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MODELING THE THERMOMECHANICAL BEHAVIOUR OF REINFORCED CONCRETE STRUCTURES EXPOSED TO NATURAL FIRE

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ABSTRACT

The prescriptive design guidelines are an essential basis when assessing a required nominal fire resistance. However, a prescriptive approach cannot be used to assess the fire performance of reinforced concrete (RC) structures during the cooling phase of natural fire scenarios. IRSN conducted a numerical study with the intention of modelling RC structures exposed to natural fire. For this purpose, an adapted finite element model was developed, using CAST3M finite element code (Verpeaux et al., 1988). The behaviour of concrete material was simulated using the Mazars elastic isotropic damage model (Mazars, 1984), while a user-defined procedure was used to calculate the concrete properties taking into account the full temperature history of the analysed structure. The effect of transient creep strain according to the model of Anderberg and Thelandersson (1976) was explicitly considered. In order to validate the implemented model, the transient tests conducted by Anderberg and Thelandersson (1976) and the experiments conducted by Maclean (2018) have been simulated considering siliceous and calcareous concrete specimens according to EN 1992-1-2 (2004). The studies presented in this paper demonstrate the adequacy of the implemented mechanical model, taking into account the transient creep strain and realistic concrete properties, for describing the behaviour of concrete structures exposed to natural fire considering the fire decay phase.

INTRODUCTION

Over several years now, the French nuclear fire safety regulation has turned from prescriptive to objective requirements. Consequently, the assessment of fire behaviour of reinforced concrete (RC) structures moves towards a performance-based approach with the consideration of thermal loading induced by natural fire scenarios. Three stages can be distinguished during a natural fire scenario: *(i)* a rapid-fire growth stage where the RC structures heat up *(ii)* a fully developed fire stage where the thermal loading of the RC structures is maximum *(iii)* a decay stage until the fire extinction where the thermal loading decreases. This last stage is currently not covered by the usual prescriptive approaches to assess the fire behaviour of RC structures. Moreover, in the past few years, unexpected accidents have shown that this decay stage can lead to dramatic failure of structures. For example, a dramatic failure occurred in an underground car park in Gretzenbach, Switzerland, after approximately 90 min of fire, while it was in the cooling phase, resulting in the death of seven firefighters. This delayed failure may result from a combination of factors which lies especially on the properties of concrete itself. In this context, this paper investigates the development of numerical models able to predict the behaviour of RC structures during both heating and cooling phases of a natural fire.

EFFECTS OF COOLING PHASE ON FIRE BEHAVIOUR OF CONCRETE STRUCTURES

Concrete has a relatively low thermal conductivity, which means that, when a concrete element is exposed to high temperatures, it might take hours for the conductive heat flow to reach the core of the element. This is due to the thermal inertia of concrete which implies massive cross-section elements need time to reach thermal equilibrium. Figure 1 shows the temperature distribution in a 20 cm thick RC wall section exposed to natural fire on one side, as obtained by thermal analysis with the CAST3M[1] software. The steel reinforcement was positioned at 5 cm from the shuttered surface. The heating phase follows the ISO-834 fire time-temperature curve for 2 hours. For the cooling phase, a linear time-decrease of temperature was adopted using a cooling rate of 5°C/min. Following EN 1992-1-2 (2004), thermal properties of concrete are assumed as recoverable in the cooling phase due to lack of data. The temperature of the core of the concrete section continues to rise during the cooling phase. The steel reinforcements reach a temperature of 362°C at the end of the heating phase (120 min). About 60 minutes later, this temperature has increased to 439°C (see Figure 2). With a cooling rate of 3°C/min, the reinforcements reach a maximal temperature of 463°C, whereas if a cooling rate of 10°C/min is adopted, the reinforcements reach a maximal temperature of 417°C.

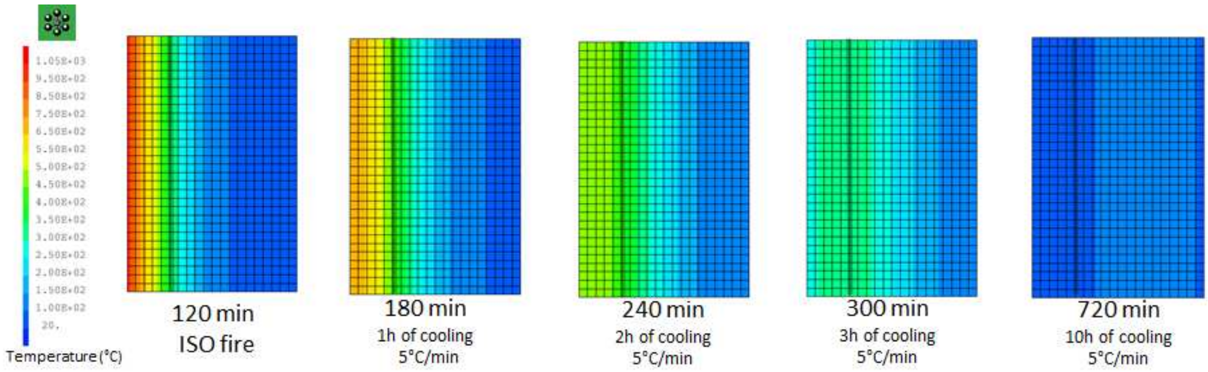


Figure 1. Temperature maps in a 20 cm thick concrete wall, at different time-steps after a 2 hours ISO-834 fire, with a cooling rate of 5°C/min (unit: °C)

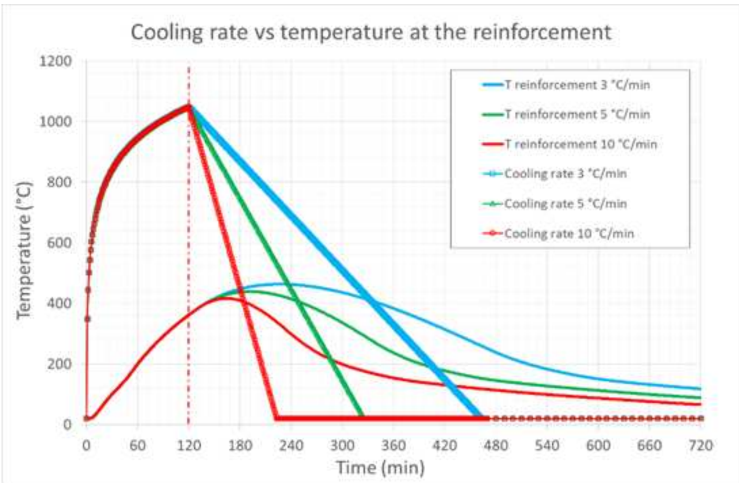


Figure 2. Time-evolution of gas phase temperature and reinforcement temperature for different cooling rates

[1] CAST3M: A general purpose code for solving partial differential equations by the finite element method, developed by the French Atomic Energy Commission (CEA).

In fire conditions, mechanical properties of concrete are irreversibly reduced during the heating phase and the cooling phase. The extent of the mechanical properties reduction depends on the maximum temperature reached in the concrete. The EN 1994-1-2 (2005) specifies an additional 10% reduction in strength during cooling. However, experimental tests have shown that this value depends on the cooling method (i.e. cooling rate). For instance, experiments by (Botte and Caspeele, 2017) show that the water-quenched concrete samples lost more strength than the water-sprayed ones. Moreover, restrained forces in the damaged structure may occur resulting from concrete thermal contraction during cooling. Consequently, there is a growing interest in understanding how concrete structure behaves during the cooling phase of fire exposition.

EFFECTS OF TRANSIENT CREEP STRAIN

Transient creep strain (TCS) can be observed when concrete is heated for the first time under sustained load (Gernay and Franssen, 2012). For a concrete structural member, an important part of the deformation at temperatures above 500°C can be associated to transient creep strain. TCS seems to be almost independent from the type of concrete, moisture and thermal expansion. However, the aggregate-cement ratio has an important influence on TCS. The addition of more aggregate reduces the magnitude of TCS, which seems to agree with the assumption that the origin of transient creep is in the cement paste (Gernay, 2012). Moreover, TCS is physically not recovered during cooling. As a consequence, the stress-strain relationships accounting for transient creep strain implicitly, such as the Eurocode-2 model (EN 1992-1-2) considers transient creep strain is recovered during the cooling phase. This consideration can lead to erroneous estimations of the fire-response of cooled concrete elements (Gernay, 2012).

The experiments show that concrete columns are more susceptible to transient creep than flexural members such as beams and slabs. This might be attributed to the fact that, in general, the full cross-section of the columns is compressed and concrete mostly contribute to its bearing capacity, as opposed to flexural members, where the compressed zone is limited. Moreover, columns unevenly exposed to fire develop asymmetrical temperature gradients leading to eccentricities, which increase the transient creep phenomenon (Alogla and Kodur, 2018). These works show that neglecting transient creep might lead to an underestimation of the axial displacement, and consequently, to wrong estimation of fire resistance duration (axial displacements up to 50% larger).

As a conclusion, concrete model should be able to reproduce the effects of transient creep strain and irreversible changes in the mechanical properties of concrete during the fire decay phase.

DEVELOPMENT OF A CONCRETE MODEL FOR STRUCTURES UNDER NATURAL FIRE

An elastic isotropic damage model (Mazars' model) has been developed in this work. Its applicability for the simulation of structures under ISO fire condition has been highlighted in our previous studies (Roosefid et al., 2020 and Roosefid et al., 2021). To extend its applicability to structures under natural fire, it is necessary to develop the concrete constitutive model by taking into account the irreversibility of transient creep strain and the loss of concrete strength during the cooling phase. This section presents the developed model including an explicit term for transient creep strain and the implementation procedure for concrete mechanical properties which are not fully recover during the cooling phase.

Transient creep strain formulation

In order to capture transient creep strain with the numerical model, the addition of an explicit term in the strain decomposition is proposed, according to Gernay et al. (Gernay and Franssen, 2012 and Gernay et al., 2013). Consequently, the concrete macroscopically measurable strains at high temperature can be divided into individual strain components according to Equation 1:

$$\varepsilon_{tot} = \varepsilon_e + \varepsilon_{th} + \varepsilon_{tr} \tag{1}$$

where ε_{tot} is the total strain, ε_e the elastic strain, ε_{th} the free thermal strain and ε_{tr} the transient creep strain.

The transient creep strain rate $\dot{\varepsilon}_{tr}$ is supposed to be proportional to the free thermal strain rate $\dot{\varepsilon}_{th}$ below a temperature of about 500°C, according to Anderberg and Thelandersson's model (Equation 2, Anderberg and Thelandersson, 1976). Above 500°C, $\dot{\varepsilon}_{tr}$ is supposed to be proportional to the temperature variation rate $\dot{T}$ (Equation 3):

$$\dot{\varepsilon}_{tr} = k_2 \frac{\sigma}{f_{c,20}} \dot{\varepsilon}_{th} \ ; \ T \leq 500°C \tag{2}$$

$$\dot{\varepsilon}_{tr} = 0.1 \times 10^{-3} \frac{\sigma}{f_{c,20}} \dot{T} \ ; \ 500°C \leq T \leq 800°C \tag{3}$$

where k_2 is a constant depending on the kind of aggregate and concrete mix, σ the applied stress and $f_{c,20}$ the compressive strength at ambient temperature.

When concrete structures are exposed to high temperatures, its section is subjected to complex stress-temperature paths. For instance a column subjected to a uniform compression, during fire exposure, could even experience decompression with possibly reaching a tension state. For modelling a structural element, the stress evolution across the section during the fire depends on the position in the section which leads to a number of configurations in the stress-temperature space, due to the development of thermal stresses in the section (Gernay and Franssen, 2012). As a consequence, it is necessary to compute the updated values of the stresses and the transient creep strains at each iteration. Also here the transient creep strains are computed in a post-treatment procedure. The value k_2 is chosen based on the work of Roosefid et al (2021).

Cooling procedure

The computation of irreversible mechanical properties during the cooling phase is treated in the present work by implementing a user procedure (PERSO1). This procedure is called by the nonlinear incremental CAST3M solver (PASAPAS) within the framework of a transient weakly coupled thermo-mechanical analysis. The initial temperature is taken as 20 °C at the nodes. At each time-step, the thermal problem is solved first and the temperature field is computed. For each node, the procedure reads the computed temperature in the current and the previous time-step, calculates the temperature increment 'ΔT' and update the currently maximum temperature reached 'Tmax' if 'ΔT' is positive (see Figure 3). At each time-step, the Young's modulus and Mazars' parameters are computed with the field 'Tmax', the material is redefined accordingly and the thermo-mechanical problem is solved.

NUMERICAL SIMULATIONS AND EXPERIMENTAL VALIDATION

Transient creep strain validation at the sample scale

In order to validate the implemented model, the transient tests conducted by Anderberg and Thelandersson (1976) have been simulated. In those transient tests, calcareous concrete cylinders (75 x 150 mm) samples have been subjected to different load levels (10%, 22.5%, 45% and 67.5% of the compressive strength of the sample) and further heated at constant rate of 5°C/min until failure.

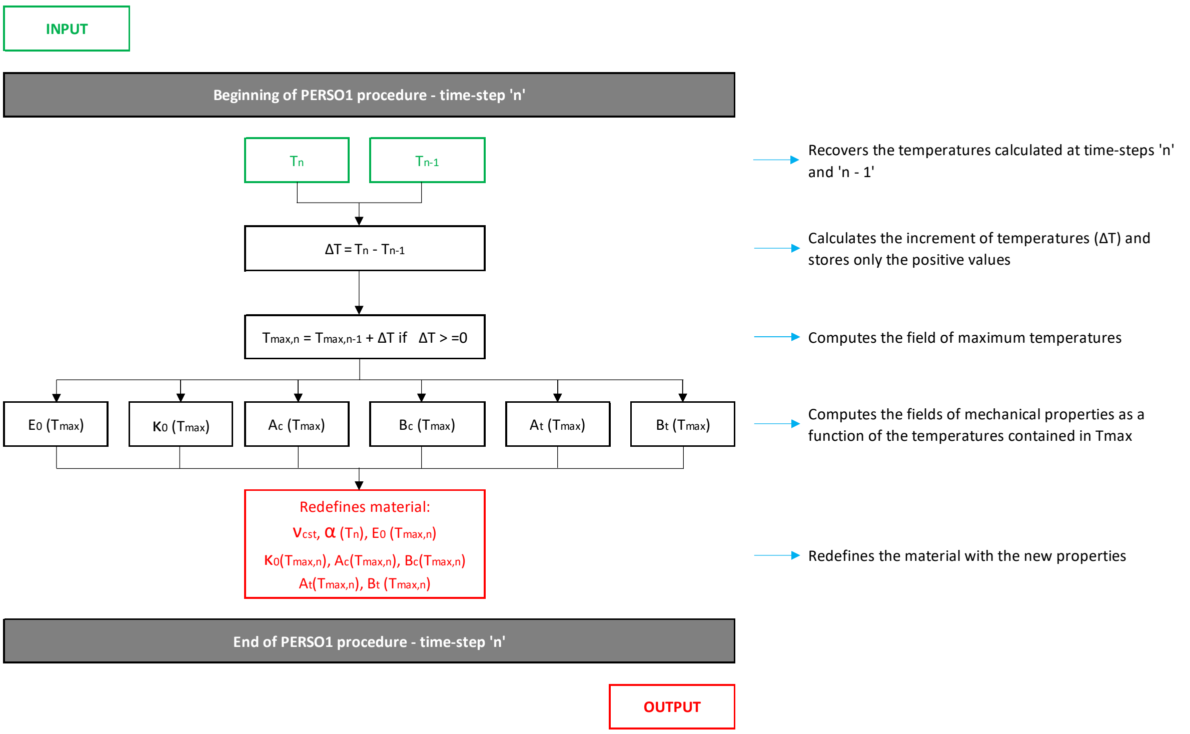


Figure 3. Scheme cooling procedure

The numerical simulation was performed considering siliceous and calcareous concrete specimens according to EN 1992-1-2 (2004), using CAST3M software (Verpeaux et al, 1988). Figure 4 compares the computed results considering siliceous and calcareous concrete for different applied stress levels with transient tests results conducted by Anderberg and Thelandersson (1976). The irregularity observed at 600°C is connected to the reduction factors recommended by EN 1992-1-2 (2004). The development of transient creep strain is thus accurately taken into account by the model.

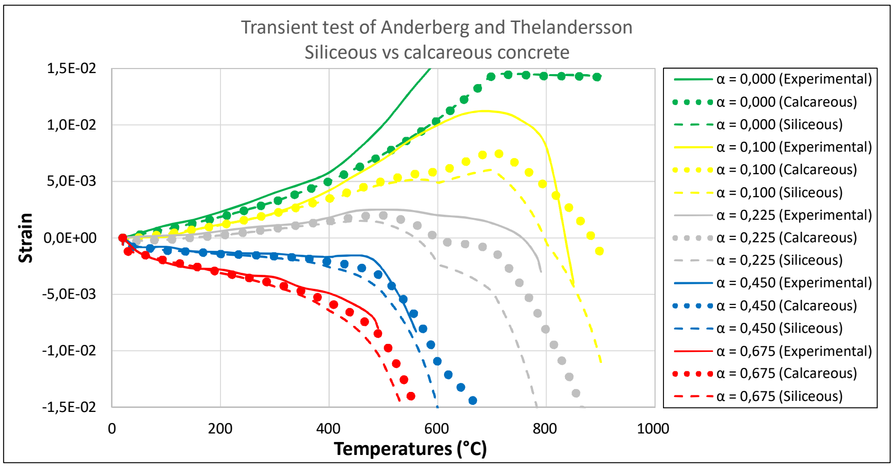


Figure 4. Comparison between experimental and numerical strains for siliceous and calcareous concrete for different applied stress levels; 'α' defined as the ratio between the applied stress and the compressive strength at ambient temperature

Experimental validation at the structural scale

The experimental database considered is the full-scale reinforced concrete columns tests conducted at the University of Edinburgh in the framework of Maclean's thesis (Maclean, 2018). All the tested columns had identical geometry, with dimensions of the cross section 150 mm by 150 mm and overall length of 1400 mm, subjected to sustained mechanical load and exposed to constant heat flux at the compressed and/or tensioned side. The columns were reinforced with four 10 mm high bond bars, one at each corner, and 15 mm thick steel plates were placed at both extremities. Pinned connections were used at the load application point and at the support. Figure 5 illustrates the setup of the experiments.

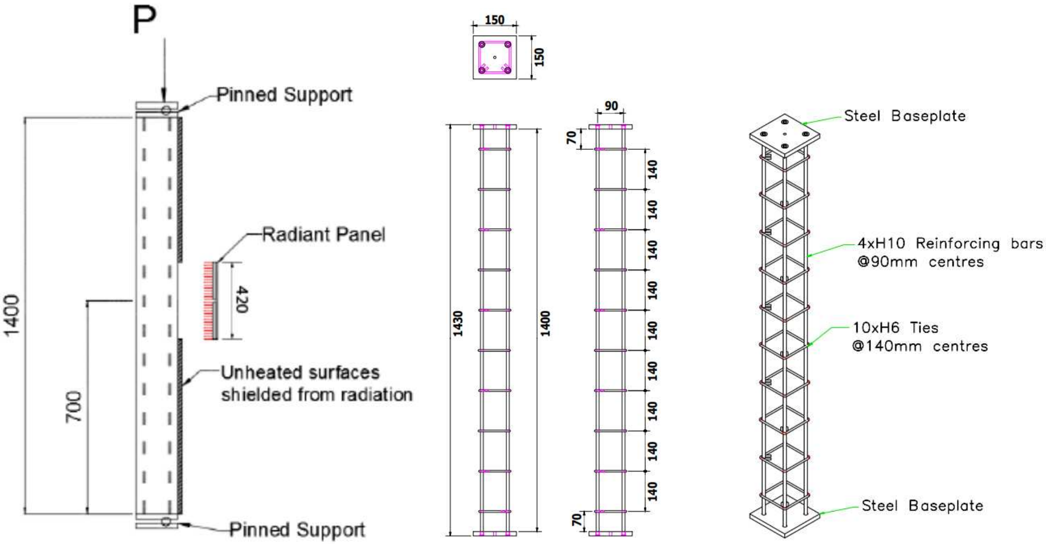


Figure 5. Maclean's experiment: on the left, geometry of a typical column specimen & on the right, details of column specimens (Maclean, 2018)

The heat flux was kept constant for 90 minutes and then removed instantly. During the experimental campaign, several parameters were tested, such as different concrete strength (30 MPa and 50 MPa), different eccentricities (5 mm, 15 mm and 25 mm), different heat flux (50 kW/m² and 70 kW/m² applied either to 1/3, or to 2/3 of the total height of the column) and different mechanical loads (either 10 kN, or 60% of the experimental bearing capacity of the columns at ambient temperature). The temperatures and displacements were measured during both heating and cooling phases. Among all the tests carried out by Maclean, three tests were selected to be simulated in the present work:

- C30-L60-E5-T: Concrete 30 MPa, load 375 kN, eccentricity 5 mm, heat flux 70 kW/m² at the tensioned side and applied to 1/3 of the height of the column;
- C30-L60-E5-C: Concrete 30 MPa, load 375 kN, eccentricity 5 mm, heat flux 70 kW/m² at the compressed side and applied to 1/3 of the height of the column;
- C30-L60-E25-C: Concrete 30 MPa, load 260 kN, eccentricity 25 mm, heat flux 70 kW/m² at the compressed side and applied to 1/3 of the height of the column.

The columns were modelled using 2d finite elements with 4 nodes to limit the complexity of the numerical model. The reinforcement was modelled using either elastic, or elastic perfectly plastic bar elements. A perfect bond between concrete and reinforcement was assumed. Thermal and mechanical properties of concrete and steel reinforcement were assumed as EN 1992-1-2 (2004).

A thermal isotropic model was used to simulate the heat transfer at the columns. A constant heat flux of 70 kW/m² was applied to the mid third of the column. Convective and radiative exchanges between the column and the environment was considered. The convection coefficient was set as 10 W/K/m² at the heat exposed face and 4 W/K/m² at the non-exposed face. An emissivity of 0.7 was assigned to both face. Figure 6 presents a comparison between the temperatures measured experimentally and the temperatures obtained by the numerical model.

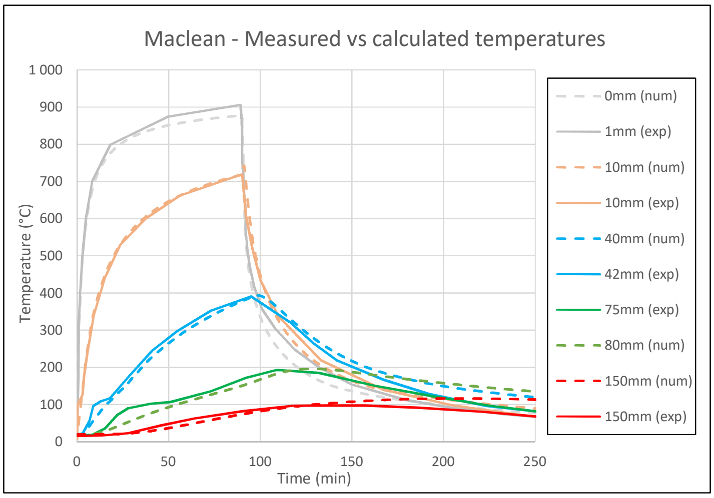


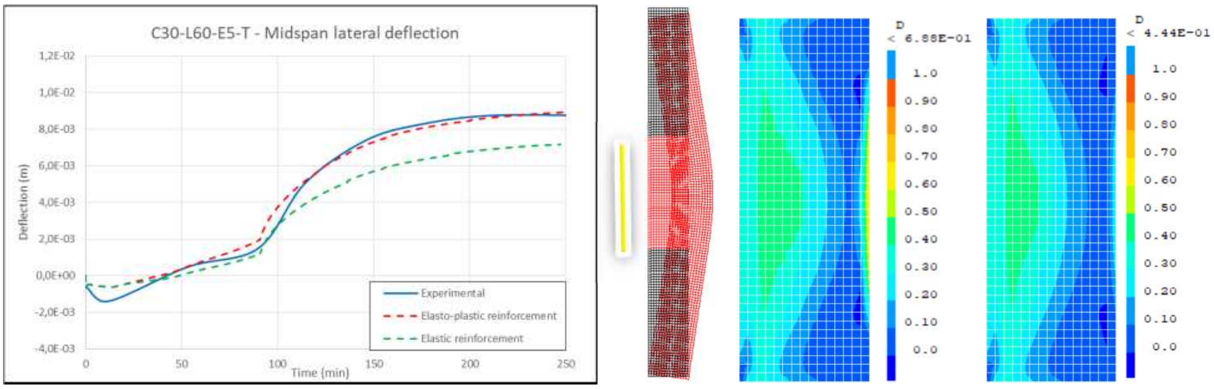
Figure 6. Heat transfer: numerical vs measured temperatures from fire exposed face

For the mechanical behaviour of concrete, the elastic isotropic damage Mazars' model in non-local mode was used. Mazars' model was attributed only to the mid third of the columns, where the heat flux was applied. For the remaining parts of the column, the behaviour was considered as elastic. This simplification does not affect the accuracy of the model, since the damage is limited to this region.

Figure 7 presents the comparison of experimental and numerical results of a tension-heated column with an eccentricity of e = 5 mm considering different behaviours of reinforcement material in the numerical model: elastic and elastic perfectly plastic. The predicted deflections are generally in good agreement with the experimentally measured deflection. However, when an elastic model is attributed to the reinforcements considering a perfect adherence between concrete and reinforcement, less damage is observed and the concrete stresses may be underestimated.

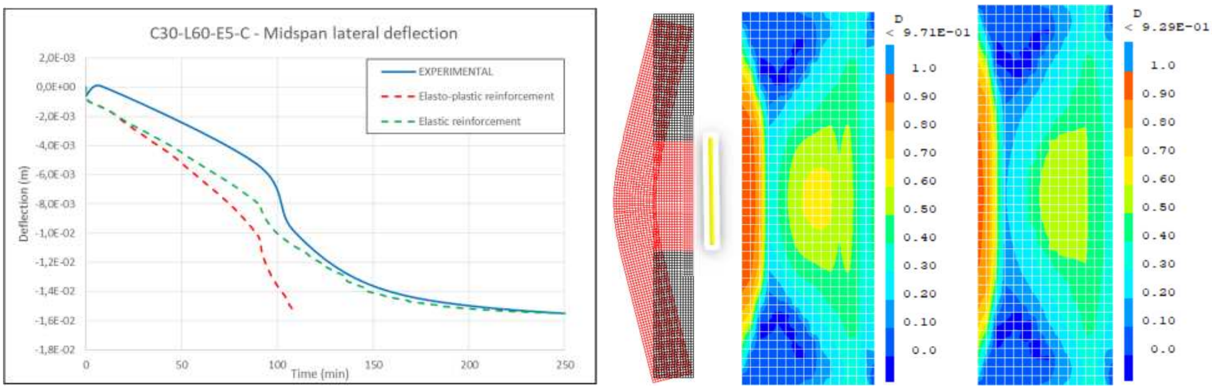
In the case where the heat flux is applied to the compressed side, the structural response tends to be more severe for the concrete. When the reinforcement is modelled using an elastic perfectly plastic model, failure is observed at 109 minutes (see Figure 8). The failure occurs due to a combination of plasticised reinforcement and highly damaged concrete. By considering the reinforcement as elastic, the stabilization of the deflection in the cooling phase is correctly captured by the model.

Compression-heated columns with higher eccentricity (C30-L60-E25-C) behave more severely than compression-heated columns with lower eccentricity (C30-L60-E5-C), as shown in Figure 9. Two tests were carried out and failure was observed in both tests at around 154 and 248 minutes. If an elastic model is used to the reinforcement the compressed zone shows less damage and the column resists. In this configuration, deflections are well predicted during the heating phase and slightly underestimated during the cooling phase. However, if an elasto-plastic model is assigned to the reinforcement, failure is observed during the heating phase, at 78 minutes. Similarly to the columns with lower eccentricity (C30-L60-E5-C), the failure occurs due to a combination of plasticised reinforcement and highly damaged concrete.



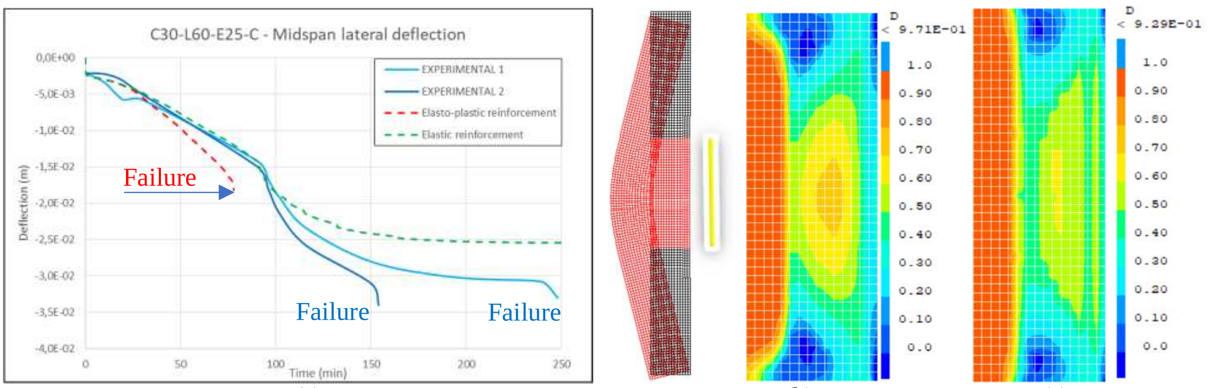
(a) (b) (c)

Figure 7. Comparison between experimental and computed results (a) & damage at the middle third of the column at the end of the test obtained with perfectly plastic reinforcement (b) and elastic reinforcement (c) for C30-L60-E5-T



(a) (b) (c)

Figure 8. Comparison between experimental and computed results (a) & damage at the middle third of the column obtained with perfectly plastic reinforcement at 109 minutes (b) and elastic reinforcement at 250 minutes (c) for C30-L60-E5-C



(a) (b) (c)

Figure 9. Comparison between experimental and computed results (a) & damage at the middle third of the column obtained with perfectly plastic reinforcement at 78 minutes (b) and elastic reinforcement at 250 minutes (c) for C30-L60-E25-C

CONCLUSIONS

This work focused on developing a model able to predict the behaviour of reinforced concrete structures during both heating and cooling phases of a natural fire. The behaviour of concrete material was simulated by means of Mazars' damage model (Mazars, 1984). Adapted procedures were developed to take into account the irreversibly evolution of concrete mechanical properties during the cooling phase and an explicit term for transient creep strain according to (Anderberg and Thelandersson, 1976) model. The evolutions of thermal and mechanical properties of concrete and reinforcing steel were defined following the recommendations of EN 1992-1-2 (2004). The model was implemented with the finite element code CAST3M.

The results obtained by the numerical simulations were in line with the experimental results. Accounting for the damage and the transient creep strains revealed to be critical for accurately predicting the structural behaviour during the heating phase, whereas addressing irreversibly reduction of concrete's properties proved to be the most important ingredient during the cooling phase. The discrepancies observed can be attributed to the limitations inherent to the model in case of perfect adherence between concrete and reinforcing bars. When the nonlinear behaviour of reinforcing bars is neglected, the model tends to overestimate the forces in the rebars. In the other hand, when an elastic perfectly plastic behaviour is attributed to the reinforcement, the numerical model predicts a too early collapse of the structure.

In conclusion, the model showed good potential in predicting the fire behaviour of reinforced concrete structures. Other structural types of elements should be modelled, such as beams and slabs, in order to evaluate in a broader sense its applicability.

REFERENCES

Alogla S. and Kodur V. R. (2018). "Quantifying transient creep effects on fire response of reinforced concrete columns," Engineering Structures, 174(132), pp. 885-895.

Anderberg, Y. and Thelandersson, S. (1976). "Stress and Deformation Characteristics of Concrete at High Temperatures: 2 Experimental Investigation and Material Behavior Model," Bulletin 54, Lund Institute of Technology, Sweden.

Botte W. and Caspeele, R. (2017). "Post-cooling properties of concrete exposed to fire," Fire Safety Journal, Vol. 92, pp. 142-150.

EN 1992-1-2 (2004). "Eurocode 2: Design of Concrete Structures, Part 1-2: General Rules. Structural fire design," CEN, Brussels.

EN 1994-1-2 (2005). "Eurocode 4: Design of composite steel and concrete structures, Part 1-2: General rules. Structural fire design," CEN, Brussels.

Gernay, T. and Franssen, J. M. (2012). "A formulation of the Eurocode 2 concrete model at elevated temperature that includes an explicit term for transient creep", Fire Safety Journal, Vol. 51, pp. 1-9.

Gernay, T. (2012). "Effect of transient creep strain model on the behavior of concrete columns subjected to heating and cooling". Fire technology, 48(2), pp. 313-329.

Gernay, T., Millard, A., and Franssen, J. M. (2013). "A multiaxial constitutive model for concrete in the fire situation - Theoretical formulation". International Journal of Solids and Structures, Vol. 50 (22-23), pp. 3659-3673.

Maclean, J. (2018). "The structural response of reinforced concrete columns during and after exposure to non-uniform heating and cooling regimes", Doctoral dissertation. The University of Edinburgh, Scotland.

Mazars, J. (1984). “Application de la mécanique de l’endommagement au comportement non linéaire et la rupture du béton de structure”, Thèse de Doctorat d’État, L.M.T., Université de Paris, France.

Roosefid M., Bonhomme M. H., Pimienta P., (2020). “Modeling the structural behavior of reinforced concrete walls under ISO fire exposure”, in *11th International Conference on Structures in Fire*, p. 262-270.

Roosefid M., Bonhomme M. H., Pimienta P., (2021).” Structural performance of reinforced concrete walls under ISO fire exposure”, in *Applications of Structural Fire Engineering*, p. 98-103.

Verpeaux P., Charras T. and Millard A., (1988). CASTEM 2000 : une approche moderne du calcul des structures, Calcul des Structures et Intelligence Artificielle, ed. par Fouet J-M., Ladevèze P. et Ohayon R, Pluralis, p. 261–271.