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FULL 3D STRESS FIELD DIAGNOSIS AT THE HEART OF A LARGE GRAVITY DAM

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Abstract. *A large gravity dam, operated by Électricité de France in south of France, suffers from alkali aggregate reaction leading to structure swelling cracking. To assess the safety level and the maintenance cost of its dam, EDF needs to have a good understanding of the stress state at the heart of the dam to ensure a reliable diagnosis of the swelling phenomenon evolution.*

In 2009, the state of stress in the dam has been studied following two independent methods: numerical modeling of the structure and overcoring field measurements. Both methods were used not to complete each other but in order to confirm independently the relevancy of each diagnosis.

On the one hand, EDF has developed its own numerical model, based on the Finite Elements Method. The main purpose of this model is to quantify both swelling, damage and stresses in terms of anisotropy and amplitude.

On the other hand, INERIS performed a 6 weeks in situ stress measurement campaign using the overcoring method. Four areas inside the dam were investigated to obtain a comprehensive 3 dimensional stress tensors. The methodology was adapted to take into account the heterogeneity of the material at the scale of the measurement cell and the low temperature of the dam structure.

Comparison of the independent results showed an excellent matching of the overcoring measurements with the numerical predictions. The paper will describe both the overcoring campaign, the results obtained along with the numerical modeling and will discuss the relevance of these methods in the context of large and major dams.

1 INTRODUCTION

To evaluate the consequences of the natural ageing of dams, operators monitor closely the evolution of certain key parameters. In particular, significant strains and apparent disorders as cracks are the sign of unexpected stresses developing in the structure. In such cases, reliable stress determination becomes a must to ensure the diagnosis of the dam integrity and thus its safety. Stresses will indicate the level of degradation of the dam, according to known behavior of its material, and in the same time it is essential information to determine whether the integrity of the dam is at stake. However it still remains a technically challenging issue since the most interesting zones are often located at the heart of the structure or close to the rock-concrete interface.

This paper is based on a study of a gravity dam operated by Électricité de France (EDF, France). This dam was built in the early 30's to supply a downstream hydroelectric power plant and is still in operation nowadays. It suffers from a common reaction between the reactive siliceous phases of the aggregates and alkalis from the cement, the alkali aggregate reaction (AAR). The reaction creates a gel that absorbs water and precipitates in the cement paste porosity connected to the reactive aggregate. When the volume of gel reaches the volume of the available porosity, a swelling pressure leads to structures swelling and cracking. To assess the safety level and the maintenance cost of its dam, EDF needs to have a good understanding of the swelling phenomena and therefore of the stress state at the heart of the dam.

In 2009, the stress state of the EDF dam has been studied following two independent ways: numerical modeling of the structure and overcoring field measurements. These methods were performed independently to confirm the relevancy of each diagnosis. The paper will describe both the numerical modeling led by EDF using the finite elements modeling (FEM) and the *in situ* overcoring measurement campaign performed by the Institute of Industrial Environments and Risks (INERIS, France). The relevance of these methods will be discussed in the context of large and major dams.

2 FINITTE ELEMENT MODELING

AAR and its effects were modeled using a phenomenological approach taking into account the different phenomena summarized in Figure 1. The main developments made in this model concern:

a) The interactions between AAR pressure and long-term strain (creep);

b) The swelling anisotropy induced by oriented cracking.

Particular attention is also paid to modeling moisture effects on both AAR and long-term strain. The AAR swelling dependence on the stress state is then a consequence of all these elementary phenomena. Hence, the mechanical effects of AAR are the consequences of a long-term internal loading due to chemical pressure (Pg in **Figure 2**). In addition to external loading (σi in Figure 2), Pg loads concrete, which is considered as a visco-elasto-plastic-damaged medium (module VEPD in Figure 2). The following paragraphs explain how the gel pressure is computed according to the environmental conditions and the strain state. Then a comparison between in-situ stress tests and finite element modeling is performed.

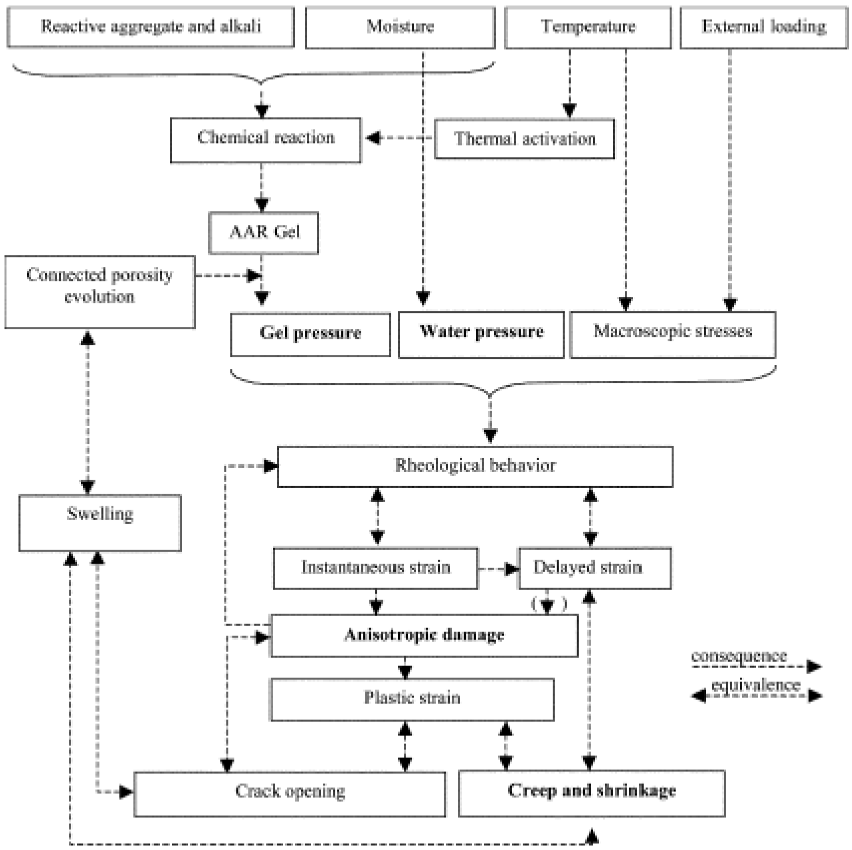


Figure 1 : Main phenomena taken into account in proposed modeling

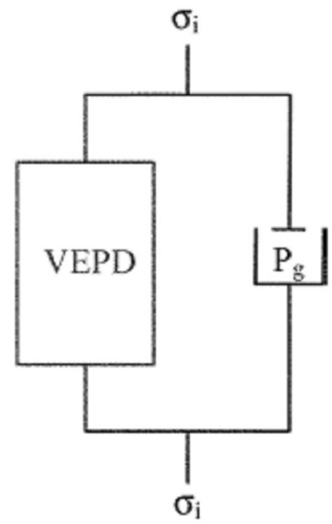


Figure 2 : Idealized view of expansive concrete behavior model (Pg is gel pressure and VEPD is visc-elastoplastic and damage concrete behavior law symbol)

2.1 Methodology

One of the main questions to address was the estimation of the residual expansion of concrete. Initially, classical residual expansion tests [1][2][4] have been carried out at according to the LPC method [12]. This method consists in measuring the longitudinal expansion and the mass variation of core samples (14 cm diameter, 30 cm length) drilled from the dam and kept into a controlled environment (38°C, relative humidity > 95%).

But the results of these tests are not realistic for a dam. So Electricité de France in collaboration with the Laboratoire Matériaux et Durabilité des Constructions (Université de Toulouse) proposed a global methodology in order to determine the AAR kinetic independently of the gel nature (Figure 3). The approach is based on the assessment of the reactive silica consumption [8]. The reactive silica consumption kinetic is determined for each aggregate size range. The amplitude of final swelling is not measured from laboratory expansion tests, but assessed from a FE inverse analysis of the affected structure. The FE modeling used, detailed in [5] and [6], is summarized below. It combines the advancement kinetic deduced from laboratory tests and the final AAR swelling which is the only parameter to be fitted with the structural FE inverse analysis. Once the final swelling has been obtained by curve fitting, the numerical modeling is tested in order to compare other expansion measurements carried out on the dam and not used for the determination of the fitted parameter. If results are right, computation can be carried out to predict the future structural behavior. This global methodology is summarized in Figure 3.

In the following sections, the constitutive equations of the FE model are briefly summarized in order to present the main modeling assumptions.

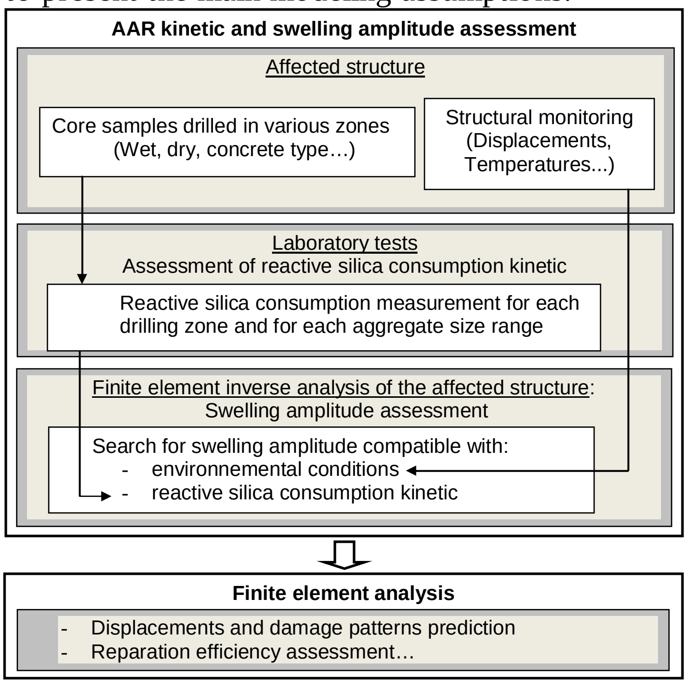


Figure 3 : Summary of global methodology

2.2 Summary of the finite element (FE) modeling

The law describing the mechanical behavior of affected concrete assumes that AAR acts on concrete through a gel pressure Pg which is combined with the water pressure Pw (Figure 5). As pressure exists into the concrete porosity, the mechanical model is based, as in [18], on a poro-mechanical formulation described in details by Grimal et al. [5], [6], [20] and summarized by Eq. 1.

$$\boldsymbol{\sigma} = \boldsymbol{C}(\boldsymbol{\varepsilon} - \boldsymbol{\varepsilon}^{an}) - \boldsymbol{b}_g \boldsymbol{P}_g - \boldsymbol{b}_w \boldsymbol{P}_w \qquad (1)$$

In concrete damaged by AAR, the total strain ε is induced by the AAR-gel pressure P_g acting into the concrete porosity, by the mechanical stress σ due to the structural loading and by the capillarity pressure P_w. As explained in [5] and [6], Pw represents the shrinkage mechanism. The anelastic strain ε^{an} includes both the creep strain and an irreversible strain associated to the cracks opening [5]. In Eq. 1, C is the damaged stiffness tensor [17]. bg and bw are parameters giving the pressures influence on the concrete matrix [18].

According to the above observations and to previous works [3][15], it is assumed that large aggregates present a lower kinetic than small ones. Indeed, if hydroxyls alkali and calcium ions diffusion coefficients are nearly the same for large and small aggregates, the chemical advancement of the consumption of reactive silica, defined as the ratio of the affected zone on the sound zones of the aggregate, depends on the aggregate radius (Figure 4). That is the reason why the size distribution of aggregate in the concrete must be discretized in several sizes (superscripts "s" in Eq 2), and a summation on $s = \{1,..,N\}$ must be done, N being the size ranges number. n^s is the number of aggregates size s for a given size range. Thus, the gel pressure (Pg) is linked to the AAR chemical advancements As following equation 2, taken from [5].

$$\boldsymbol{P_g = M_g \left\langle \sum_{s=1}^{N} \left\langle n^s A^s f\, V_a^s - V_p^s \right\rangle - \left\langle b_g tr(\varepsilon) \right\rangle \right\rangle} \tag{2}$$

V_a^s is the volume of one aggregate of size 's' and $f\,V_a^s$ is the maximal volume of gel created by the aggregate. A^s is the advancement of AAR reaction for a given aggregate size 's'; it is defined as the fraction of the volume of AAR-gel produced at a given time by the maximal volume which can be produced by the aggregate. It evolves from 0 for the aggregate which has not been still attacked by the reaction to 1 when the reactive silica of the aggregate has been totally attacked. It represents also the fraction of reactive silica consumed by the reaction. Therefore $n^s A^s f\, V_a^s$ represents the volume of AAR-gel created by ns aggregates at a given time.

V_p^s is the available porosity connected to the aggregate "s" (cf. Eq. 6). bgtr(ε) is the additional connected porosity due to the concrete strain and it includes the AAR cracks through the anelastic strain (ε_{an}). The positive part symbol $\langle x \rangle = (x \text{ if } (x > 0), 0 \text{ otherwise})$ points out that the pressure Pg appears when these two porosities are filled by the gel. The coefficient Mg in Eq 2 is the bulk coefficient of the gel. The fitting of bg and Mg, given by Grimal and al [6], requires free and constrained swelling tests to be carried out.

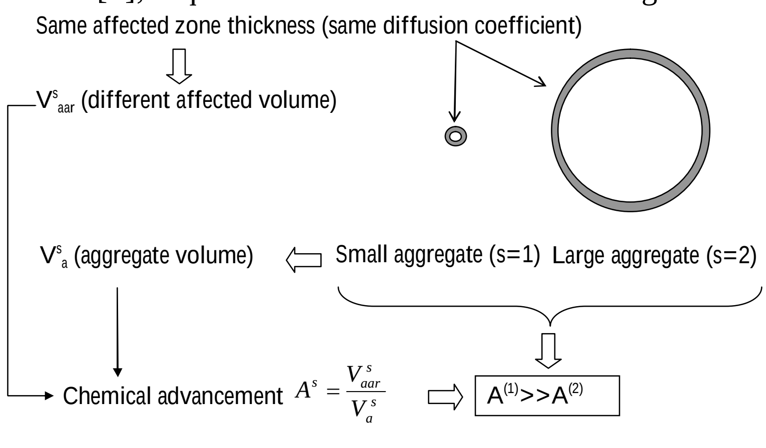


Figure 4: Difference of chemical advancement between a small (1) and a large (2) aggregate s = aggregate size index

In the FE modeling the chemical advancements A^s are computed for each aggregate size chosen to describe the aggregate size distribution. For this, a numerical step by step integration of the differential evolution equation (3) is used. This equation is inspired from [16] and [5]. It takes into account the "in situ" environmental conditions (temperature θ and liquid water saturation degree in the pores Sr).

$$\frac{\partial A^s}{\partial t} = \underbrace{\alpha_{20}{}^{s}.\exp\left(-\frac{E_a}{R}\left(\frac{1}{273+\theta}-\frac{1}{293}\right)\right)}_{\alpha_\theta^s}\frac{\langle Sr - Sr^0\rangle}{(1-Sr^0)}\langle Sr - A^s\rangle \tag{3}$$

With α_{20}^s the kinetic constant to be fitted, Sr the liquid water saturation degree of the porosity and Sr^0 the saturation degree threshold upper which the reaction occurs (estimated to 40%, according to Poyet's experimental results [16]). θ is the temperature in °C, Ea the activation energy (usually about 47000 J/M [24.748 Btu/M] [5]), and R the gas constant (8.31 J (4.37e-3 Btu) /M/°K). This differential formulation of the AAR advancement allows environmental conditions variations to be taken into account. It is obtained from the non linear equation of mass transfer detailed in [5][6][7] and numerically solved with the FE method and boundary conditions imposed from the measurements of in situ saturation degree. The measurements of the saturation degree have been performed on samples taken from the dam by dry concrete sawing. Just after the sawing, the samples were sealed in watertight packaging and the measurements of saturation degree were carried out as soon as possible. Hence, the AAR relevant parameters to be fitted were: the kinetic constants α^s (one per aggregate size range, $s=\{1,..,N\}$) and the AAR-gel rate f. The presentation of the kinetic constants assessment from laboratory tests of the reactive silica consumption and the adjustment of parameter f is not presented here but, a metholdology is explained in [9].

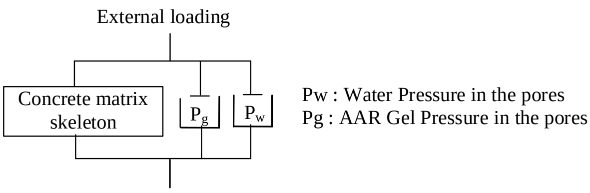


Figure 5: One dimensional idealized view of expansive concrete behaviour model

2.3 Finite element inverse analysis to assess the swelling amplitude

The fitting achievement consists in the determination of the constant f (Eq 2). The constant f obtained from the accelerated tests cannot be used for the dam's analysis because of the difference of ASR-gel nature between the long term reaction in the dam (calcium-silica gel) and the short term reaction in the accelerated tests (alkali-silica gel). In order to assess this constant, the model is fitted on the observed behaviour of the dam. The constant is then iteratively assessed to adjust the FE model response (in terms of structural displacements) to one measurement performed on the dam. In the calculations, the final swelling of concrete containing several ranges of aggregate sizes is assumed to be the sum of the final swelling contributions of each range of aggregate size (summation on N in Eq 2). The FE model includes also the structural effects of the mechanical boundary conditions on the swelling [10][11][13][14][19]. In Eq. 3, kinetics constants α_{20}^s are assessed according to the stress free expansion of the mortar (Table 1) and the environmental conditions of the dam (Eq. 11). All the other mechanical parameters needed to use the FE model were measured on drilled samples. Thus measurements of compressive and splitting tensile strengths, Young's modulus and creep strains were carried out during the experimental study of the dam.

The variation of the vertical displacement measured on a point of the dam is used to fit the parameter f (**Figure 6** (a) vertical direction). It is the only measurement on the dam which is used for the parameter fitting. Then, the lateral, and the upstream-downstream displacement of the same point (**Figure 6** (a)) can be simulated with a good accuracy.

The lateral and the downstream-upstream movement of the other part of the dam are also explained by the model (**Figure 6** (b)). Then a prediction of the dam movements can be done for the next decades (**Figure 6** (a) and (b)).

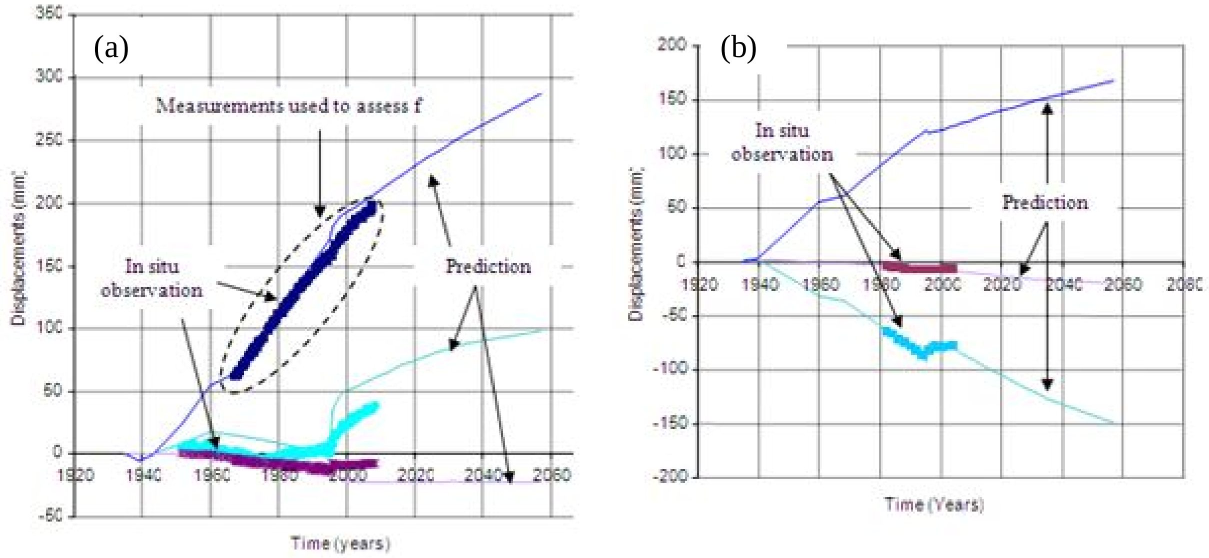


Figure 6 : (a) Fitting of the swelling amplitude on vertical structural displacements

The FE structural modeling proposed in [5][6] is also able to compute the damage field into the dam, due to the AAR evolution. This damage evolution is not presented in this paper.

To study the behavior of a gravity dam, the state of stress in the heart of the dam must be known. To verify the concordance between modeling and reel state of stress, a comparison is made. The reel state of stress is obtained by an original tests campaign.

In the following section, the in-situ test methodology is explained in first and afterward the comparison between FEM analysis and in situ test is performed.

3 *IN SITU* MEASUREMENTS CAMPAIGN

Nevertheless some of them are limited to a superficial measurement (e.g. flat jack) or to a 2D stress determination (e.g. USBM cell overcoring). Overcoring of 3D strains cells, as CSIRO cell or BORRE cell, is one of the most commonly used method as it enables the determination of a comprehensive three dimensional stress tensor in one single point. The basic principle of the method is summarized here after. The key steps are detailed in the following sections.

This method was put in practice in the large gravity dam of EDF, during a measurement campaign in winter 2009-2010. It had already been successfully tested at the very same dam by INERIS in 1991 using USBM and CSIRO cells. CSIRO cell had proved to be the more relevant in the context of the dam. Four measurement locations were planned, two on each side of the dam and with at least two successful tests to be performed at each location. The tests were run in the concrete, near the rock foundation, in average 20 meters far from the dam wall. In a single location, successive tests were performed about one meter from each other to obtain redundant data, representative of the local state of stress. Application of each step of the method to the case review of the EDF dam is also described in the following sections. A picture of the drilling engine is given in Figure 7.

Figure 7 : Drill on site. The tank for heated water is seen on the right (cf. section 3.2).

3.1 Basic principle

The overcoring method consists in measuring the strains that develop on the wall of a small diameter borehole (pilot hole) when this one is relieved from the surrounding *in situ* stress field by overcoring, as shown in Figure 8. Assuming a rock constitutive law (usually linear elasticity), the *in situ* stresses may be determined by inversion of the measured strains. This inversion requires the values of the material elastic parameters which are usually determined from biaxial testing on the retrieved sample.

The biaxial test consists in loading the overcored cylinder with a known radial stress. The comparison between applied stress and measured strains will indicate the material elastic parameters. During the biaxial test, strain curves normally show circumferential contraction and axial elongation as the overcored sample is laterally loaded. At the end of the unloading phase, the strain readings fall back to zero if the rock is perfectly elastic, or keep a slight circumferential contraction if the rock experiences permanent deformation.

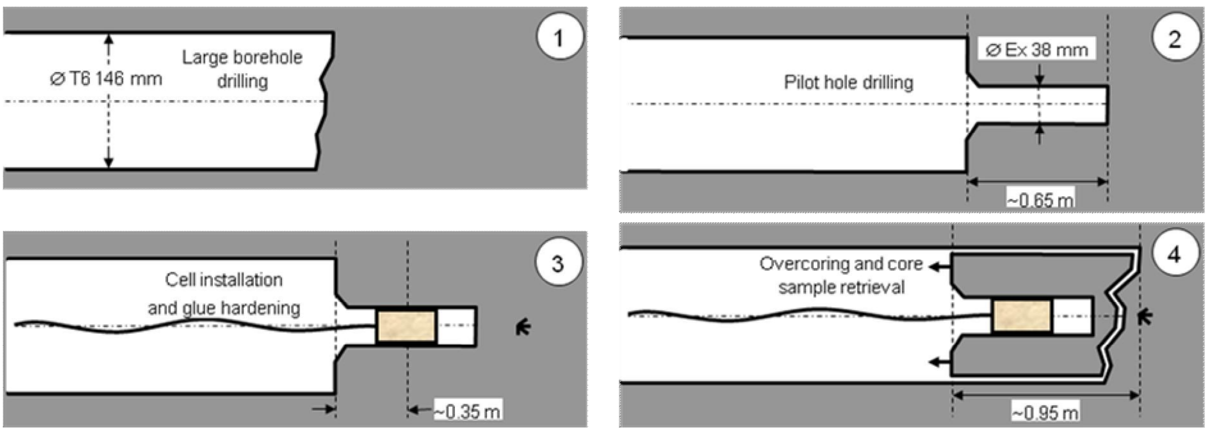


Figure 8 : Overcoring principal steps.

3.2 Strain cell installation

The principle of the overcoring method is based on strains measurement in the material (rock, concrete, etc.) when it is relieved from the surrounding stress. The point of measurement in the rock mass or in the structure is reached by a straight 146 mm diameter borehole. The drilling might be done with water or air, depending on the material, as long as it does not change its mechanical properties. This large borehole should be stopped a few dozen centimeters before the considered point of measurement and continued with a 38 mm diameter pilot hole. This pilot hole is then equipped with a strain measurement cell. INERIS uses the CSIRO Hi12 cell: this cell is a soft hollow inclusion of diameter 36 mm in the wall of which are embedded 12 strain gauges oriented 0°, 45°/135° and 90° from the borehole axis [21]. It is also equipped with a temperature gauge near the position of the strains gauges. The cell should be installed deep enough in the pilot hole to avoid any influence of the large borehole on the *in situ* stresses.

The cell is glued to the rock walls using an epoxy resin formulated according to the host rock temperature. The complete hardening of the glue is of uttermost importance for the success of the overcoring test as the glue has to faithfully transmit the upcoming strains of the core sample to the gauges of the CSIRO cell. Incomplete hardening of the glue will result in incoherent measurement strains. Two factors will have to be taken into account to achieve satisfying hardening: the curing time and temperature. Epoxy glue takes several hours to reach total hardening within a given range of temperature. A few Celsius degrees below that range curing time might either increase in a drastic way or merely never be reached. This aspect is particularly important in structures as dams that can be significantly colder than rock mass. If relevant, artificial ways of increasing the temperature during the curing time should be considered.

Finally fractured zones should be avoided since they are unfavorable to a satisfying coupling between the gauges and the pilot hole. Moreover fractures can create singularities in the stress which are not representative of the local stress at the considered location. Such zones might be spotted through the analysis of the core sample from the pilot hole. However its small diameter makes it often difficult to get it in one or few parts.

In the case of the EDF dam, the concrete temperature approaches 9 °C. It was known from the previous tests that 9°C was not sufficient to ensure a good hardening within a reasonable curing time (i.e. inferior to 24 hours) even with 4-10°C epoxy. Two systems were put in place to increase temperature during curing time. As the boreholes were drilled using water, which causes no particular problem in concrete, the drilling water was heated up to 50°C. After all drilling operations, heated water was injected for about 2 hours in order to warm the concrete mass around the pilot hole. This operation was led as long as possible since the concrete would cool down quickly due to the rather low temperature of the dam. Then heating resistors were adapted at the end of the rods system used to set the CSIRO cell in place (cf. Figure 9). Even with the help of these resistors a significant loss of temperature after the heated water was stopped could not be avoided. However the resistors prevented the temperature from decreasing too much. Such resistors should obviously not be used in a down borehole as water would eventually stay in or infiltrate into the borehole.

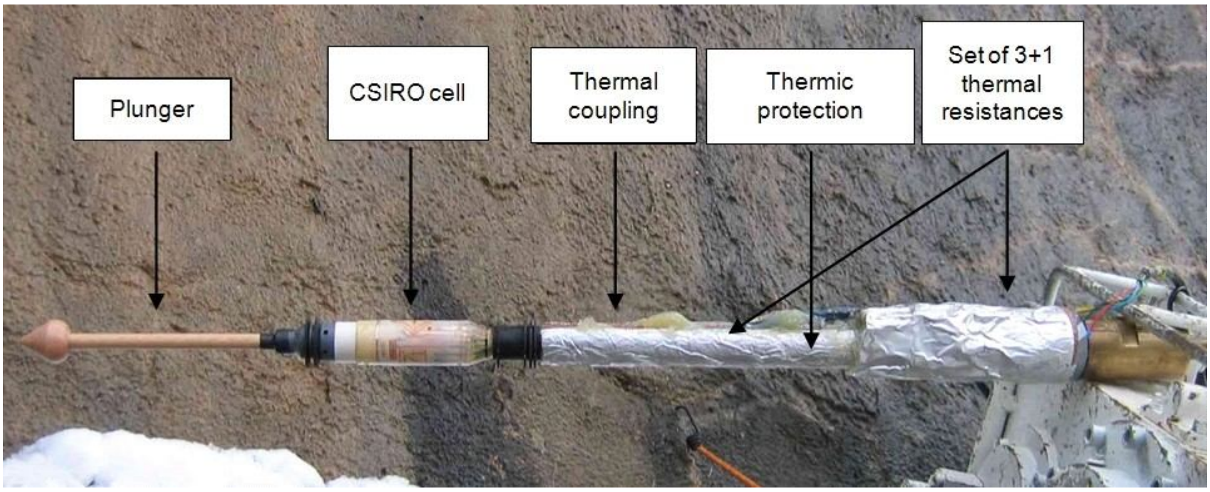


Figure 9 : CSIRO cell installation device.

During curing time the strains should already be recorded and monitored as well as the temperature near the strain gauges. Actually the reach of plateau values for the strains is a necessary but not systematically sufficient condition for the hardening to be considered as complete. Figure 10 shows typical strain curves during curing. Temperature starts from 21°C, tends to decrease after the stop of the heated water circulation, but eventually increases in the second half of the curing time. The average temperature during curing is approximately 18°C, which is way more favorable to a satisfying hardening of the glue than the original 9°C of the dam. At this temperature, curing typically lasts about 20 to 22 hours. Considering this and the time needed for drilling and acquisition operations, two tests can be performed over three working days.

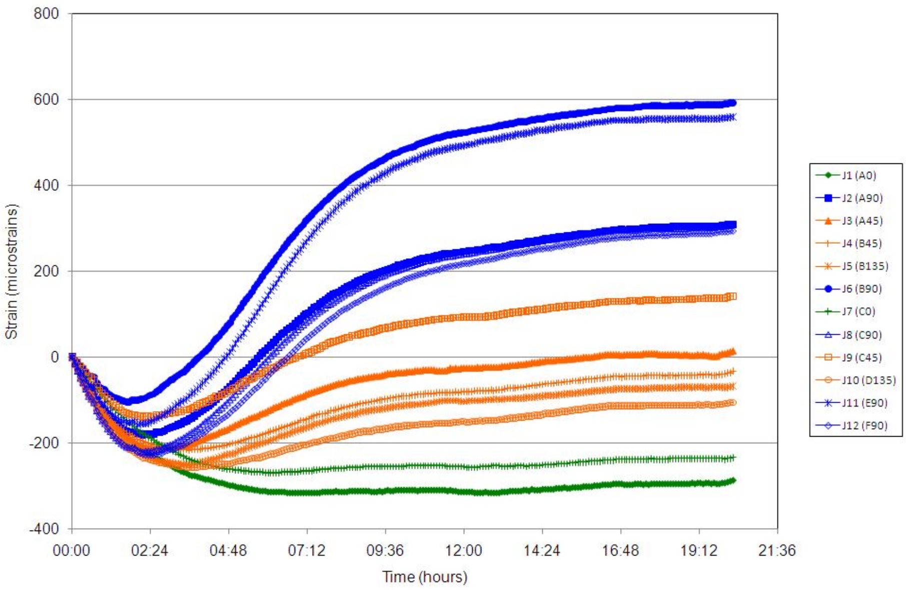


Figure 10 : Typical strains curves during curing time.

3.3 Overcoring

Once the strain cell has been installed and the epoxy glue successfully hardened, overcoring operations can start. The large diameter borehole is continued over the pilot hole equipped with the CSIRO cell in order to completely relieve the core sample from the stresses of the rock mass or the dam. The core sample will finally be retrieved from the borehole for further testing.

During the overcoring operation special care must be taken to the stability and the speed of drilling. In slightly fractured areas, unstable or vibrating drilling might lead to the opening of preexistent fractures or to the disking of the core sample. In such case, the gauges are no longer correctly coupled with the core sample, the strains cannot be trustfully measured and the test has to be considered as failed.

Figure 11 shows the typical evolution of strains on the wall of the pilot hole of an overcoring test. The recorded strains usually show stable null readings at the start of overcoring. As it passes the strain gauge position, which is basically located at the center of the CSIRO cell, the drilling bit creates a temporary radial stress that causes a local minimum on the orthoradial gauges and a maximum on the axial gauges. Once the drilling bit has gone past the gauges, the strains curves reach plateau values which are generally used as an input for stress determination. Clear plateau values can be considered as the proof of a good coupling between the gauges and the core sample. Mediocre coupling would result in various anomalies as a strong drift or to a decrease of the plateau values for example.

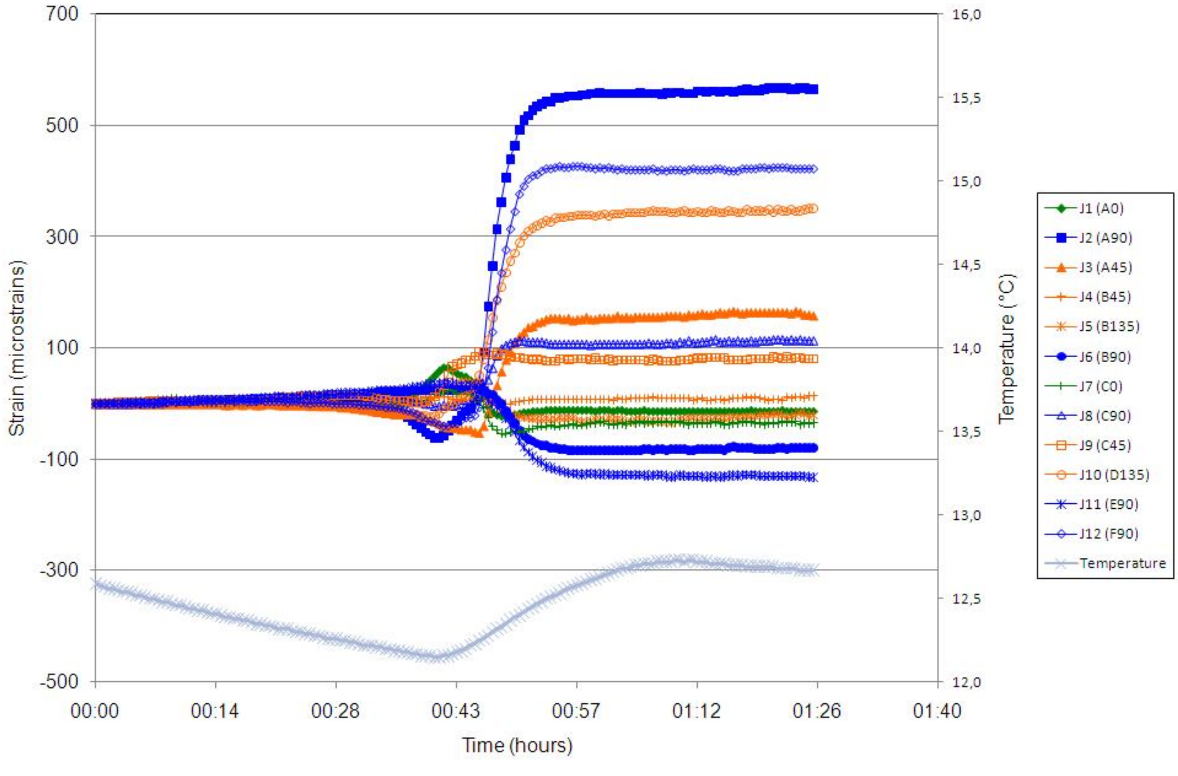


Figure 11 : Example of strains curves after an overcoring test.

Temperature had an important influence on the concrete of the EDF dam: tests showed that a few degrees lower temperature on a core sample equipped with a CSIRO cell provoked significant decreases of strains values. Thus the temperatures at the beginning and at the end of the overcoring should be as close as possible. Heated water was used once again for drilling operations with a real-time close monitoring of the influence of heating on the cell temperature itself. Less than 0,8°C were observed between start and end temperatures over the performed tests, which was considered as much satisfying.

A multi-parameters acquisition unit is necessary to monitor in real-time the strains of the CSIRO cell gauges, the temperature of the CSIRO cell, the temperature of the heated drilling water and the precise position of the drilling bit toward the gauges. A rugged system is also a must if the tests are performed outdoor. In particular environment temperature might have an influence on the measurement (trough cables mostly). This aspect should be checked before any overcoring test. INERIS uses a specifically designed receiver dedicated to strains measurement with CSIRO cells, part of the high rate geotechnical acquisition system SYTGEO®. All data are concentrated to one software interface, SYTGEOscop, which allows to monitor simultaneously all the above mentioned parameters. In particular, drilling speed and pressure might be adapted to maintain a slow and stable drilling advancement speed.

3.4 Biaxial testing

The determination of stress from strains measurement requires the knowledge of the elastic parameters of the material. For an isotropic material only the Young's modulus E and the Poisson's ratio ν are needed. These parameters can be easily determined by simple tests in laboratory. However even if average values can be measured with great reliability, tests on nearby core samples often present a high dispersion. To benefit from the greatest accuracy, it is recommended to determine the local values of E and ν at the very location of the overcoring test. Therefore, a biaxial test is performed on the overcored sample, which is still equipped with the CSIRO cell.

The test consists in applying a radial isotropic pressure through a Hoek cell to the core sample. Strains are monitored in a way similar to the overcoring operation. Due to the radial pressure, orthoradial gauges will undergo compressive strains, giving the Young's modulus, while axial gauges will expand, as seen in Figure 12. The relation between axial strains and nearby orthoradial strains will indicate the Poisson's ratio. To ensure the best representativeness of the results, the applied pressure should be similar to the expected state of stress and calculus should be made considering unloading cycle rather than loading cycle.

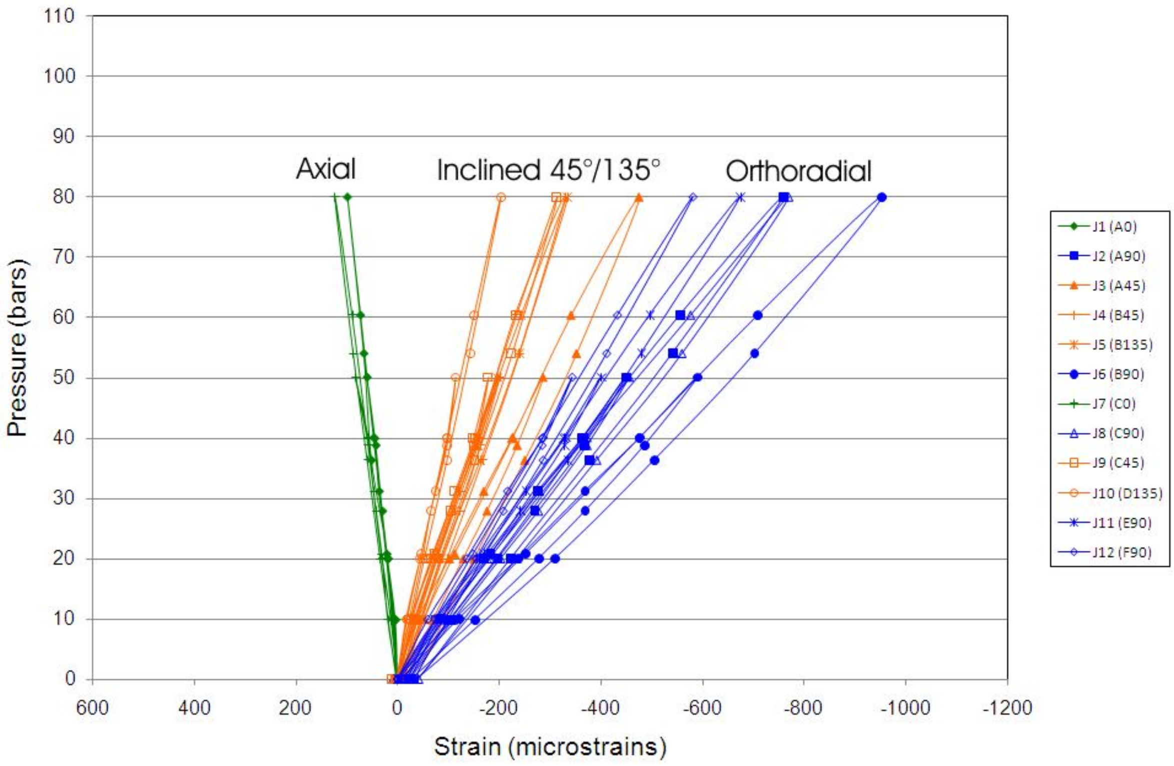


Figure 12 : Example of biaxial test strains curves. The response of the orthoradial gauges shows a significant heterogeneity.

Furthermore, a biaxial test can be considered as an ultimate way to validate the success of an overcoring test. After a loading cycle, the strain should ideally go back to zero or maintain a little plastic deformation. However an extension of the orthoradial gauges after the unloading cycle means a mediocre hardening of the epoxy glue, which could have not been identified through the curing strain curves.

Finally the biaxial test proved to be even more valuable in the context of the EDF dam. The concrete being melted with gyps, it cannot be strictly considered homogeneous at the scale of the CSIRO cell, one gauge possibly in contact with gyps, another with concrete. This heterogeneity was revealed through the different responses of the orthoradial gauges toward the radial pressure (cf. Figure 12). Variations in response up to 30% compared to the mean strain were observed, indicating a rather significant heterogeneity which had to be taken into account in the inversion process

3.5 Stress determination

The typical back-analysis relies on three major assumptions:

- The overcored rock is considered to be a homogeneous linearly elastic material. As said above, in the case of the EDF dam this assumption was not totally verified, due to the presence of gyps in the concrete. However the heterogeneity caused by the gyps was taken into account directly in the strain data, as it will be described below, the linear relation between strains and stress was preserved ;
- The pilot hole is considered to be an infinite circular hole. This is a reasonable assumption as long as the CSIRO cell is installed sufficiently far from the pilot hole collar and bottom ;

- The coupling between the cell and the surrounding material is considered to be perfect. This emphasizes the need for a reliable hardening of the epoxy glue.

Using these assumptions, elasticity theory enables to calculate the strains ε_i^* expected at the location of each gauge after the complete relief of the rock (plateau values) as a function of the far-field stress tensor $[\sigma_0]$. Doing this for all gauges leads to a system of linear equations relating the final strain ε_i^* expected on each gauge to the six independent components of the far-field stress tensor. The problem of determining the stress tensor $[\sigma^0]$ thus comes down to inverting the following system (4):

$$[\varepsilon]^{n*1}=[A]^{n*6}.[\sigma^0]^{6*1} \tag{4}$$

where ε is the set of actual strains measurements, A is the influence matrix taking into account the geometry as well as the elastic parameters of both the material and the epoxy glue [22] and n is the number of measurements. For a single measurement location, data from successive overcoring tests can be used, as the tests were run close to each other (approximately 1 meter). This allows merging the information from multiple tests in one single inversion, the final result not being a mere average of individual inversions.

INERIS uses the SYTGEOstress® software to solve the inversion problem using the least-square method. Each inversion results in the best estimation of the complete *in situ* stress $[\sigma^0]$ resolved in a chosen Cartesian coordinate system, as well as the major (σ_1), intermediate (σ_2) and minor (σ_3) principal stress values and orientations. The quality of the results can be evaluated by using the inverted stress tensor $[\sigma^0]$ to retro-calculate the strains ε_i^* and compare them to the actual measurement. A simple way to achieve this comparison is to concentrate the information in a single indicator such as the Coefficient of Variation, CoV (5) [23]:

$$CoV = \sqrt{\frac{\sum_i(\varepsilon_i-\varepsilon_i^*)^2}{\sum_i(\varepsilon_i^*)^2}} \tag{5}$$

This indicator can be expressed as a percentage and its value should be appreciated according to the number of measurements taken into account. For a 12 gauges system, a small value (inferior to 10%) will be the sign of both good measurements and reliable hypothesis of inversion. Higher values (between 10% and 20%) would still be acceptable, while CoV superior to 20% indicate a mediocre or failed overcoring data set. As the system is over-determined, due to the high number of gauges on the CSIRO cell, problematic data might be disregarded. Clearly faulty gauges will be suppressed from the data set before any inversion. However the use of certain gauges, apparently valid, might result in a significant increase of the CoV, thus degrading the quality of the result. Their removal should be considered carefully to ensure the best inversion quality while maintaining a decent amount of data in the system (typically at least 9 gauges out of 12 should kept).

Finally, as seen in the biaxial test section, the EDF dam concrete could not be considered as homogeneous at the scale of the CSIRO cell due to the gyps. This heterogeneity was taken into account by balancing the strain values with the ratio of the Young’s modulus calculated with the nearest orthoradial gauge over the average Young’s modulus [24].

3.6 *In situ* measurement campaign results

Despite the coldness of the concrete and weather during the operations, curing time was limited to less than 22 hours due to the heated water system and to the heating resistors set up in the pilot hole. Plateau values were systematically reached. Good control over the drilling water temperature was achieved in order to keep a constant temperature of the CSIRO cell. Great attention was also paid to the drilling speed in order not to damage the core samples. Only one important fracture was encountered, most of the core samples were obtained intact. Therefore biaxial test could be performed on most of them, resulting in a great accuracy for elastic parameters determination.

As previously stated, biaxial tests showed a rather significant heterogeneity in the elastic parameters of the concrete of the EDF dam. The strains values were balanced according to the local Young’s modulus. It proved to improve the accuracy, i.e. decreasing the Coefficient of Variation, for several overcoring tests. An example is shown in Fig 13.

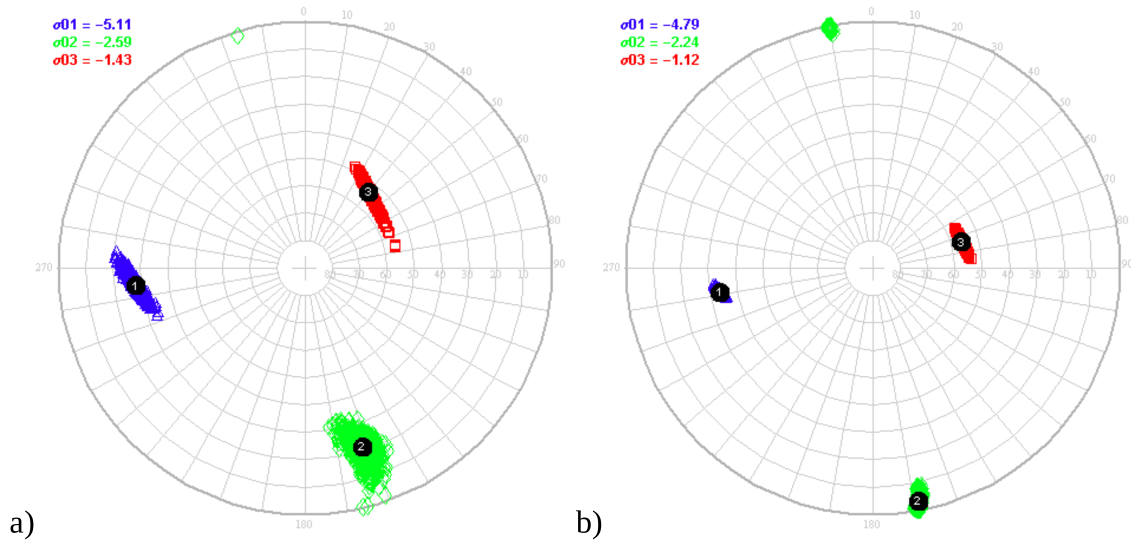


Figure 13 : Example of representations of principal stresses in the EDF dam, a) with uniform elastic parameters, b) with heterogeneity taken into account.

Four test locations were planned with at least two successful tests in each location. Actually even if the validity of a test can be estimated from the overcoring curves, from the biaxial test and from the inversion results, at least two tests with similar results are needed to reliably validate the determined stress tensor. For each measurement location, the successive test showed great redundancy, in terms of stress values and orientations. Therefore only two tests had to be run in three of the four locations. In the last measurement location, one test was failed due to a bad coupling between the CSIRO cell and the concrete, resulting in only nine tests for four measurement locations with the above described protocol.

Few gauges were disregarded, with at least 10 gauges for each test. The accuracy of the plateau values resulted in CoV typically inferior to 10%. To definitely validate the individual results combined inversions were run for each location. The results were systematically consistent with the individual inversions and were kept as the final stress tensor determination.

In a general manner the major principal stress was found to be parallel to the dam axis and in most cases perpendicular to the above rock foundation. The minor stress was found perpendicular to the dam axis in three locations. All principal stresses are inferior to 6

MPa, with a global average near 2,1 MPa. This rather low stress state compared to stress states usually found in rock mass in mining or underground laboratories proved the usefulness of the overcoring method even in surface structure with low stress states.

All measurements were led on purpose with no preconceived idea of the actual *in situ* stress state. Comparison of the *in situ* data by INERIS was then made by EDF with the prediction results from the numerical modeling. The correlation between the two methods proved to be excellent and is described in section 4.

4 COMPARISON BETWEEN FEM ANALISYS AND IN SITU MEASUREMENTS

In order to perform relevant calculations, a comparison between overcoring campaign and modeling is performed.

Three characteristics zones are chosen for the comparison. The first and the second zones are near the contact concrete-foundation, in right river and in the central part and the third at the heat of the upper left river (Figure 14 and Figure 15). The comparison between modeling and overcoring test results are given Table 1.

The Figure 14 and Figure 15 show the main compressive stress and the main tensile stress on the downstream face of the dam.

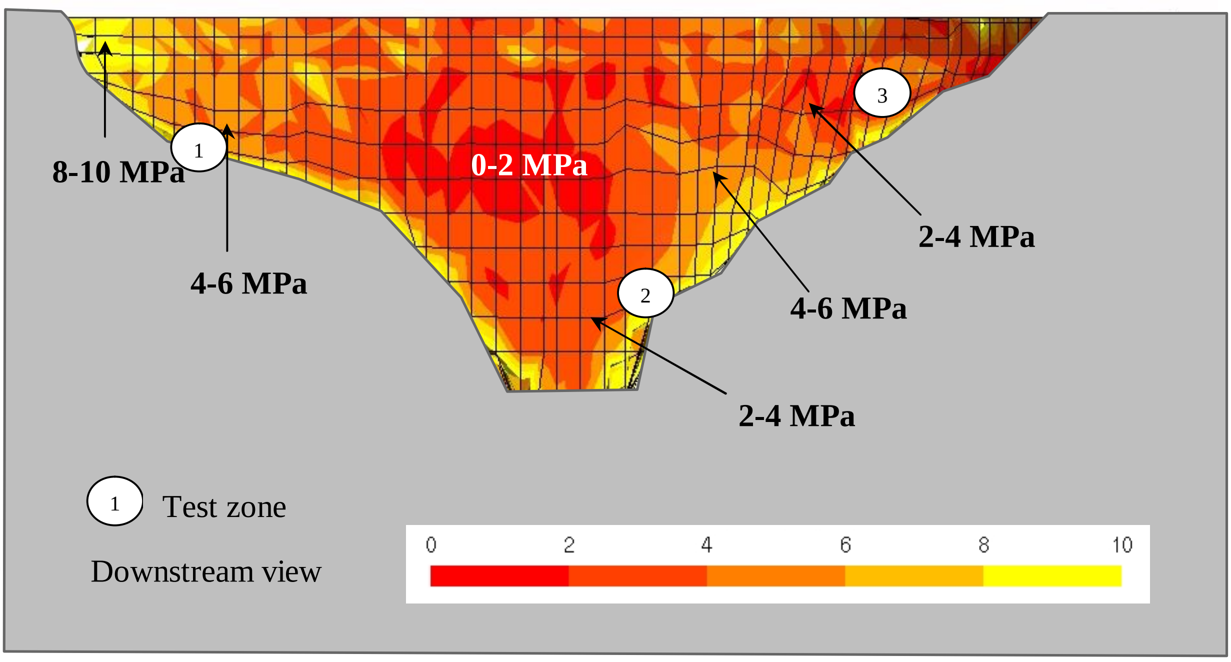


Figure 14 : Main compressive stresses on the downstream face

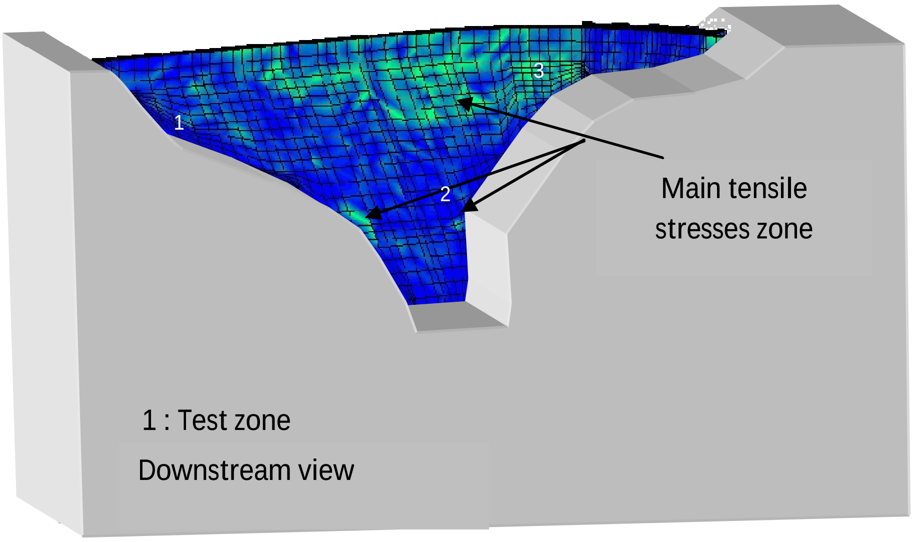
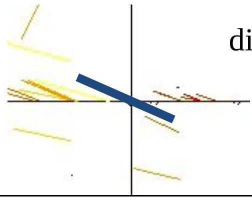
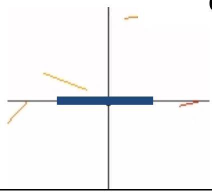
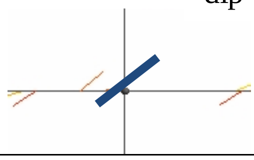


Figure 15 : Main tensile stresses on the downstream face

Table 1 : Comparison between modeling and overcoring test

	Modeling	Test
Zone 1 (Positive stress are compressive)	$\sigma 1$ = 5 MPa dip : 26 ° 	$\sigma 1$ = 5.5 MPa dip : 37 °
Zone 2 (Positive stress are compressive)	$\sigma 1$ = 3.5 MPa dip : 0 ° 	$\sigma 1$ = 4.0 MPa dip : 2 °
Zone 3 (Positive stress are compressive)	$\sigma 1$ = 5.5 MPa dip : 40 ° 	$\sigma 1$ = 3.5 MPa dip : 45 ° Tensile zone mesured

In the first zone, the value of the main compressive stress compute is very similar to tests results. The direction compute is lesser than test results.
In the second zone, the direction and the value of the main compressive stress compute are very similar to tests results.
In the third zone, the direction of the main compressive stress compute is very similar to tests results. The value for the modeling is greater than experimental results. In this zone, the tensile zone measured by the in-situ experimentation is also reproducing by the modeling.

For each tests zone, the main compressive stresses are found. So, the model is able to describe the displacement evolution, the damage and also the stress field of a structure. This result is obtained by taking into account simultaneously all phenomenon involves in the concrete (creep, damage and AAR evolution).

5 CONCLUSION

Facing the chemical swelling induced by the AAR, it was essential to perform a reliable diagnosis of the actual state of stress in the EDF dam, in order to evaluate the progression of the swelling. Two approaches were simultaneously considered, hopefully to confirm the relevancy of each method in the context of the large gravity dam: numerical modeling, using the finite elements modeling and *in situ* measurements, using the overcoring method. The two methods were led to their term and the independent results were compared.

This paper has shown that it was quite difficult to use the result of usual accelerated residual swelling test in order to predict the structural behavior of an AAR-damaged dam. In order to perform relevant calculations, the mechanical properties of the concrete in the structures have also to be precisely known. Measurements of strength, modulus and creep characteristic should be carried out. The method has been successfully tested. The results show that the fitting of the amplitude, using one significant displacement combined with the laboratory determination of the chemical kinetic parameters, allows other displacements of the dam and a realistic stresses field to be assessed. Based on this work, a prediction of the dam’s displacements, damage and stress fields have been carried out for the next decades.

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Regarding the *in situ* campaign the overcoring protocol had to be adapted to the particular conditions of the dam: low temperature of the concrete, outside difficult conditions and heterogeneity of the elastic parameters of the concrete. Two specific systems were designed to heat the pilot hole before the installation of the CSIRO cell and to maintain the highest temperature possible during curing time. Good coupling of the epoxy with the concrete was achieved in less than 22 hours, allowing performing successful overcoring tests in a reasonable amount of time. As the overcored samples were obtained intact, biaxial tests have been run on most of them, allowing to determine precisely the local elastic parameters and to evaluate their heterogeneity at the scale of the CSIRO cell. For each of the 4 measurement locations, great redundancy was obtained with the inversions, thus validating the stress estimations and limiting the number of needed tests. The rather low stress state (major principal stress inferior to 6 MPa) appeared not to be a limitation for the overcoring method, more often used in underground environments with higher stress states. Tested successfully in 1991 and in 2009 in the context of a large dam, the CSIRO cell proved to be adapted to stress determination and could therefore be considered for long term application as 3D stress monitoring with an in-place cell linked with an outdoor acquisition unit.

The comparison of the results of each method showed a high consistency in the orientation and values of the calculated stresses. The numerical modeling designed by EDF and the measurement campaign led by INERIS were performed independently and are therefore validated. Moreover the global process was to validate the understanding of the AAR swelling and the hypothesis put in the FEM through a confrontation with *in situ* measurements. Hence the combined approach is necessary to obtain a reliable and comprehensive diagnosis of the integrity and safety of the dam.

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