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Mehdi Asali, Bruno Capra, Jacky Mazars, Jean-Baptiste Colliat

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Numerical Strategy for Forecasting the Leakage Rate of Inner Containments in Double-Wall Nuclear Reactor Buildings

Mehdi Asali¹*,², Bruno Capra³, Jacky Mazars⁴ and Jean-Baptiste Colliat⁵

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Abstract

In the context of life extension of nuclear reactor buildings while ensuring safety and regulatory requirements, a numerical strategy is proposed to compute and forecast the air leakage rate evolution of inner containments in double-wall reactor buildings under standard long-term operation. In order to reduce the numerical cost of such complex computations, the proposed strategy is based on a coarse but adapted mesh together with a chained weakly-coupled thermo-hydro-mechanical modeling. The total leakage rate is then computed with a specially-designed tridimensional finite element based on a non-linear thermal analogy.

This methodology is used to model the behavior of the VeRCoRs mock-up, a simplified nuclear reactor building at scale 1:3, built and monitored by EDF. Results obtained until the first pre-operational pressurization test have been discussed in a dedicated benchmark organized by EDF. The proposed methodology provides delayed strains and leakage results in good agreement with available experimental data. A blind prolongation until the first decennial test of the mock-up is presented and analyzed.

1. Introduction

The containment building represents the third and last protection barrier of nuclear reactor buildings (NRB), after the fuel sheath and the casing of the primary coolant system. Thus, it plays a major role for ensuring the containment of radioactive elements in case of an accident and potential failure of the first two barriers.

This study concerns the double-wall reactor building technology without steel liner, which represents for instance 24 out of the 58 assets of EDF (French nuclear operator). In that case, most of the leak tightness is ensured by an inner containment in prestressed and reinforced concrete. Ageing mechanisms in concrete and prestressing losses are phenomena, among others, that could modify this tightness capacity over time. To ensure safety, internal leakage rate (ILR) tests are performed every ten years to measure the air leak rate of NRB and to check it stays below an imposed regulatory value (1.5 % of the internal volume per day). If not, expensive and time-consuming mitigation and repair techniques must be set up and outage of the tested unit may be extended, leading to important losses of income.

In the current context of life extension of its assets beyond the initially designed 40 years of operation, EDF has built an experimental containment vessel mock-up at scale 1:3 within the frame of the VeRCoRs project (VERification Réaliste du CONfinement des RéacteurS, which means realistic verification of reactor containments, see https://fr.amiando.com/EDF-vercors-project.html). Considering that drying of concrete is the main phenomenon governing the leakage rate evolution, the scale of the mock-up accelerates the ageing of the structure by a factor 9 compared to a full-size NRB, enabling EDF to have access to data representative of a 60-year-old NRB in 2021.

The behavior of the mock-up is finely monitored since the beginning of its construction with more than 500 sensors and 2 km of fiber optic cables acquiring daily measurements by default and hourly measurements during ILR tests. More than 1000 samples of on-site concrete are or will be used for material and behavior characterization. The concrete composition is also available for extra tests on specimens conducted by laboratories.

From January to October 2015, EDF has proposed a first international benchmark dedicated to the behavior of the mock-up until its first ILR test. Three main themes have been benchmarked:
- Early age, with the prediction of the gusset behavior from pouring to ten months;
- Containment history, with the prediction of strains, stresses and cracking in the whole structure at the end of prestressing and at the end of the 5.2 bar absolute pressurization plateau;

1R&D Consultant, OXAND France, Avon-Fontainebleau, France. *Corresponding author, E-mail: mehdi.asali@oxand.com
2Doctoral student, Univ. Lille, CNRS, Centrale Lille, Arts et Métiers Paris Tech, FRE 3723 – LML – Laboratoire de Mécanique de Lille, Lille, France.
3Technical and Innovation Manager, OXAND Group, Avon-Fontainebleau, France.
4Professor Emeritus, Grenoble Institute of Technology, Grenoble, France.
5Professor, Dept. of Mechanics, Univ. Lille, CNRS, Centrale Lille, Arts et Métiers Paris Tech, FRE 3723 – LML – Laboratoire de Mécanique de Lille, Lille, France.
Leakage, with the prediction of the total air leakage rate at the end of the pressurization plateau. The results of the benchmark (Corbin and Garcia 2016) have been presented and discussed during a dedicated workshop, held beginning of March 2016 at EDF R&D center “Les Renardières”, in France.

To predict accurately the air leakage rate of NRB, the coupling of thermal, hydric and mechanical (THM) effects has to be taken into account. Although a wide range of more or less sophisticated models are already available to represent each physical phenomenon occurring in concrete (such as drying, delayed strains, cracking or evolution of permeability), global strategies aiming at computing the final leakage rate of a full-size construction are less common. Indeed, a good compromise between the refinement of discretization, the complexity of behavior laws and the strength of couplings has to be reached to tackle the global problematic of air leakage rates for industrial purposes. From the point of view of computational cost, this leads either to fully-coupled non-linear strategies (Wang and Hutchinson 2005; Nicklash et al. 2005; Jason and Masson 2014) only applicable to structural elements or to full-structure computations based on weakly-coupled elastic strategies and post-treatments with the help of design codes formulae (Mozayan et al. 2012). In those strategies, the representation of localized cracking is often one of the most complex steps.

In that context, the present paper introduces a numerical strategy aiming at forecasting the leakage rate evolution of inner containments during operation. It focuses on a chained weakly-coupled THM modeling used to represent, at lower numerical cost, the physical behavior of the structure (Fig. 1). First, the models implemented for each physical phenomenon are described, with a 3D finite element (FE) specially designed for air leakage computation through unsaturated porous and cracked concrete. Then, the proposed strategy is applied to the VeRCoRs mock-up. Main obtained results are analyzed and compared with experimental data when available and benefit from discussions and feedback from the first VeRCoRs benchmark.

2. Modeling principles of the numerical strategy

After having justified (Asali et al. 2014) that the main assumption of a weak coupling between THM effects is valid for inner containments under standard operation, a 3D numerical strategy is proposed to compute the air leak rate. The underlying philosophy is to use simple physically representative models to describe accurately those complex structures. At that stage, no early age behavior of concrete is considered.

Despite the weak coupling of phenomena, computations are chained in the following order to take into account their relative influence.

2.1 Thermal model

First, the spatiotemporal temperature field $T(x,t)$ is computed (in K) with the classical non-linear heat equation,

$$\rho c_p \frac{\partial T(x,t)}{\partial t} + \nabla \cdot [-\lambda(T)\nabla T(x,t)] = 0 \quad (1)$$

where $\nabla$ is the differential “nabla” operator ($\nabla$ being the gradient operator and $\nabla \cdot$ the divergence operator), $\rho_c$ is the concrete density (in kg·m$^{-3}$), $c_p$ is the concrete thermal capacity (J·kg$^{-1}$·K$^{-1}$) and $\lambda$ is the concrete thermal conductivity field (in W·m$^{-1}$·K$^{-1}$).

2.2 Hydric model

In a second step, the water saturation field $S_r(x,t)$ is computed (values between 0 and 1). Considering that drying of concrete is mainly due to liquid water transfer, the hydric problem reduces to a single diffusion equation in (Eq. 2a) (Verdier 2001). Thermal dependency of the hydric diffusion coefficient $D(S_r(x,t), T(x,t))$ is modeled with an Arrhenius-type law as proposed by Granger (1995) in (Eq. 2b).

The sake of concision in all equations, the notation $D(T_r, T)$ for fields depending on other computed fields is now used.

$$\frac{\partial S_r(x,t)}{\partial t} + \nabla \cdot [D(T_r, T)\nabla S_r(x,t)] = 0 \quad (2a)$$

$$D(T_r, T) = \frac{K_w S_r k_{pc}}{\mu \phi} \frac{P_r T(x,t)}{T_0^{\alpha}} \exp\left(\frac{K_{\text{act}}}{R T_r}\right) \quad (2b)$$

where $K_w$ is the water intrinsic permeability of concrete (in m$^2$), $k_{pc}$ is the water relative permeability field (values between 0 and 1), $\mu$ is the water viscosity (in Pa·s), $\phi$ is the concrete porosity, $P_r$ is the capillary pressure field (in Pa), $T_0^{\alpha}$ is the reference temperature (in K) at which all hydric properties of (Eq. 2) are defined or measured, $E_a$ is the thermal activation energy of concrete (in J·mol$^{-1}$), $R$ is the universal gas constant (in J·mol$^{-1}$·K$^{-1}$).

The water retention model of Van Genuchten (1980) is used to define the evolution of the capillary pressure field with the saturation degree,

$$P_r (S_r) = P_r (S_r^{1-n} - 1)^\frac{1}{n} \quad (3)$$

where $P_r$ (in Pa) and $n$ (>$1$) are model parameters and $m = 1 - 1/n$. Mualem’s law (Mualem 1976) gives the evolution of the water relative permeability field with

Fig. 1 Representation of the THM strategy.
saturation,
\[ k_i(S_i) = \sqrt{S_i} \left[ 1 - \left( 1 - S_i^{1/n} \right)^n \right]^2 \]  

(4)

2.3 Mechanical model
The third step of the methodology aims at computing the total strain state of concrete. Prestressing losses in tendons are modeled according to ETC-C formulae (AFCEN 2010) with a perfect bond between concrete and steel cables. Passive reinforcement bars are not explicitly taken into account.

In order to model delayed effects in prestressed concrete over time, the concrete total strain tensor field \( \varepsilon(x, t) \) is split into five main components,
\[ \varepsilon(x, t) = \varepsilon^e + \varepsilon^{bc} + \varepsilon^{dc} + \varepsilon^{ds} + \varepsilon^{th} \]  

(5)

where \( \varepsilon^e \) is the elastic strain tensor, \( \varepsilon^{bc} \) is the basic creep tensor, \( \varepsilon^{dc} \) is the drying creep tensor, \( \varepsilon^{ds} \) is the drying shrinkage tensor and \( \varepsilon^{th} \) is the thermal expansion tensor.

(a) Thermal expansion model
The thermal expansion tensor is considered as isotropic and proportional to the temperature,
\[ \varepsilon^{th}(x, t) = \alpha_{th}(T(x, t) - T(x, 0))I_3 \]  

(6)

where \( \alpha_{th} \) is the thermal expansion coefficient (in K\(^{-1}\)) and \( I_3 \) is the 3D identity matrix.

(b) Drying shrinkage model
The drying shrinkage tensor is considered as isotropic and is proportional to the saturation degree,
\[ \varepsilon^{ds}(x, t) = \kappa_{ds}(S_i(x, 0) - S_i(x, t))I_3 \]  

(7)

where \( \kappa_{ds} \) is the drying shrinkage coefficient (without unit).

(c) Drying creep model
Concrete’s total creep is higher when drying compared to basic creep in autogenous conditions (Pickett 1942). In order to take this Pickett’s effect into account, the drying creep strain is considered as proportional to the stress and hydric states of concrete,
\[ \varepsilon^{dc}(x, t) = \kappa_{dc} \left| S_i(x, t) \right| \frac{\sigma(x, t)}{E_x} \]  

(8)

where \( \kappa_{dc} \) is the drying creep coefficient (in Pa\(^{-1}\)), and \( \sigma \) is the effective stress tensor field (in Pa).

(d) Basic creep model
Basic creep is described through a Burger’s rheological model (Fig. 2) modified by Hilaire (2014) in order to take into account a possible dissymmetry between creep in tension and in compression (Eq. 16), a logarithmic evolution of creep strains with time (Eqs. 11, 13) and biaxial creep effects with a creep Poisson’s ratio different from elasticity (Eqs. 11, 12). Evolution laws of the model are the following,
\[ \varepsilon^{bc}(x, t) = \varepsilon^{kv}(x, t) + \varepsilon^{sm}(x, t) \]  

(9)

\[ \sigma_a(x, t) = \alpha_{bc}(\bar{\sigma}) + \bar{\sigma} \]  

(10)

\[ \dot{\eta}_{sm}(t)\varepsilon^{sm} = (1 + \nu_{bc})\sigma_a - \nu_{bc}tr(\sigma_a)I_3 \]  

(11)

\[ \dot{\eta}_{kv}\varepsilon^{kv} + \dot{\eta}_{dc}\varepsilon^{dc} = (1 + \nu_{bc})\bar{\sigma} - \nu_{bc}tr(\bar{\sigma})I_3 \]  

(12)

where \( \varepsilon^{kv} \) is the reversible basic creep strain field, \( \varepsilon^{sm} \) is the non-reversible creep strain field, \( \sigma_a \) is the stress field taking into account a decrease or an increase of creep strain in tension compared to compression through the use of the coefficient \( \alpha_{bc} \) and \( \nu_{bc} \) is the basic creep Poisson’s ratio. Moisture and temperature impact creep strains as follows,
\[ \dot{\eta}_{am}(t, T, S_i) = \frac{k_{am}}{S_i} \exp \left( \frac{E_a}{R} \left( 1 - \frac{1}{T/T_{ref}} \right) \right) \]  

(13)

\[ \dot{\eta}_{kv}(T, S_i) = \frac{\eta_{kv}}{S_i} \exp \left( \frac{E_{kv}}{R} \left( 1 - \frac{1}{T/T_{ref}} \right) \right) \]  

(14)

\[ \dot{\eta}_{dc}(T, S_i) = \frac{k_{dc}}{S_i} \exp \left( \frac{E_{dc}}{R} \left( 1 - \frac{1}{T/T_{ref}} \right) \right) \]  

(15)

where \( k_{am} \) is the stiffness associated to the non-reversible ageing dashpot (in Pa), \( k_{dc} \) is the stiffness associated to the reversible creep strains (in Pa), \( \eta_{kv} \) is the viscosity associated to the reversible creep strains (in Pa\(\cdot\)s) and \( E_{dv} \) is the thermal activation energy associated to water viscosity (in J\(\cdot\)mol\(^{-1}\)).

(e) Elastic-damageable model
Isotropic damage evolution is described with the \( \mu \)-model (Mazars et al. 2015) in order to take into account the unilateral effect (crack closing and reopening) occurring with cycling inflating tests. Main evolution laws are,
\[ E \varepsilon^e(x, T) = (1 + \nu)\bar{\sigma} - \nu tr(\bar{\sigma})I_3 \]  

(16)
\[ \sigma(x,t) = (1-d)\tilde{\sigma}(x,t) \]  
\[ d(x,t) = 1 - \left(1 - d\right)Y(\varepsilon_{in}) = Ae^{B(1-S_{S})} \]  
\[ \varepsilon_{in}(x,t) = \varepsilon - \varepsilon_{e} + \chi(\varepsilon_{e} - \varepsilon_{f}) \]

where \( E \) is the Young’s modulus of concrete (in Pa), \( v \) its Poisson’s ratio, \( d \) is the scalar effective damage variable field (values between 0 and 1, a non-null value meaning active cracking either in tension or compression), \( \sigma \) is the total stress tensor field (in Pa), \( A \) and \( B \) are coefficients depending on the stress state of concrete and defined from post-peak responses of concrete in pure tension and pure compression, \( Y \) is a driving variable for damage evolution depending on the equivalent strain tensor \( \varepsilon_{e} \), \( \chi \) is a coupling coefficient between elastic and total creep strains (value between 0 and 1).

To avoid mesh sensitivity due to the softening behavior of concrete, parameter \( B \) is adjusted with element sizes (Hillerborg et al. 1976).

2.4 Air leakage model

Air leakage computation is the final step of the methodology. Cracks within concrete being a localized but important path for leakage, a dedicated 3D finite element (FE) has been developed to consider an equivalent cracking state without explicitly meshing the defects. Its principle, shown in Fig. 3, consists in superimposing a Darcy’s flow within the unsaturated porous concrete matrix and a Poiseuille’s flow within a perfectly plane crack.

Under the assumption of small perturbations, all quantities are expressed in relation to the system’s initial configuration and the crack is seen as a 2D plan of area \( \Sigma \) (in m²) associated to a uniform opening \( w \) (in m) in the 3D porous matrix of volume \( \Omega \) (in m³). The air flow is considered stationary and compressible, air being a perfect gas. Air flux field \( q_{a} \) within the matrix (in kg·s⁻¹·m⁻²) is following Darcy’s law,

\[ \begin{align*}
q_{a} = -\frac{M_{g}}{RT\mu_{g}}k_{m}^{*}(S_{S}) \nabla P_{g}^{2}(x,t)
\end{align*} \]

where \( k_{m}^{*} \) is the air relative permeability field (values between 0 and 1), \( \mu_{g} \) is the air viscosity (in Pa·s), \( M_{g} \) is the air molar mass (in kg·mol⁻¹) and \( P_{g} \) is the air pressure field (in Pa). The evolution of the air relative permeability field follows Mualem’s law as modified by Verdier (2001),

\[ k_{m}^{*}(S_{S}) = \sqrt{1 - S_{S}^\alpha(1 - S_{S}^\alpha)^{\mu/2}} \]

Air flux field \( q_{p} \) within the crack (in kg·s⁻¹·m⁻²) is following Poiseuille’s law with no friction or tortuosity coefficient to reduce the flow, which is a conservative assumption,

\[ q_{p} = -\frac{M_{g}}{RT\mu_{g}}\frac{w''}{24}(I_{3} - n_{e}^{*} \otimes n_{e}^{*})\nabla P_{g}^{2}(x,t) \]

where \( n_{e}^{*} \) is a unit crack surface normal. Adding the two contributions (Eqs. 20, 22) to mass conservation leads to the strong formulation of the problem to solve,

\[ \nabla \cdot q_{a} + \nabla \cdot q_{p} = 0 \]

Complete details concerning the final FE formulation and implementation can be found in (Asali et al. 2014).

Crack openings and orientations in each FE, needed to compute the total leakage rate, are obtained according to (Jourdain 2014) following (Matallah et al. 2010). In Fig. 4, a crack-effective strain tensor field \( \varepsilon_{cod} \) is defined from the elastic-damageable behavior of concrete (Eq. 24). Its maximal eigenvalue in tension (if any) is used to compute the crack opening (Eq. 25) and the associated eigenvector defines the crack surface normal.

\[ \varepsilon_{cod}(x,t) = \varepsilon - \frac{1}{E}(1 + v)\sigma - vtr(\sigma)I_{3} \]

\[ \varepsilon_{f} = \frac{1}{\sqrt{2}}\max\left[ \max\left(\text{Sp}(\varepsilon_{cod})\right), 0 \right] \]

where \( \text{Sp}(\varepsilon_{cod}) \) is the spectrum of \( \varepsilon_{cod}^{*} \), i.e. the set of its eigenvalues.

Total air leakage is then defined as the quantity of air leaving the internal volume of the mock-up. Thus, nodal air flux vectors are projected and integrated over the whole internal surface, giving a mass leak rate value.
MQ expressed in kg·s\(^{-1}\). In order to compare leakage rates of all NRB independently of each ILR test temperature and pressure conditions, EDF converts mass flow rates into volumetric flow rates expressed at normal temperature and pressure conditions,

\[
Q^{\text{norm}}_Q = 3600 \frac{R}{M_g} \frac{T^{\text{norm}}}{P^{\text{norm}}} Q_0
\]

(26)

where \(Q^{\text{norm}}_Q\) is the volumetric total leakage rate (in \(\text{Nm}^3\cdot\text{h}^{-1}\)) at normal temperature \(T^{\text{norm}} (0\, ^\circ\text{C})\) and normal pressure \(P^{\text{norm}} (101315\, \text{Pa})\).

3. Application to the VeRCoRs mock-up

3.1 Used tools

The numerical strategy presented in section 2 is implemented within the general framework of Code_Aster (http://code-aster.org/). Code_Aster v12.3 is used as the FE solver for all THM and air leak computations. The cracked FE of section 2.4 is developed as a new nonlinear thermal model within Code_Aster. The concrete delayed strain model of section 2.3 is implemented as an external user behavior law through MFront v2.0.1 (http://tfel.sourceforge.net/).

3.2 Mesh

The coarser mesh proposed for the benchmark is linearized before being used for all computations. It includes 34694 nodes in 3D concrete elements (penta- and hexahedra) and 16066 nodes in 1D cable elements (bars). 295 cables (horizontal, vertical, gamma and dome) and their deviations are taken into account (Fig. 5(a)). Despite vertical and circumferential coarse meshing of concrete (Fig 5(b)), the 10-element progressive fineness within the 40 cm thickness is suitable to represent phenomena linked to drying (Fig. 5(c)).

3.3 Boundary conditions and loadings

As the overall structure is subject to a changing environment as well as to a complex construction phasing process, the on-site evolutions of loadings and boundary conditions (BC) are simplified before being applied over the whole considered time frame. The main construction planning of the mock-up is considered as such:
- Erection is done at once, the starting date of all computations is on 30 November 2014;
- Due to water aspersion on the mock-up’s walls, the structure begins to dry only from 1 April 2015;
- Prestressing occurs between 6 May and 12 August 2015 with a reduced number of tensioning sequences;
- Two main life phases of the structure are occurring: pre-operational until 28 January 2016 during which the mock-up is subject to its external environment, then operational starting on 1 April 2016. The transition between both phases is linear.
- The first three ILR tests of the mock-up are considered in the present study: two pre-operational tests (PT0 and PT1) occurring between 10-14 November 2015 and 24-28 January 2016 respectively, and the first decennial test (DT1) between 20-24 April 2017.

(a) Thermal BC and loadings

Soil temperature applied at the bottom of the raft is kept constant during the whole computation at 10.5 °C (initial value). During pre-operation, measurements within concrete are available to calibrate imposed temperatures. No daily temperature variations are considered, only linear evolutions between extremal values.

Operating temperature conditions are 35 °C at the inner side and 15 °C at the outer side of the wall. Temperature is kept constant during each decennial test at 10 °C (Fig. 6).

(b) Hydric BC and loadings

Hydric conditions are naturally expressed in terms of
relative humidity (RH, value between 0 and 1). In order to convert RH to saturation, Van Genuchten’s model (Eq. 3) is completed with Kelvin’s law,

$$P_s(RH, T) = -\frac{\rho_lRT}{M_f}\ln(RH)$$

(27)

where $\rho_l$ is the water density (in kg·m$^{-3}$) and $M_f$ is the water molar mass (in kg·mol$^{-1}$).

As for temperature, soil RH is kept constant at 0.98 (initial value). During pre-operation weather station measurements are imposed at the inner and outer sides with linearization between extremal values. Operating HR conditions are 0.45 at the inner side and 0.6 at the outer side of the containment, with no interruption during decennial tests (Fig. 7).

(c) Mechanical BC and loadings

As no early age behavior is considered, initial stress and strain states are null. The bottom of the raft is embedded during the whole computation. Reinforcement bars are not explicitly meshed, but participate in the dead weight of the structure (extra 100 kg·m$^{-3}$ for concrete density). The raft and anchoring ribs remain visco-elastic without damage. Prestressing is simplified compared to the real construction planning to only 14 grouped steps (instead of 41 sequences in 16 phases), with an initial tension of 848 kN for each cable. All ILR tests are identical: they last four days, with a 24-hour ramp from standard atmosphere to maximal pressure (5.2 bar abs.), a 24-hour plateau at maximal pressure and 48 h of deflating (Fig. 8).

(d) Air leakage BC and loadings

During ILR tests, inner and outer air pressure follows the evolution of Fig. 8. The raft being submerged during ILR tests, it is not considered as a possible leakage pathway (null flux).

3.4 Modeling and material parameters

Concerning concrete, the mean value of parameters measured on-site for each lift is used when available (density, porosity, Young’s modulus, tensile strength, thermal expansion coefficient, air apparent permeability). Other concrete parameters are either provided (thermal capacity, linear thermal conductivity with temperature, activation energy, fracture energy) or identified on specific lab tests (water intrinsic permeability, Van Genuchten’s model parameters, drying shrinkage coefficient, basic and drying creep parameters) (Fig. 9).

Due to lack of some characterization tests, biaxial creep effects, distinction between creep in tension and in compression and coupling of creep with damage have not been taken into account.

Cable parameters come from design and manufacturer values. The complete set of parameters used for the present study is available in Appendix A.

4. Results and discussions

In this section, obtained numerical results are presented and analyzed. They are compared to available experimental data at four observation points, considered as representative of the behavior of the structure in standard areas (far from any change in concrete geometry or tendon deviation). Points H1, H2, H5 and H6 are monitored with vibrating wire and temperature sensors to measure and correct vertical and tangential strains. They all lie on the same horizontal plane at mid-height of the cylindrical part. Groups H1-H2 and H5-H6 are diametrically opposed, H1 and H5 being located at the outer side, H2 and H6 at the inner side of the wall (see Fig. 10).
4.1 Temperature in concrete

Temperature of concrete follows external seasonal variations during the first year of the structure. Slowly changing BC and loadings associated to the used thermal parameters lead to an almost homogenous temperature of concrete within the cylindrical part and within the dome (less than 1 °C-difference between mid-thickness and outer sides).

4.2 Drying of concrete

Figure 11 shows the evolution of the saturation degree along line H1-H2 in the thickness of the cylinder (40 cm) before each ILR test. During operation, leakage tests DT2 to DT6 occur in May 2018, June 2019, July 2020, August 2021 and September 2022 respectively.

As for temperature, saturation at the inner and outer sides of the mock-up is following seasonal variations during the first year. With the identified Van Genuchten’s parameters, imposed RH values of 0.6 and 0.45 correspond to imposed saturation values of 0.64 and 0.57 respectively.

In the dome and cylindrical part, the saturation level at the core of concrete thickness is decreasing with little impact of imposed pre-operational RH cycles, meaning global drying of the structure.

Lacking concrete sorption isotherms data to identify Van Genuchten’s parameters independently of the concrete water intrinsic permeability on Fig. 9(a) and some RH measurements within the mock-up, drying kinetics of the model is not yet experimentally confirmed.

4.3 Evolution of concrete total strains

Figure 12 shows the evolution of total strains within the mock-up at the four observation points. All measured strains of Fig. 12 are corrected from temperature effects according to the following formula (with null initial strains),

$$\varepsilon_{\text{corr}}(t) = \varepsilon_{\text{raw}}(t) + \alpha_s(T(t) - T(0))$$  \hspace{1cm} (28)

where $\varepsilon_{\text{corr}}$ is the corrected measured strain, $\varepsilon_{\text{raw}}$ is the raw measured strain and $\alpha_s$ is the wire thermal expansion coefficient (in K⁻¹, steel material). Until the first
ILR test, computed total strains (dashed lines) are compared with available corrected strain measurements (plain lines). The mechanical computation is then extended to the first decennial test DT1.

According to Fig. 12(a), kinetics of delayed effects as well as strain levels are well reproduced in the vertical direction. According to Fig. 12(b), kinetics of delayed effects is well reproduced in the tangential direction, but compressive strain levels are globally underestimated by the model, especially at points H5 and H6. This gap may be explained by the fact that only uniaxial creep tests are available to calibrate a biaxial creep model leading to a homogeneous behavior of concrete in the vertical and tangential directions.

To be more precise in the level of short-term strains, the drying shrinkage model may have to be modified. As shown in Fig. 9(b), the chosen model is preferentially able to fit the long-term constant level of strains and is less accurate for the first months. Its impact on the total strain in Fig. 9(d) remains noticeable for the mock-up during the first months after erection. However, keeping in mind that the strategy aims at representing the long-term behavior of full-size NRB, this effect could be accepted in a first approach.

According to Fig. 12, numerical results between the four considered points show less discrepancy compared to experimental data. The gap between H2 and H5 vertical strains is around 25% for the computation and 45% for measurements. For tangential strains, the gap between H1 and H6 is around 10% for the computation and 25% for measurements. This observation may be due to the homogeneous mechanical properties taken into account in the computation, which are the mean values of the real scattering of on-site concrete properties between the different lifts. Lower scattering of numerical and experimental tangential strains compared to the vertical ones is due to the fact that all four points were taut together in the horizontal direction (they are at the same height), which was not the case in the vertical direction (they are diametrically opposed).

4.4 Damage of concrete

Figure 13 shows the computed damage field in the mock-up at the end of each maximal pressure plateau. For the sake of clarity, only maximal values (between 0.7 and 1) are kept. Areas where cracks are connected and going through the concrete thickness are also highlighted.

During PT0, a small number of FE is damaged. The gusset and its junctions to anchorage ribs are lightly damaged at the outer side of the containment (value lower than 0.2, in the first layer of FE only). According to Fig. 13(a), areas at the top and bottom of the material hatch are more highly damaged, from the inner side to the middle of the thickness. Associated crack openings range from 10 to 51 µm. Compared to visual inspection in Fig. 14 (not able to detect crack openings lower than 100 µm), the computed results are in good agreement in the cylinder, dome and hatch areas where no crack is identified but only defects like porosity lines, corrosion or honeycombing.

Nevertheless, vertical cracks are observed at the outer side of the gusset during the inflation test (in thick green lines in Fig. 14). Over the 30 identified cracks, only 6 have an opening exactly equal to 100 µm, the other ones having no measured value. This cracking originates...
Fig. 13 Evolution of damage in the mock-up during ILR tests.
from the early-age behavior of the structure and, despite being closed by prestressing, reopens during ILR tests. As the present computation starts with a null initial state and a structure built in only one step, this phenomenon could not be represented in associated results. But the capacity of the model to close and reopen cracks enables to take into account another initial stress-strain-damage state, coming from previous early-age computation for instance.

During PT1, the first thin layer of FE at both sides of the containment is damaged at a low average level (value around 0.4). This generalized surface behavior, not encountered in other computed ILR tests, may be due to temperature and humidity gradients that are still evolving during the pre-operational phase. The maximal crack opening at mid-thickness within the material hatch increases to 90 µm. According to Fig. 13(b), more highly damaged areas on the inner and outer sides (values greater than 0.7) are linked to geometrical accidents in concrete and to tendons deviations: anchorage ribs, material and personal hatches. As it is not connected through the concrete thickness, this diffuse surface cracking does not impact significantly the total leakage rate of the mock-up.

During DT1, the first layer of FE at the inner side of the containment is damaged at a medium average level (value around 0.5), which may be due to the temperature gradient imposed between the internal and external sides during operation. Cracking progresses in the material hatch: 70 % of its thickness is continuously cracked from the inner side, with a maximal opening of 160 µm. According to Fig. 13(c) and contrary to previous pre-operational tests, two cracks connected through the thickness are computed at the two junctions between the gusset and anchorage ribs. The crack opening within the gusset and the first lift above it between 200 and 400 grades evolves from 150 µm at the inner side to 40 µm at the outer side.

### 4.5 Air leakage of the mock-up

Table 1 shows the evolution of air leakage rate computed at the end of maximal pressure plateau for the first three ILR tests (PT0, PT1 and DT1). The total leakage is compared to in-situ measurements for the two pre-operational tests. The distribution of air leakage by area is also shown. However this distribution is provided at the inner side for the computation while experimental results are only collected at the outer side of the inner containment. It can be noticed that only 57 % of the total internal leak is measured at the outer side: it is indeed difficult to inspect 1100 m² of concrete during the 24 h of maximal pressure and the focus is put on visually leaking areas (soap aspersion). The influence of modeled cracking on the total leakage rate is quantified by comparing computed results to an extra computation taking into account the unsaturated-concrete darcian flow only.

A first computation has been launched using the mean value of air apparent permeabilities \((3.13 \cdot 10^{-16} \text{ m}^2)\) as the intrinsic permeability of concrete. In that case, no
Klinkenberg’s effect (Klinkenberg 1941) is taken into account leading to overestimating the total leakage rate during PT0 (42.9 Nm$^3$·h$^{-1}$, +557 % compared to the experimental measure). Thus the intrinsic permeability value for the proposed computation is calibrated on this first ILR test (to 5·10$^{-17}$ m$^2$). For PT0 and PT1, this value enables to compute total air leak rates that are very close to experimental measurements (respectively -13 % and -3 %).

For the proposed computation, air leakage is mainly due to concrete porosity during PT0 and PT1 (98 % and 84 % of the total leak respectively). Even if cracking occurs at that stage, it is not sufficiently going through concrete thickness to participate significantly to the total leakage, as stated in the previous section. This is corroborated by analyzing the leakage rate distribution by areas, which is globally proportional to the surface of each zone. The influence of the hatch and gusset does not evolve between the first two tests and all the leakage increase is due to the cylinder and to the dome, which are thinner and on which drying has more effect during those three months.

During DT1, cracking of concrete is contributing significantly to the total leakage rate. Indeed, leakage through porosity does not evolve between PT1 and DT1 (and only represents 14 % of the total leak), contrary to leakage through the gusset and the cylinder which increases strongly (respectively 357 and 7.8 times). Associated to the damage (Fig. 13(c)) and air flux mapping, the leaking increase is mainly caused by the continuous cracks at the junctions between the gusset and the two anchorage ribs.

The computed total leakage value of 55.6 Nm$^3$·h$^{-1}$ may be high due to the conservative assumption of not considering any friction coefficient for the air flow within cracks. For instance, using a classical reduction factor of 0.1 (Picandet 2001) on the Poiseuille’s part of the flow during DT1 would lead to a total leakage rate of 12.5 Nm$^3$·h$^{-1}$. In that case, the leak through concrete porosity would still be prevailing (62 %). This coefficient could be identified on experimental measures when available.

Considering that air leak distribution measured during PT0 at the outer side could be extrapolated at the inner side of the containment, experimental data show a local behavior different from the one predicted by the computation. The gusset area participates significantly to the total leakage. Indeed, leakage through the gusset and the cylinder which increases strongly (357, 7.8 times). As associated to the damage (Fig. 13(c)) and air flux mapping, the leaking increase is mainly caused by the continuous cracks at the junctions between the gusset and the two anchorage ribs.

The identification procedure, based on lab results for the mock-up, could be adapted to in-situ total strains measurements, for instance to define a representative creep Poisson’s coefficient. As only few characterization tests are available on full-size NRB compared to VeRCoRs, this feature seems necessary. (5) The 3D FE designed for air leak computations is currently modeling one crack related to the mechanical state of concrete. But its formulation enables patching as many independent cracks as desired in the same FE. This feature could also be used to consider initial defects of the structure such as porosity lines or leaking lift joints. (6) Finally, the non-early-age effect assumption of the strategy may need to be reconsidered in light of experimental observations. Lack of initial early-age cracks leads to underestimating the gusset leaking during PT0, which would be problematic only if crack openings in this area evolve with time. If not, the simplest way to take into account the initial damage of the gusset could be to locally increase the intrinsic permeability to recover experimental results. If crack openings increase in the gusset, an initial early-age computation would be necessary to define a non-null strain-stress-damage state for the proposed strategy which is able to close them during prestressing and reopen them during ILR tests.

Considering conservative assumptions, using simple modeling techniques (non-linear thermal analogy for drying and for air-leak computations) and a delayed strain model representative of important phenomena with few parameters to identify (compared to other models available in the literature and FE codes), the proposed methodology is numerically cost-effective and thus enables taking into account variabilities and uncer-
tainties of main parameters. It could be used as a qualitative and quantitative decision-support tool to help operators: (1) Pre-empting and optimizing leak mitigation actions before each decennial ILR test and consequently avoiding outage extensions and associated losses of income; (2) Forecast the behavior of their assets in the context of long-term operation and life extension beyond design in order to demonstrate their compliance with safety and regulatory requirements.

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## Appendix A: Modeling and material parameters

| Parameter                                      | Symbol | Value     | Unit          | Source                                      |
|------------------------------------------------|--------|-----------|---------------|---------------------------------------------|
| Concrete density                              | $\rho$ | 2395      | kg·m$^{-3}$   | Mean value of on-site lifts                 |
| Concrete thermal capacity                     | $C_p$  | 880       | J·kg$^{-1}$·K$^{-1}$ | Lab test result                           |
| Concrete thermal conductivity $\lambda(T)$   | $\lambda(T)$ | 6.77T – 239  | 10$^3$ W·m$^{-1}$·K$^{-1}$ | Lab test result                           |
| Concrete water dynamic viscosity               | $\mu_w$ | 1.002·10$^{-3}$ | Pa·s |                                            |
| Concrete porosity                             | $\phi$ | 0.146     | -             | Mean value of on-site lifts                 |
| Concrete thermal activation energy $E_a$      | $E_a$  | 28000     | J·mol$^{-1}$  | Lab test result                           |
| Universal gas constant                        | $R$    | 8.314     | J·mol$^{-1}$·K$^{-1}$ |                                            |
| Reference temperature                         | $T_{ref}$ | 20 (293.15) °C (K) | Lab test result |                                            |
| Liquid water density                          | $\rho_l$ | 998.3     | kg·m$^{-3}$   |                                            |
| Water molar mass                               | $M_l$  | 18·10$^{-3}$ | kg·mol$^{-1}$ |                                            |
| Concrete water intrinsic permeability $k_{int}^i$ | $k_{int}^i$ | 6.0737·10$^{-20}$ | m$^2$ | Identification on weight loss lab test curve |
| Van Genuchten’s parameters                    | $n$    | 1.3014    | -             | Mean value of on-site lifts                 |
|                                                 | $P_r$  | 17.607·10$^6$ | Pa |                                            |
| Concrete Young’s modulus                      | $E$    | 36.849    | GPa           | Default value (no info)                    |
| Concrete Poisson’s ratio                      | $\nu$  | 0.2       | -             | Default value (no info)                    |
| Reinforced concrete density                   |       | 2495      | kg·m$^{-3}$   | $\rho_s + 100$                             |
| Concrete thermal expansion coeff. $\alpha_{th}$ | $\alpha_{th}$ | 1.22·10$^{-3}$ | K$^{-1}$ | Mean value of on-site lifts                 |
| Basic creep Poisson’s ratio                   | $\nu_{bc}$ | 0.2     | -             | Default value (no info)                    |
| Basic creep tension/compression coeff. $\alpha_{bc}$ | $\alpha_{bc}$ | 1   | -             | Default value (no info)                    |
| Water viscosity activation energy $E_a^*$      | $E_a^*$ | 18700     | J·mol$^{-1}$  | (Hilaire 2014)                             |
| Concrete fracture energy (tension)            |       | 100       | N·m$^{-1}$    | Lab test result                           |
| Concrete yield stress (tension)               | $f_t$  | 3.89      | MPa           | Mean value of on-site lifts                 |
| Damage parameter (tension)                    | $A_t$  | 0.9       | -             | Default values (no info)                   |
| Damage parameters (compression)               | $A_p$  | 1.25      | -             | Default values (no info)                   |
| Coeff. Coupling creep and damage $\chi$      | $\chi$ | 0         | -             |                                            |
| Drying shrinkage coefficient                  | $\kappa_{dr}$ | 1.0265·10$^{-3}$ | - | Identification on 1D shrinkage lab test curve |
| Basic creep coefficients                      | $\eta_{bc}$ | 141.84·10$^6$ | GPa·s | Identification on 1D basic creep lab test curve |
|                                                 | $k_{am}$ | 81.63     | GPa           | Identification on 1D total creep lab test curve |
| Drying creep coefficient                      | $\kappa_{dc}$ | 6.8968·10$^{-11}$ | Pa$^{-1}$ | Identification on 1D total creep lab test curve |
| Air molar mass                                 | $M_g$  | 29·10$^{-3}$ | kg·mol$^{-1}$ |                                            |
| Air dynamic viscosity                          | $\mu_g$ | 1.8·10$^{-5}$ | Pa·s | Identification on 1D creep test curve       |
| Concrete air intrinsic permeability $k_{int}^g$ | $k_{int}^g$ | 5·10$^{-17}$ | m$^2$ | Identification on first ILR test            |
| Wire steel thermal expansion coeff.            | $\alpha_s$ | 1.15·10$^{-4}$ | K$^{-1}$ | (Corbin and Garcia 2016)                   |