ_h and 1.1 for those oriented along σ_H). This significant ratio difference between USC and UA facies is related to the much higher strength of USC (at least 50% higher) owing to the higher carbonate content. As a direct consequence, the nature of the damage induced by the excavation is different between USC and UA, with much smaller damage in USC, more akin to micro-cracking.

In the clay unit (UA), given the magnitude of the r-ratio, there is much greater and more extensive fracturing in the ground with the appearance of shear and extension/traction fractures. An extensive characterization of the excavation-induced fracture network was performed based on side wall analysis (Fig. 10a) and systematic analysis on a cored borehole (Fig. 10b). Armand *et al.* (2014) showed that most of the excavation-induced fractures (75%), mainly at the front, appear in mode II or III (shear fracture) and 25% are in mode I at the main level of the URL; these were not observed in drifts at the −445 m level. Excavation with road header or hydraulic hammer does not change the extension of the induced fracture. The support method plays a role in the extent of the excavation fractured zone. In drift GRD excavated with a road header under a shield, the concrete segment is a stiff support

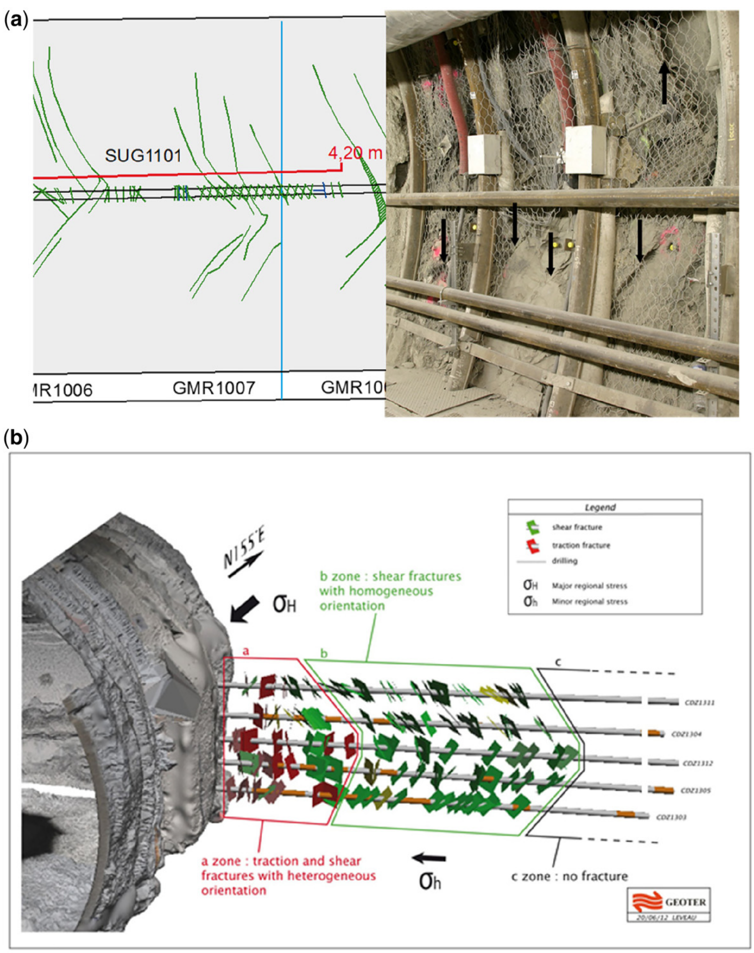

Fig. 10. Excavation-induced fracture pattern observed in the CMHM: (**a**) chevron fracture pattern observed at the wall of the gallery scale in relation to the face progression (left); and (**b**) 3D visualization of the fracture network in a gallery (GET) parallel to the major horizontal stress.

compared to sliding arches or shotcrete with yieldable concrete wedges. However, in a 'soft' support drift like GCS or GET, rock bolts and shotcrete, applied less than 1 m from the front face, start to work very early, while in a GRD drift the concrete segments start to work when the backfill material fills the whole gap at the roof at nearly 2 diameters of the front. This means that during the first steps of excavation, the rock wall at the roof is not supported, which implies larger deformation in the rock and larger extent of the induced fracture network. Armand *et al.* (2014) proposed a conceptual model for the excavation fracture network and also showed that the excavation fracture network pattern is similar on the scale of a borehole (10 cm in diameter) for a micro-tunnel 80 cm in diameter to a drift of 6 m in diameter. Guayacán-Carillo *et al.* (2018, 2019) confirm that there is no scale effect up to a 9 m diameter gallery.

In HADES, considering a uniaxial strength between 2 and 2.5 MPa (Coll 2005; Bernier *et al.* 2007), the ratio r is larger (between 3.6 and 4.5) which explains why the stress path (a decrease in the mean effective stress and increase of deviatoric stress) during excavation meets the strength criterion in the shear behaviour. At the CMHM, the stress path meets the shear criterion nearer to the tensile domain (Vu *et al.* 2020). In a poorly indurated clay like Boom Clay, the behaviour is more ductile and shear bands appear at a high deviatoric stress. During the construction of the Connecting Gallery (Bernier *et al.* 2002; Mertens *et al.* 2004), the observations allowed the fracture pattern in the surrounding formation to be determined. The orientation of the encountered fractures consists of two conjugate fracture planes: one in the upper part, dipping towards the excavation direction, the other in the lower part, dipping towards the opposite direction (Fig. 11). The two planes were curved and intersected at the mid height of the gallery. After the tunnel construction, cored boreholes indicated a radial fracture extent of about 1 m ($0.25 \times D$, where D is the gallery internal diameter). It is noteworthy that the observed herringbone fracture pattern is similar to the fracture pattern observed on a smaller scale along cores as a result of the drilling of these cores (Fig. 12).

Very similar excavation-induced fracture patterns around galleries and boreholes are observed in Callovo-Oxfordian claystone and Boom Clay. Shear herringbone fractures were observed ahead of the gallery excavation front up to half a diameter at the CMHM and up to about 1.5 diameters in HADES. In both clay formations, eye-shaped

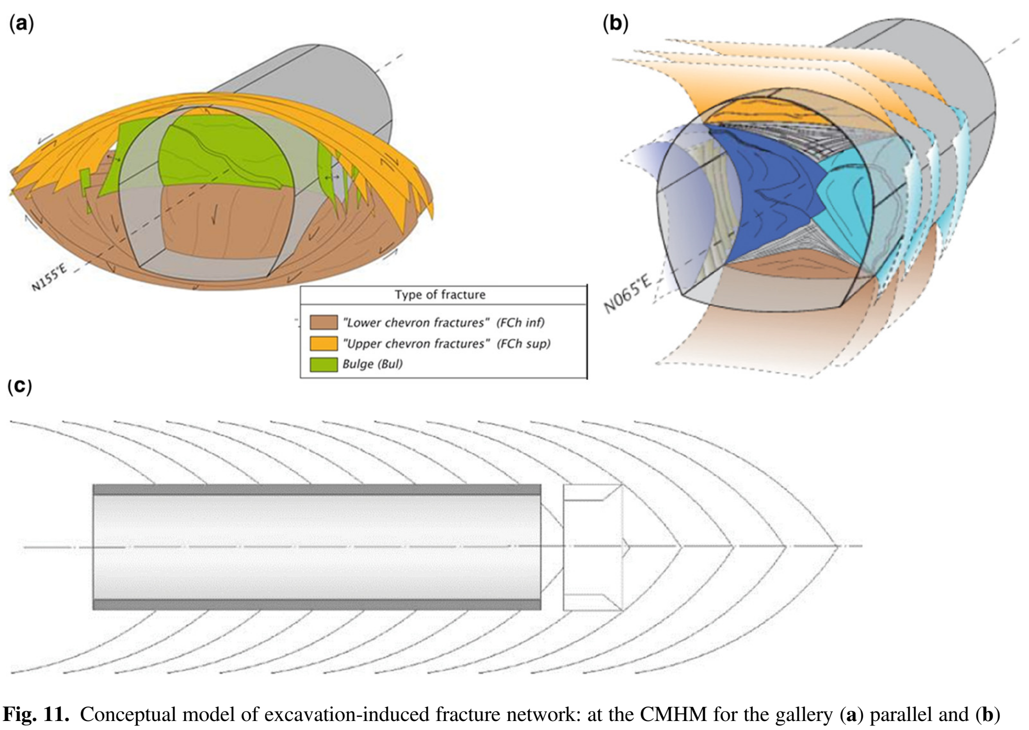

Fig. 11. Conceptual model of excavation-induced fracture network: at the CMHM for the gallery (**a**) parallel and (**b**) perpendicular to the major horizontal stress and (**c**) at the HADES URL with chevron fracture (diagram) ahead of the front face (Mertens *et al.* 2004).

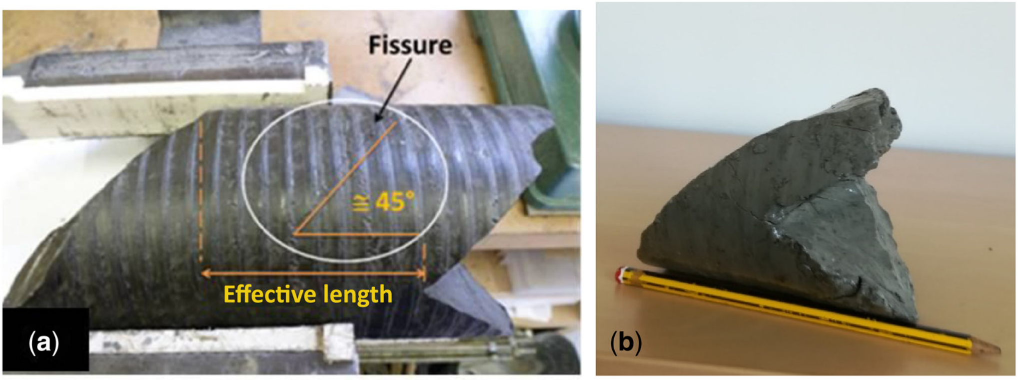

Fig. 12. Shape of core sample in (**a**) Boom Clay (Dao 2015) and (**b**) Callovo-Oxfordian claystone (ahead of the front face).

fracture patterns with shear fracture are observed despite differences in the hydromechanical response. Extensional fractures are not observed in Boom Clay. This difference could be explained by the brittleness of the Callovo-Oxfordian claystone compared with the more plastic/ductile behaviour of the Boom Clay. The ductility allows for large plastic deformations to be distributed throughout the clay, whereas with brittle materials, failure appears localized at low deformations and therefore several fractures appear for the material to have large deformations. With brittle response, there is also the instability effect that creates a kind of ‘chain reaction’ with the appearance of several fractures. Analysing the fracture density with depth from the wall in the fractured zone around galleries at the CMHM, more fractures are observed beside the gallery wall than inside the block because the Callovo-Oxfordian claystone exhibits a more brittle response under a small confining pressure. Surprisingly, cores can also have the same fracture pattern (Fig. 12). This shows that the fundamental processes are to some extent the same, with the importance of the hydromechanical coupling in a nearly undrained condition owing to the low permeability during the excavation.

Another important aspect for long-term safety (Blümling *et al.* 2007) is the sealing/healing process which has been extensively investigated on Boom Clay in the laboratory and *in situ* in the framework of SELFRAC (Vervoort *et al.* 2006; Bernier *et al.* 2007).

Similar tests were performed in Boom Clay and Callovo-Oxfordian claystone. An artificial fracture was created in a sample (Bernier *et al.* 2006; Di Donna *et al.* 2019; Giot *et al.* 2019) then saturated with water. Different setups were used (constant volume cell or triaxial cell) and several levels of confining pressure were applied to the samples. Water injection into the sample induced very fast closure of the fracture even for low confining pressure. The main process involved is the swelling of clay on fracture lips (Davy *et al.* 2007). The closure was observed using advanced imaging techniques such as micro computed tomography (see Fig. 13). The main lesson learned from these tests is the demonstration of a self-sealing capacity of Boom Clay and Callovo-Oxfordian claystone taken from the clay-rich level. The water permeability measured on the sealed material is close to that measured on the undisturbed material.

The main difference between Callovo-Oxfordian claystone and Boom Clay is the time scale of closure. As shown in Zhang (2015) on Callovo-Oxfordian claystone, a fast decrease in water permeability happens when water comes into contact with the clay in the fracture corresponding to swelling. This phase is followed by a low decrease in permeability corresponding to a rearrangement of clay structure in the fracture and to creep. After both phases, the water permeability approaches that of undisturbed material. In Boom Clay, the water permeability reduction, and the fracture closure, are faster. These differences are mainly related to the nature of the clay, which is in one case a poorly indurated clay (Boom Clay) and in the other an indurated clay (COx). Note that no healing is observed for Boom Clay and Callovo-Oxfordian claystone; the memory of the fracture after the self-sealing is still present and can be reactivated by hydromechanical processes. This can be demonstrated through gas breakthrough tests. The gas breakthrough pressure is much lower in the fractured and self-sealed material than in the undisturbed material (M’Jahad *et al.* 2016). Observation of the microstructure in the fracture after self-sealing showed differences, mainly owing to the presence of macropores (Gonzalez-Blanco *et al.* 2022).

De La Vaissière *et al.* (2015) showed that the fracture network induced by excavation at the wall of the gallery GET (see Fig. 10) was initially interconnected and opened for gas flow (particularly

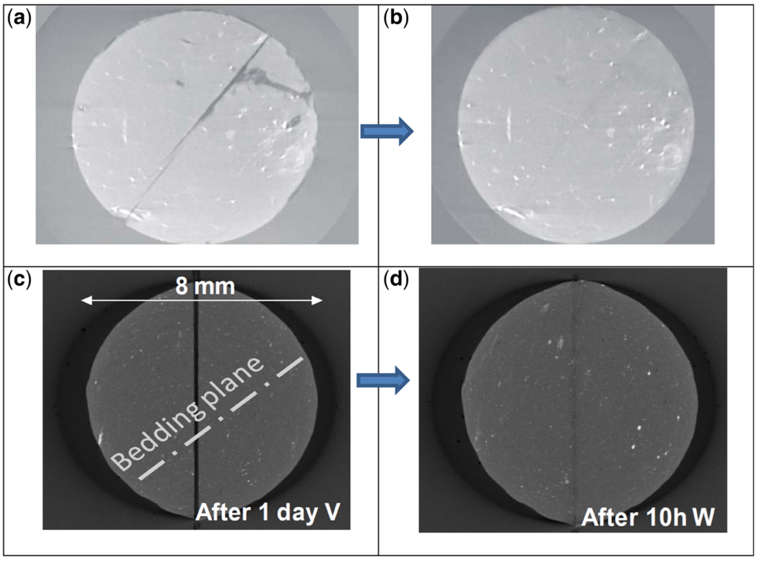

Fig. 13. Visualization of the self-sealing process by the micro computed tomography technique after saturation of the fracture: (**a**, **b**) in Boom Clay (Bernier *et al.* 2006); (**c**, **d**) in Callovo-Oxfordian claystone (Di Donna *et al.* 2019).

along the drift) and can be progressively closed when a mechanical loading is applied to the drift wall. Moreover, the evolution of the excavation fracture network after unloading indicated an irreversible and partial closure of the fractures. Following this loading/unloading stage, the remaining fracture network was resaturated to demonstrate the ability of self-sealing of the Callovo-Oxfordian claystone without mechanical loading. Transmissivity/conductivity of the fracture network was evaluated by conducting repetitive hydraulic tests. During this hydration process, the effective transmissivity of the excavation fracture network dropped owing to the swelling of the clay minerals surrounding the fractures (Fig. 14). The hydraulic conductivity evolution was relatively fast during the first few days. Low conductivities ranging up to 10^{-10} m s^{-1} were observed after 4 months. Cross-hole tests showed the disappearance of the preferential network interconnectivity pathway along the drift axis (major pathway for performance assessment purposes, owing to its potential to act as a seal bypass), but heterogeneous conductivity effects persisted. This experiment is one of the *in situ* tests performed in ANDRA's URL, showing the self-sealing capacity of the Callovo-Oxfordian claystone at large scale.

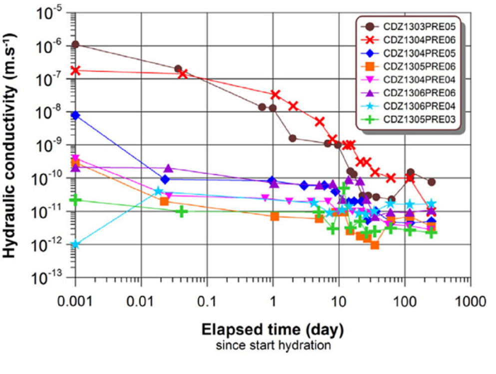

Fig. 14. Evolution of the hydraulic conductivity since the start of hydration for the repetitive hydraulic tests at the CMHM in the Callovo-Oxfordian claystone (de La Vaissière *et al.* 2015).

While the sealability of fractures is widely acknowledged for poorly indurated clays owing to their plasticity/deformability, this process for stiffer clay rocks is less expected. Tests at sample scale and *in situ* show that this process is rapid for the Boom Clay and also occurs in the Callovo-Oxfordian claystone and Opalinus Clay, but with slower evolution. The comparison between the behaviour of these different clay formations and the understanding of the small-scale processes gives confidence in the control of the phenomena.

Seal experiments

Repository closure is an important issue and requires the installation of seals in several places (galleries, shafts, access ramps) to limit water flow and radionuclide transfer from the disposal cells to the biosphere. In the context of underground radioactive waste repositories, pure bentonite or bentonite/sand mixtures are widely used both for sealing and for the engineered barriers installed close to the waste (Sellin and Leupin 2013).

In addition to the characterization of the thermo-hydromechanical behaviour of these materials, questions arise concerning their installation in tunnels and shafts. For instance, a seal in Cigéo is about 10 m in diameter and 20 m long. This means that a large-scale programme has to be developed to improve knowledge about physical processes occurring in these materials, in particular during hydration, but also to be able to test at a large scale the installation of bentonite-based components.

Several experimental programmes carried out in the HADES URL since the end of the 1990s have provided valuable data for the design/construction of seals, for the knowledge about physical processes and for the subsequent implementation of experiments in the CMHM.

A wide range of technologies is available to build and install all the components of the seals in the tunnels and some of them have been tested in HADES. Feedback from the installation of swelling clays in the form of compacted blocks or in the form of powder/pellet mixtures has helped to extend the range of solutions in relation to the performance expected for these structures. In particular, the use of pellet mixture in the RESEAL project and the techniques applied, such as X-ray tomography (Fig. 15a), to evaluate the evolution of properties in an initially heterogeneous material contributed to better understanding of the processes involved during the saturation of granular bentonite materials (Van Geet *et al.* 2009). Data obtained contributed to the improvement of models in which interactions between micro- and macro-porosity are essential. This topic related to the initial heterogeneities in bentonite-based components is still being studied within the framework of the European Beacon project (Sellin *et al.* 2020) and internal programmes conducted by ANDRA.

One of the technical solutions studied by ANDRA is to install the seals using pellet/powder mixtures. The capacity to build a seal with this method was demonstrated at a large scale in the

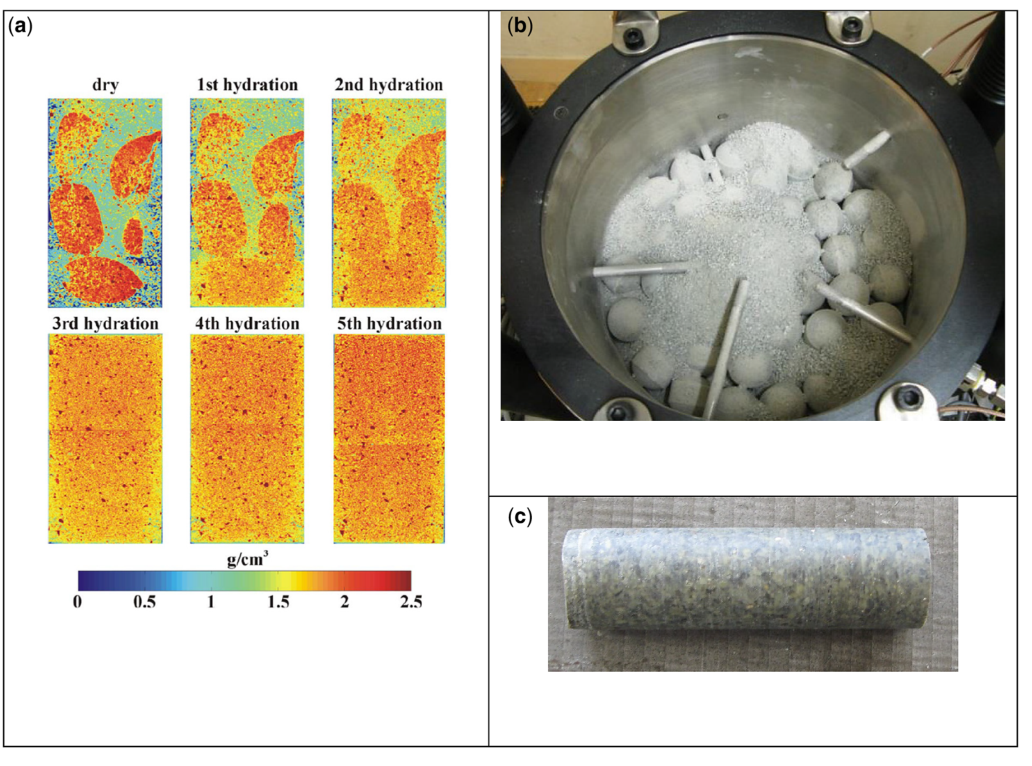

Fig. 15. (**a**) Micro computed tomography of a pellet mixture in the RESEAL project (Van Geet *et al.* 2009); (**b**) pellet/crushed pellet mixture used in the full scale seal (FSS) project; and (**c**) FSS mixture after hydration (Bernachy-Barbe *et al.* 2020).

FSS (full scale seal) demonstrator developed during the EC project DOPAS (Hansen *et al.* 2016). In this demonstration test, a mixture composed of bentonite pellets of 32 mm diameter and crushed pellets was installed.

Several small-scale experiments were performed to evaluate the swelling pressure and the permeability of the mixture (Fig. 15b). These tests included the analysis of dry density distribution after hydration. After dismantling the samples, it was no longer possible to see the initial granular structure (Fig. 15c) as observed in the RESEAL tests. Nevertheless, precise analysis of dry density (Bernachy-Barbe *et al.* 2020) showed a non-uniform profile of dry density along the sample height. This was confirmed by several tests such as those performed by Ciemat (Villar *et al.* 2021) on other kinds of swelling materials. These tests considered the evolution of the two layer materials: one layer consisted of pellets and the other of a compacted block. This configuration is representative of real situations where initial gaps between compacted blocks and the host rock are most often filled with a pellet/powder mixture. The initial contrast of dry density is high (1.6 g cm^{-3} for blocks and 1.3 g cm^{-3} for pellet mixture). After hydration, even if there is an attenuation of the dry density contrasts with a much more homogeneous system, there are still differences between the two layers.

The dismantling of large-scale tests such as the EB in Mont Terri (Mayor *et al.* 2005) or FEBEX in Grimsel (Villar and Lloret 2007) and further analysis of dry density in bentonite also indicated non-uniform distribution of dry density. Nevertheless, the consequences for the expected properties with regard to the function and the performance of these bentonite components are quite small and do not compromise the correct functioning of bentonite-based components in a repository.

One of the main issues associated with seals is how the gas migrates in and around the bentonite components after hydration. Gas injection tests in the RESEAL borehole performed in HADES yielded valuable information about gas pathways around a bentonite seal. The gas breakthrough occurred at a pressure level of about 3.1 MPa. This pressure is close to the total radial stress measured in the bentonite seal. The pressure sensor responses showed clearly that the gas did not flow through the bentonite seal. However, some evidence was found that gas flowed along the interface between the bentonite seal and the host rock or through the borehole EDZ of the host rock (Van Geet *et al.* 2009). A similar test was performed in the CMHM in the PGZ2 programme. The PGZ2 experiment is a set of five horizontal boreholes drilled between two galleries to allow access from both sides and to avoid inserting sensor equipment through the bentonite plugs. Two kinds of materials were tested: (1) pre-compacted bentonite plugs made with a mixture of MX80 and sand (70/30% in mass) with a dry density of about 1.8 g cm^{-3}; and (2) a pellet/powder mixture made with MX80 directly built in place, with a dry density of about 1.54 g cm^{-3}. The resaturation of the bentonite plugs was only done by natural water from the host rock without any kind of water injection. In each borehole, multiple pore pressure and total pressure sensors were installed to track as closely as possible swelling pressure evolution and water saturation.

In one of the boreholes (PGZ1013), the bentonite–sand mixture full saturation was reached after about 18 months. The hydraulic conductivity of the seal was close to 2–4 10^{-13} m s^{-1} and the swelling pressure measured axially was about 4.6–5 MPa. This value is slightly lower than the target swelling pressure (7 MPa). This is due to the drilling which leads locally to a larger borehole diameter and to some residual voids at the base of the bentonite plug. The volume available for bentonite swelling is then larger than the theoretical one. These values are consistent with measurements performed on samples in constant volume cells for a dry density around 1.7 g cm^{-3}. A gas test was performed to evaluate the efficiency of the seal and the surrounding claystone. The gas breakthrough occurred at a pressure level between 0.5 and 1.5 MPa above hydrostatic pressure and well below the swelling pressure of the seal measured axially. The main conclusion of this test is that the most probable location of the gas pathway is the interface between bentonite plug and claystone or the borehole damaged zone itself. Several authors have shown that in damaged claystone, even if self-sealing occurs, the gas entry pressure can be significantly lower than that for sound claystone even with confining stress (M'Jahad *et al.* 2016).

The results obtained from the PGZ2 programme are consistent with those from the RESEAL borehole tests. The main lesson is that at the repository scale, gas could be released through the interface and damaged zone along the repository drifts without any disturbance of the seal. Many arguments now seem to show that gas breakthrough pressure in these zones is much lower than in intact clay/claystone or in a bentonite plug.

HADES feedback to the ANDRA research programme

In the 1980s, HADES was the first URL dedicated to the study of the geological disposal of radioactive waste in a deep clay formation and in this sense it inspired all other URL developments, especially those in claystone like the CMHM and the Mont Terri URLs. ANDRA especially relied heavily on the work carried out in HADES, through numerous co-operation projects (participation in *in situ* experiments) both directly between ANDRA and ONDRAF/NIRAS and SCK CEN (EURIDICE), and within European projects (CLIPEX, NF – PRO, RESEAL, etc.). This was driven by a dual objective: (1) to prepare CMHM-specific experimental programmes (methodology, experimental devices and protocols, etc.); and (2) to improve general knowledge on the behaviour of clay/claystones, in particular in terms of similarity and differences between the various clay/claystone formations (indurated or non-indurated clays).

The experience gained by HADES (and also at Mont Terri URL) in the construction of underground structures in clay at great depths, the instrumentation for the hydromechanical monitoring of structures and around openings and the implementation of tests in relation to sealing has enabled the CMHM to be developed more rapidly and to provide feedback for the development of the Cigéo project. HADES and the CMHM show that different methods, also with road header under a shield, are effective to excavate and support galleries in clay/claystone.

Despite the differences in responses between the plastic poorly indurated Boom Clay and the stiffer indurated Callovo-Oxfordian claystone, several processes show similar trends. Of special interest is the hydromechanical behaviour, with a strong coupling between mechanics and hydraulics leading to pore pressure evolution during excavations in a quasi-undrained condition (linked to the low permeability of the clay/claystone). Dilatant/contractant volumetric strain leads to changes in pore pressure, even at a significant distance from the excavated structures. Shear fractures appear at the front and/or ahead of the front face owing to stress changes regarding the strength of the clay/claystone. Although the fundamental processes are to some extent the same, their relative importance and response rates are different. Another significant aspect is the evolution in the properties of the damage zone. After water saturation, a rapid drop in permeability is observed in both clays/claystones, even if the rate decreases faster in the Boom Clay. This comparison imparts robustness to the understanding of the processes leading to self-sealing of fractures in clay/claystone.

The long-term behaviour of the structures at the CMHM shows that the convergence of the galleries continues with low but not zero velocities, which leads to the loading of the concrete linings. The analysis of these data is important for the design of Cigéo. The duration of the monitoring of the hydromechanical behaviour at HADES is much longer but still shows loading of the structures, highlighting the importance of understanding the long-term behaviour.

Data acquired from bentonite seals at HADES yielded valuable knowledge about swelling bentonite behaviour and gas pathways around a bentonite seal. These data are useful for demonstrating the robustness of the seals and shaping the demonstration programmes implemented by ANDRA.

Competing interests 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, except from the agencies to which the authors are affiliated.

Author contributions **GA**: writing – original draft (lead); **FP**: writing – original draft (supporting), writing – review & editing (equal); **JT**: writing – original draft (supporting), writing – review & editing (equal); **AD**: writing – original draft (supporting), writing – review & editing (equal); **XLL**: writing – original draft (supporting), writing – review & editing (equal); **SL**: writing – original draft (supporting), writing – review & editing (equal).

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Data availability Data sharing is not applicable to this article as no datasets were generated or analysed during the current study.

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