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HYDROMECHANICAL RESPONSE TO A MINE BY TEST EXPERIMENT IN DEEP CLAYSTONE

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ABSTRACT
In order to demonstrate the feasibility of radioactive waste repository in deep geological formation, an underground research laboratory is being constructed by Andra (French national radioactive waste management agency) in eastern France, in a Callovo-Oxfordian claystone. 15 boreholes were drilled from a drift at -447 m to install sensors around the shaft (6 m diameter) at depth -460 m to -474 m in order to record the hydromechanical behaviour of the claystone during a shaft sinking (drill and blast method). This paper is devoted to the analysis of the mechanical and hydromechanical behaviour observed during the shaft sinking. Analytical approach used herein allows to realistically evaluate the undrained response of the shaft neighboring with agreement with the in situ measurements. For the transient phase, prediction is qualitatively in accordance with measurement. In addition, deformation and displacement measurements are successfully compared to a simple 3D elastic calculation performed with the real face advancing. This emphasizes the quality of the data set which would be compared in the Modex-Rep European project with complex numerical modelling (poro-elasto-plasic-damage models, creep behaviour, …).

RÉSUMÉ
En France, l'Andra (Agence Nationale de gestion des déchets radioactifs) est en charge des études pour la faisabilité d'un stockage de déchet radioactif haute activité à vie longue, dans une formation géologique profonde comme des argiles. Pour cela, l'Andra a construit le laboratoire de recherche souterrain de Meuse Haute Marne dans le Nord-Est de la France (à 300 km environs de Paris) dans une formation d'argilite du Callovo-Oxfordien qui se trouve entre 420 m et 550 m au niveau du puits principal. La première expérimentation géomécanique réalisée dans le laboratoire est un « mine by test » autour du puits principal (diamètre 6 m). A partir d'une galerie se trouvant à -445 m, 15 forages ont été réalisés pour installer des capteurs et mesurer le comportement hydromécanique de l'argilite entre -460 et -470 m lors du creusement du puits principal. Les évolutions des mesures in situ sont présentées et comparées avec la solution analytique poro-élastique du creusement d'un puits infini. Les mesures de déplacement sont comparées avec les résultats d'un calcul élastique en 3D. Ces analyses simples montrent la cohérence et la qualité des différentes mesures in situ qui serviront de données de référence dans le projet Européen ModexRep, où le creusement du puits est simulé avec des modèles 3D complexes (poro-elasto-endo-plasticité, fluage, ….).

1. INTRODUCTION

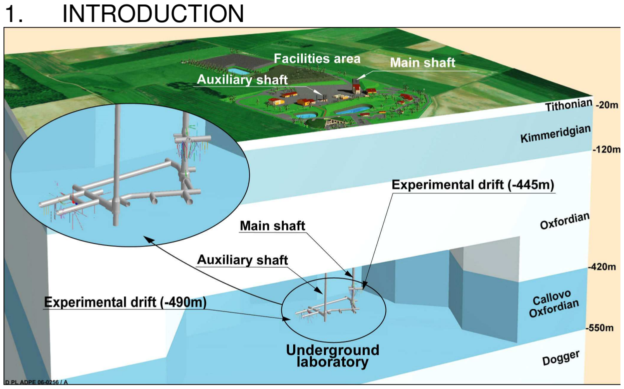

Figure 1. 3D view of the Meuse/Haute-Marne Underground Research Laboratory

In order to demonstrate the feasibility of a radioactive waste repository in the claystone formation, the French national radioactive waste management agency (Andra) started in 2000 to build an underground research laboratory (URL) in the village of Bure (near the boundary between the Meuse and Haute-Marne Departments) at nearly 300 km East of Paris. The host formation consists of a claystone (Callovo-Oxfordian argillites) and is approximately 500 m deep and 130 m thick (Andra 2005a). The Callovo-Oxfordian argillites are overlaid and underlain by relatively impermeable carbonate formations. The thermohydromechanical behaviour of the argillites and the EDZ are ones of the key issues being investigated in many underground experiments carried out or planned in the Meuse/Haute-Marne URL (Su *et al.* 2000, Piguet 2001, Delay *et al.* 2005). Figure 1 presents the drifts network in the Meuse Haute-Marne URL.

Among those experiments, 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 argillite formation to shaft sinking.

The investigated zone is located between 460 and 474 m in depth, where the secant modulus of the argillites is about 5600 MPa and the mean value of the uniaxial compressive strength is 21 MPa. The strain at failure during compression tests on argillite core samples is about 1 to 1.5%. *In-situ* stress in the argillite layer is: $\sigma_z = \gamma Z$; $\sigma_h \approx \sigma_v$, σ_H / σ_h close to 1.2, with the exact value varying with depth and with the rheological characteristics of the respective layers. The horizontal major stress is oriented NE155° (Wileveau *et al.* 2005).

This paper recalls the concept and instrumentation of the REP experiment, and presents the hydromechanical in situ measurements and the first analysis of the results focusing on pore pressure and displacements in the adjacent massif. First analysis are based on analytical solution of the sinking of an infinite shaft in an anisotropic stress field and simple elastic 3D calculations

2. DESIGN OF THE REP EXPERIMENT

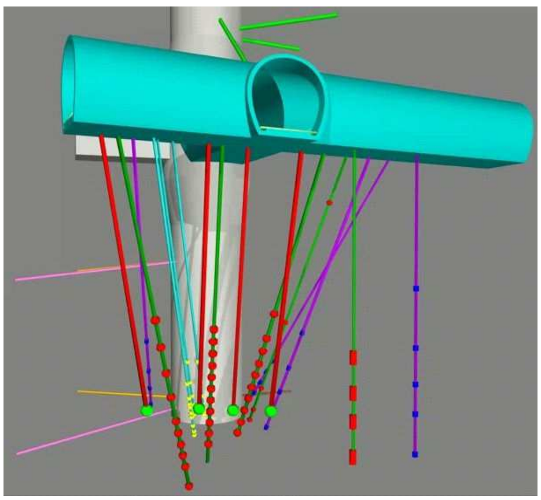

Figure 2. Layout of the REP experiment

In order to monitor the hydromechanical behaviour of the argillites around the REP zone (–460 to –476 m), 15 instrumentation boreholes (20 to 30 m depth) were drilled from an experimental drift (–445 m) in December 2004 (Figure 2). About 120 hydromechanical sensors are placed in these boreholes as follows: 20 for pore pressure, 20 for displacements, 4 CSIRO 12-gauges cells for strains, 10 for inclination and 27 piezoelectric sensors (figure 2). Furthermore, two radial boreholes were specifically drilled at –467 m to measure radial-displacement behind the excavation face. At this depth the convergence of the shaft has although been measured along 6 diametral positions. These sensors constitute a monitoring matrix for the hydromechanical behaviour of the argillites. In this paper, displacements and pore pressures are discussed. Further in situ data, like evolution of velocity during the shaft sinking could be found in Armand and Su (2006).

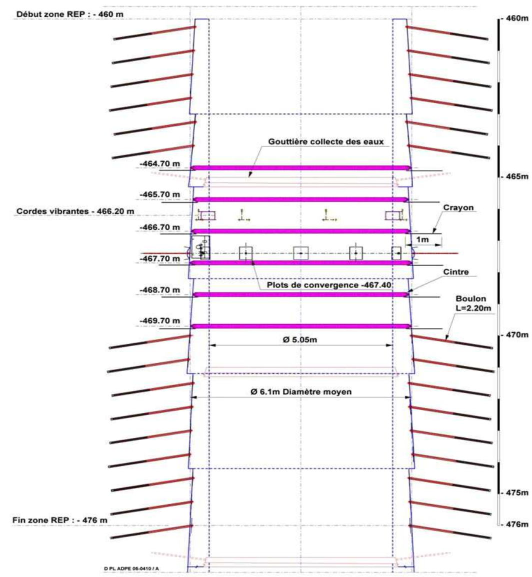

Figure 3. Cross-section of the access shaft (6 m diameter) between –460 m and –475 m

2.1 Step of the experiment

The REP experiment consists of following steps:

- Step 1: Drift excavation at the –445 m level and instrumentation of the REP zone (July-December 2004): The drift is the starting point from which REP experiments are run. Sensors and measuring systems were installed in December 2004. Pore pressure chambers were emplaced first in order to have the longer pressure build up;
- Step 2: Initial characterisation through dilatometer tests and velocity measurements and stabilisation of sensors (December 2004-February 2005): a sensor-stabilisation phase was required for all sensors. Especially in the case of pore pressure measurements, the build up of pressure was quick and the pressures were nearly stabilized before the beginning of the shaft sinking;
- Step 3: Resumption of shaft sinking: shaft advancing resumed on 10 March 2005, with the instrumentation matrix monitoring in real-time the effects of the approaching working face.

In situ measurements described in this paper occurred during the shaft sinking from -451 to -483.3 m level. At 483.3 m the sinking operations were stopped middle of August 2005 in order to install other extensometers and to perform measurement. After the sinking started again to reach 510 m, but the hydromechanical parameters in the REP zone, are less affected and are not presented in this paper.

2.2 Shaft-excavation method

A 6.1 m diameter shaft was sunk with a drill-and-blast method. Blast pattern was designed to excavate rock over a height of 2 to 3.1 m. Figure 3 shows a shaft cross-section in the REP study zone.

The standard support system before application of concrete lining consists of 20, 2.2 m length, bolts with wire mesh and shotcrete in order to stabilise the rock wall and to prevent spalling.
In the main REP zone (between –464.7 and –469.7 m), the support system is replaced by six TH21/548 arches. Each arch is made up of five elements measuring 4.5 m in length. The arches are designed to slide with a friction force of 120 kN.
The final lining consists of a layer of at least 0.5 m of B45 concrete injected by 6-m plots at a time. The distance between the final lining and the working face ranges from 18 m to 24 m, or even 30 m in certain cases.

3. RESULTS OF THE EXPERIMENT

3.1 Displacement field in the rock mass

Figure 4 shows the evolution of displacements within the rock as measured in borehole REP2202 compared with the head of the REP2202 extensometer.
Before sinking resumed, measurements were stable, which means that there were no relative displacements between anchoring points. When sinking resumed, the shaft depth was 453.2 m and all anchorings of the REP2202 extensometer laid at least 6.20 m below that level. As the working face progressed towards the REP zone, an initial compression phase is observed, followed by an extension phase. As long as shaft sinking has not reached the anchoring level, the extensometer remains in compression. As soon as the shaft passes the anchoring level, the extensometer starts to expand.
This behaviour was expected because the displacement field located below the shaft-sinking face includes movements that are rather vertical due to void attraction and compress the extensometer, whereas anchorings located at right angle with the sunk shaft are subject to the convergence effects of the shaft and generate an expansion of the extensometer.
At every blast, an instantaneous displacement is measured over the extensometer. During mucking, the recorded displacements are lower in amplitude.
At –480.3 m, increments of instantaneous displacements start to be very low, but an increase varying between 0.5 and 0.8 mm was observed between 15 July and 1 September 2005. The displacements measured during that period are not induced by the sinking of the shaft, but rather to its convergence.
It is interesting to point out that the radial distance between the first (REP2202_DFO_01) and the last (REP2202_DFO_10) measurement points is 3.7 m. The last point is located at 3 cm from the shaft wall. By projecting the relative displacement horizontally along the borehole, it is possible to observe, after the passage of the working face, that the relative horizontal displacement between points REP2202_DFO_01 and REP2202_DFO_10 stabilises at 0.17 mm in compression. After the passage of the working face, a maximum relative displacement in compression reaching 3 mm between the two sensors is recorded.

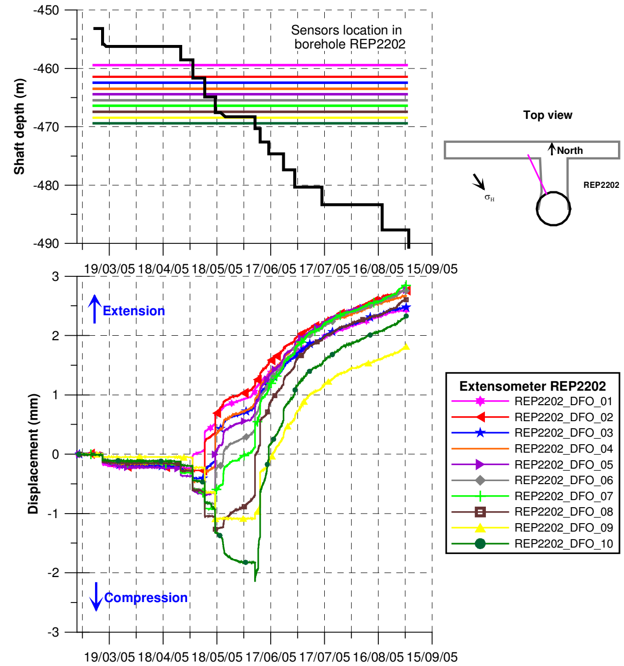

Figure 4. Measured displacements in borehole REP2202

By analysing the relative displacement between points REP2202_DFO_09 and REP2202_DFO_10, a compressive deformation of 3.4e-4·on the horizontal plane is reached after the passage of the working face. When the face has passed the deepest anchoring level, a relative displacement in extension is observed.

After the passage of the working face, measurement points do not return to their initial positions. A permanent extension is observed on all sensors. However, only very slight variations were detected concerning the relative positions between sensors REP2202_DFO_1 to 8, which reflects a quasi-elastic behaviour of the rock where these sensors are located. The behaviour of REP2202_DFO_9 and 10 is different from the others. Their positions close to the shaft wall (0.48 and 0.83 m, respectively) suggest that anelastic deformations occur within the argillite close to the shaft wall.

The measured displacements in borehole REP2201, oriented in the σ_h direction, are similar to those for borehole REP2202. A significant deferred displacement of that borehole head is observed, thus complicating considerably the interpretation of measurements collected in that borehole.

3.2 Evolution of pore pressure during shaft sinking

Figure 5 shows the evolution of pore pressure in borehole REP2101 during the sinking of the access shaft. The borehole is oriented in the direction of the major horizontal stress. The distances between the measurement sensors and the wall range from 1.8 to 5 m.
Before sinking resumed, pore pressures were almost stable at 3.4 to 3.9 MPa. On 8 July, a pulse test was

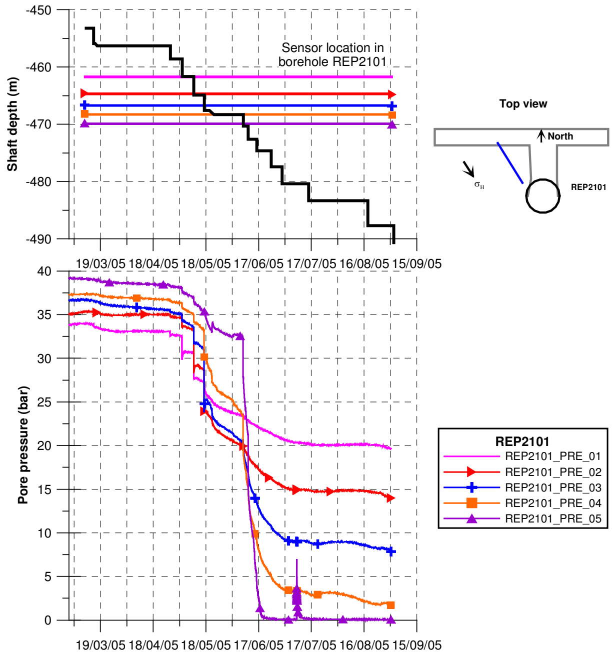

Figure 5. Evolution of pore pressures in borehole REP2101 (chambers at 1.8 m to 5 m from the wall shaft)

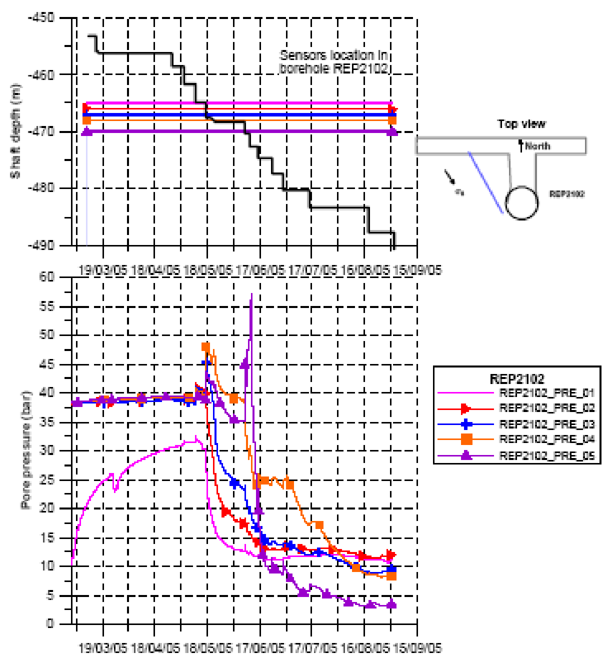

Figure 6. Evolution of pore pressure in borehole REP2102 (chambers at 1.1 to 2.3 m from the shaft wall)

carried out in Chamber No. 5 in order to measure the permeability of that desaturated chamber.

The first blast generates a small reduction in pressure, followed by a new stabilisation. This variation has an amplitude of 0.01 MPa for the furthest pressure-measurement chamber from working face and 0.09 MPa for the nearest. Successive blasts induce an instantaneous variation in the pore pressure.

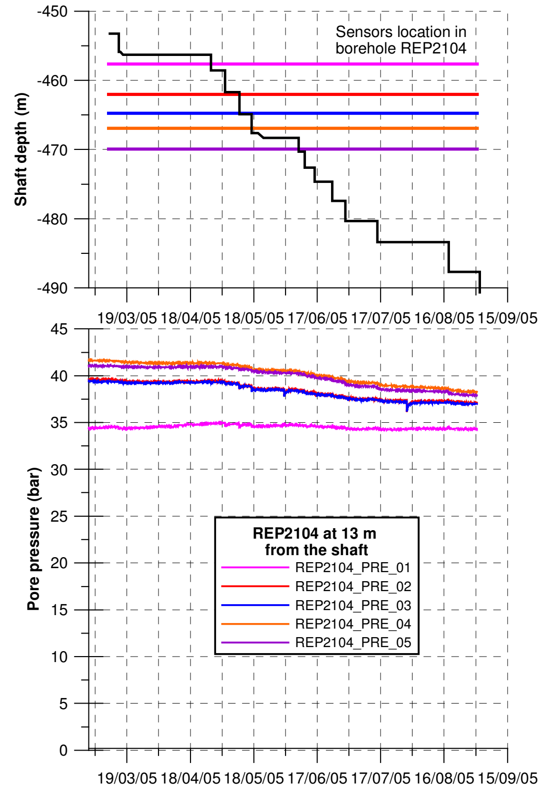

Figure 7. Evolution of pore pressure in the vertical borehole REP2104 at 13 m from the shaft wall

The amplitude of those jumps increases in proportion with the proximity of the working face with the chamber level. The most significant reduction is observed when shaft sinking passes the level of the measurement chamber and it increases when the wall is close to the chamber. It varies between 0.29 MPa in Chamber No. 1 at 4.85 m from the wall and 0.69 MPa in Chamber No. 5 at 1.7 m from the wall. This type of coupled hydromechanical behaviour is systematically observed in all the boreholes located in the vicinity of the shaft.

In the case of borehole REP2101, after the passage of the level of each chamber, they all show a decrease in pore pressure. Pressures in the chambers tend to stabilise in August 2005 when sinking operations stopped at a depth of 483.36 m. Pressure distribution depends on the distance from the shaft, and pressures range from atmospheric pressure for the chamber located at 1.7 m from the wall to 2 MPa for the furthest chamber from the shaft (at 4.85 m from the wall) as clearly illustrated in figure 5.

In borehole REP2102 (figure 6) oriented in the direction of the minor horizontal stress versus the shaft, high overpressures are observed in chambers before the face reaches the chambers level. Those overpressures increase in proportion to the distance of the measurement chambers from the shaft wall. They may vary from 0.06 MPa in Chamber No. 1 at 2.25 m from the wall to 1.6 MPa in Chamber No. 5 at 1.09 m from the wall. As

soon as the working face passes the chamber level, the pore pressure begins to decrease.
Pressure in the chambers tends to stabilise in August 2005 when sinking operations stopped at a depth of 483.36 m. As in borehole REP2101, pressure distribution depends on the distance from the shaft, and pressures range from 0.4 MPa in the chamber located at 1.1 m from the wall to 1.4 MPa for the furthest chamber from the shaft (at 2.25 m from the wall).

The evolution of pore pressure in borehole REP2103 is also recorded. The borehole is oriented rather in the direction of the minor horizontal stress. This borehole exhibits the same pore pressure evolution. Chambers which are oriented in the direction of the minor horizontal stress versus the shaft exhibit drop down of the pore pressure with the face advancing Chambers which are oriented in the direction of the major horizontal stress versus the shaft exhibit over pressure before the face reaches the level of the chambers and a decrease after.

Borehole REP2104 is vertical and located at 13 m from the shaft. Successive blasts induce an instantaneous decrease in the pore pressure. Between March to beginning of September, the decrease in pore pressure is near 0.5 MPa (figure 7).

4. ANALYSIS OF THE *IN SITU* DATA

It should be noticed that the REP experiment has generated many numerical modelling before and after its completion, especially in the framework of the European Modex-Rep project. This section does not present this sophisticated numerical modelling, but provide simplest analysis in order to check the consistency of the in situ data base. A 3D elastic calculation is used to investigate the displacement field as a function of the face advancing.

4.1 3D elastic modelling of the shaft sinking

Due to the anisotropy of the initial stress field, the first numerical computations are performed in 3D with the use of the three-dimensional finite difference code, *FLAC^{3D}*.
Calculations have been performed with horizontal stresses anisotropy of 1.3 (maximum of the anisotropy found in the layer).
In order to better represent all of the measurement points, a full geometry including the REP zone is modelled (top of model at – 414 m; and bottom of model at – 514 m). In addition, the lateral boundary of the geometrical model is located at 75 m from the shaft axis. This geometry includes the three main units (A, B&C) encountered in the argillites (Andra 2005a and b).

Both unit A and units B&C are modelled as linear-elastic, transversely isotropic (with horizontal plane of isotropy) whose properties, used for simulation, are derived from laboratory and in situ characterisations:
Unit A :
° $E_1=E_{//}=8620$ MPa; $E_3=E_\perp=9450$ MPa; $\nu_{12}=\nu_{23}=0.3$
Units B&C :
° $E_1=E_{//}=6960$ MPa; $E_3=E_\perp=8700$ MPa; $\nu_{12}=0.22$; $\nu_{23}=0.2$

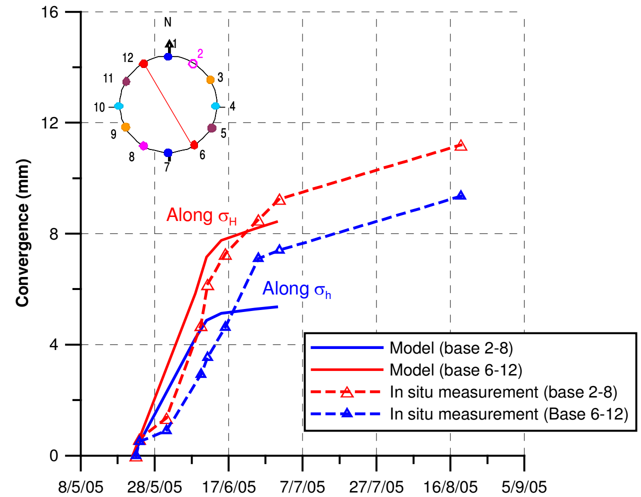

Figure 8. Comparison between measured and calculated convergences

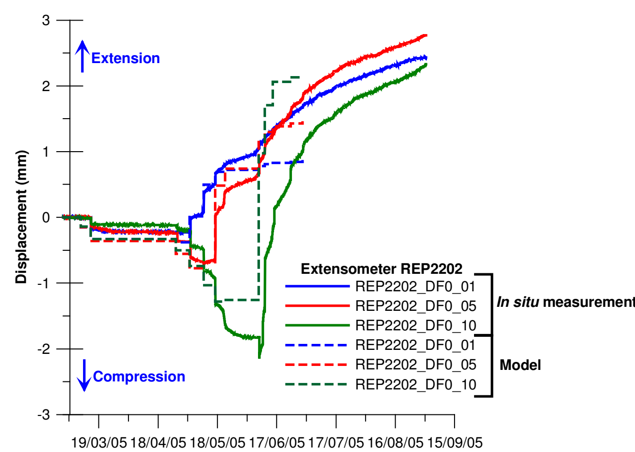

Figure 9. Measured and calculated displacements in borehole REP2202

For all units, the shear modulus G12 is evaluated based on the well-known Saint Venant's formula and valid for most of published experimental data (Worotnicki 1993).

The modelling sequence is performed as follows : *firstly*, the model without excavation was consolidated under in situ stresses depending on the depth (i.e. model loaded using a combination of gravity, in situ stresses and boundary stresses in order to produce the in situ stress field) and *secondly*, the excavation of shaft was carried out (roller boundaries were applied to the lateral sides of the model) according to the real advancing of the shaft sinking. That corresponds to 12 steps of excavation between -451.4 and -481.3 m level.

Figure 8 shows the comparison between the in situ measurements and the calculated convergence at depth -467 m. On base 6-12 in the direction of the major horizontal stress, a good agreement between the model and the in situ measurement is find until 30 of June (shaft face at -480.3 m). After this date, the increase of the calculated convergence is small with the face advancing. The measurement shows that the convergence continues to increase versus time even if there are just two steps of excavation in one month. The convergence exhibits long term behaviour due to creep and/or pore pressure diffusion. The elastic model couldn't catch this type of behaviour, but predicted well the short term convergence in the major horizontal stress direction. In the minor horizontal stress, at depth -480.3 m, the convergence (base 2-8) is under estimated with the elastic model showing that plastic deformation occurred in this direction.

Figure 9 presents the measured and calculated displacements of 3 anchors in the extensometer REP2202 and confirms that the 3D elastic model well predicts the deformation in the direction of the major horizontal stress. When the working face is before the anchors location, compression is seen in the extensometer and predicted by the numerical model. Every steps of excavation imply a jump in displacement. When the working face reaches the sensor depth, extension is measured and calculated. The displacement magnitude is well predicted until the end of June.

4.2 Poroelastic approach

4.2.1 Problem position

Consider an infinite cylindrical borehole excavated in a saturated porous rock subject to an anisotropic in situ stress field with isotropic component *P*0 and deviatoric component *S*0. The borehole boundary is free to drain and is exposed to atmospheric pressure. The initial pore pressure field is *p*0. This problem can be analysed by assuming plane strain conditions and instantaneous drilling of the borehole.

The two-dimensional poroelastic solution of this problem can be obtained by the transforms space of Laplace. In the general case, there is no analytical solution in real space: results in time domain can be derived using numerical inversion techniques. Nevertheless, for small values of time (short term response where the time is much smaller than the characteristic time) the general relations developed by Cheng and Detournay (1988) tend towards the short-time asymptotic solution.
This solution is formulated by superposition of asymptotic solutions for three fundamental loading modes : (1) a far field isotropic stress (classical Lamé solution) where rock strain is entirely associated with deviatoric strain, then without mechanism of pore pressure; (2) a virgin pore pressure distribution; and (3) a far field deviatoric stress.
For each of these three loading modes (denoting by superscript i), the short-time asymptotic solution for pore pressure generation is given by equation (1) where ν is the undrained Poisson's ratio; B_s : Skempton 's coefficient; erfc : complementary error function; r : radial distance; a : radius of borehole; c : fluid diffusivity; θ : angular position with respect to x-axis, and t : time.

For loading mode 1, the stress field and displacements are independent of the time and given by well-known Lamé solution, whereas the asymptotic solution for stresses and displacements (modes 2 and 3) are detailed in Detournay and Cheng (1988).

$$\begin{cases} p^{(1)} = 0 \\ p^{(2)} = p_0\left[1-\sqrt{\frac{a}{r}}erfc\left(\frac{r-a}{\sqrt{4ct}}\right)\right] \\ \qquad -\frac{p_0}{8}\sqrt{\frac{a}{r}}\left(1-\frac{a}{r}\right)\left\{\sqrt{\frac{4ct}{a^2\pi}}\exp\left(-\frac{(r-a)^2}{4ct}\right)\right. \\ \qquad \left.-\left(\frac{r}{a}-1\right)erfc\left(\frac{r-a}{\sqrt{4ct}}\right)\right\} \\ p^{(3)} = \frac{4}{3}S_0B_s\left(1+\nu\right)\left[-\sqrt{\frac{a}{r}}erfc\left(\frac{r-a}{\sqrt{4ct}}\right)+\frac{a^2}{r^2}\right]\cos 2\theta \end{cases} \tag{1}$$

The equations relative to these fundamental solutions (stress, pore pressure, displacements and derived strain) were programmed with the formal computation software *Mathematica*, for a quick and efficient sensitive analysis of poroelastic problems. Comparison with a numerical computation with the code *FLAC* allows us to validate our programming.

4.2.2 Application to REP experiment

Like all the chambers of pore pressure measurements (REP2101, REP2102, REP2103 and REP2104) are located between -462 and -470 m level (figure 2), we will consider a section at the average depth of -467 m, and corresponding to the section SMGR2. With respect to this section, we assume that the shaft is infinite and instantaneously excavated.
In consequence, these assumptions are identical to those of the 2D poroelastic problem previously described. So, the corresponding constitutive equations were thus used for a first critical analysis of pore pressure measurements.

The poromechanical properties used for reference case are based on a large laboratory characterisation (Andra 2005b) and tabled below (Table 1).

Table 1 – Poromechanical properties used

Undrained Young modulus, E [MPa]	5600
Undrained Poisson ration, ν [-]	0.3
Porosity, n [-]	0.15
Intrinsic permeability, k [m^2]	5×10^{-20}
Biot coefficient, b [-]	0.6
Biot modulus, M [MPa]	6542

The initial *in situ* stress (at –467 m) is : $\sigma_v=\sigma_h$=-11.3 MPa ; σ_H=-14.7 MPa (compressive stresses are negative) leading to : *P*0= $(\sigma_H + \sigma_h)/2$ and *S*0 = $(\sigma_H - \sigma_h)/2$ with σ_h oriented in the direction of x-axis.

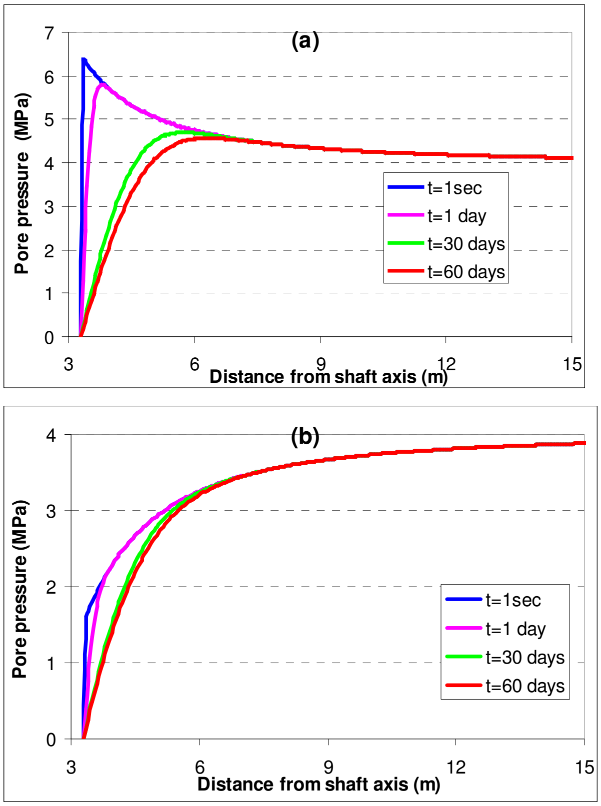

Figure 10. Evolution of pore pressure : (a) along σ_h-direction; (b) along σ_H-direction

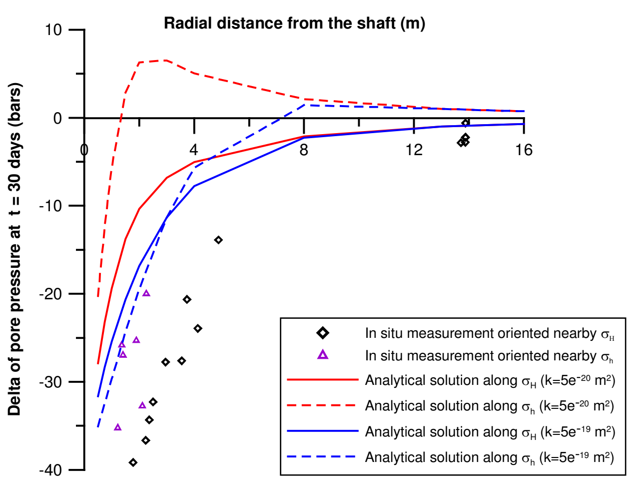

Figure 11. Evolution of pore pressure at 30 days after excavation

Figure 10 shows the distribution of pore pressure according to the radial distance in the directions of initial principal stresses for time ranged between 1 sec and 60 days. Just after the shaft drilling (t=1s) in the previously described anisotropic initial stress, we note the instantaneous response induces overpressures in the direction of the minor horizontal stress (θ = 0°). Indeed, the instantaneous solution is accompanied by high variations of mean stress (increase here). Except in the vicinity of shaft wall where null pore pressure is prescribed, the response of material is not drained and induces a peak of pore pressure located inside the rock mass. Location of this peak progresses toward the rock mass and decreases in magnitude with time. These overpressures dissipate in the time in relation to the fixed pore pressure applied at the shaft wall which will impose itself in front of the undrained disturbance.

The instantaneous response induces underpressures in the direction of the major horizontal stress (θ = 90°).in relation with the reduction in the induced mean stress in this direction.

This tendency to develop some overpressures in the direction of σ_h or some underpressures in the direction of σ_H was also observed on the site measurements at the passage of the face. More precisely, as in the previous analytical results, pressure sensors oriented with an angle ranged between -45° and +45° with respect to the σ_H-direction do not record some overpressures. Overpressures are observed in all the chambers with an angle less than 45° of the σ_h-direction. This character of development of overpressures or underpressures inside the rock mass, also highlighted by the poroelastic analysis, manifest the obviousness of the anisotropy of the in situ stresses. Table 2 summarizes the comparison between measurements and the results of poroelastic analysis of the induced overpressures and underpressures respectively near σ_h-direction and σ_H-direction.

The evolution of the measured pore pressure is therefore consistent with the poroelastic model, because the highest overpressures occur in the closest measurement chambers to the direction of the minor horizontal stress and the strongest underpressures occur in the closest measurement chambers to the major horizontal stress, thus confirming the sound operation of the experimental system.

Figure 11 presents the evolution of pore pressure at 30 days after the shaft excavation. Analytical solution has been used with two different permeability of the rock mass (5×10^{-20} m^2 and 5×10^{-19} m^2). 30 days after the shaft sinking if the permeability is equal to the lower one, overpressures have to be seen in the direction of the minor horizontal stress. If the permeability is higher, overpressures have been dissipated and pressures are nearly equal in all directions. The in situ data shows clearly that after 30 days pore pressures are higher in the direction of the minor stress proving rock mass permeability value measured *in situ*. However, after 30 days, the analytical solution exhibits higher pore pressure as a function of the distance to the shaft. The discrepancy between the analytical solution and the in situ measurement could be explained by different factors:

- the rock mass behaviour is not purely elastic around the shaft (see section 4.1)
- the permeability is affected by the sinking in the vicinity of the shaft due to the EDZ (excavated damage zone)

Table 2 – Induced overpressures and underpressures (MPa)

	Chamber- Distance to the shaft (m) - Angle to σ_H (°)	In situ measurement Δp1	Δp2	Analytical solution Δp1	Δp2
REP2101	1-4.85-21.8	-0.29	0.50	-0.31	0
	2-3.69-24.3	-0.46	0.66	-0.38	0
	3-2.68-26.6	-0.45	1.22	-0.44	0.02
	4-2.25-28.9	-0.52	1.60	-0.48	0.10
	5-1.71-31.7	-0.69	2.80	-0.51	0.36
REP2102	1-2.26-58.1	0.06	1.58	0.40	0.18
	2-1.89-63.8	0.31	2.47	0.62	0.40
	3-1.41-70.2	0.60	3.02	0.88	0.74
	4-1.36-77.4	0.88	1.52	1.13	1.15
	5-1.09-91.9	1.58	5.12	1.36	1.63
REP2103	1-2.11-55.5	0.24	2.42	0.36	0.33
	2-2.24-30.0	-1.55	1.69	-0.47	0.13
	3-2.5-16.8	-0.68	2.15	-0.65	0.03
	4-3.55-6.4	-0.53	1.06	-0.59	0
	5-4.13-1.9	-0.24	1.37	-0.52	0

with $\Delta p1=p(0^+) - p_0$ and $\Delta p2=p(0^+) - p(30d)$
p_0 the initial pore pressure
$p(0^+)$ the pore pressure just after the excavation,
p(30d) the pore pressure one month after the excavation

5. CONCLUSIONS

Among the 120 sensors installed for the experiment, 112 worked during the shaft-sinking, thus providing a important data base to understand to coupled hydromechanical behaviour of the claystone argillite of the Callovo-Oxfordien. Measured parameters (mainly displacements and pore pressures) are consistent and their evolutions confirm that:

- measured deformations are low. Argillites react almost elastically. Irreversible low-amplitude strains are only observed nearby the shaft wall in the direction of the minor horizontal stress in the REP zone;
- the evolution of pore pressure depends on the advance of the excavation work, the radial distance from the wall and the orientation of the measurement chamber in relation to the anisotropy of the *in-situ* stress field;
- the location of excavation-induced overpressures and underpressures is consistent with the stress concentrations generated by the sinking of a shaft and show the obviousness of the anisotropy of the initial stress field.

Simple analysis shows the consistency of the data set measured during the shaft sinking and emphasizes a strong hydromechanical coupling of the argillite rock behaviour (especially mechanical on hydraulic).

In addition to the data base provided by the REP experiment and the present simple data analysis, a numerical modelling program with more sophisticated models (taking into account HM coupling, plasticity, creep, …) and the real sequences of shaft excavation (in time and space) is undertaken within the framework of the European Modex-Rep project. Publications concerning these models and there predictions are under way.

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