Fire behaviour of modular reinforced concrete buildings – numerical simulation of the thermomechanical response

E. Castilho[1], J.P. Firmo[2]

[1]*Instituto Superior Técnico, University of Lisbon, Portugal, eloisa.castilho@tecnico.ulisboa.pt*

[2]*Instituto Superior Técnico, University of Lisbon, Portugal, joao.firmo@tecnico.ulisboa.pt*

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Abstract
This paper presents a numerical study on the fire behaviour of prefabricated modular reinforced concrete buildings, developed within the framework of the (on-going) R2UTechnologies project. The main objectives of this work are: (i) to verify the fire resistance of the structural elements with respect to the (Portuguese) legal framework; and (ii) to use insights from numerical analyses to design experiments targeting critical aspects. Thermal and mechanical sequentially coupled analyses were performed, using the commercial software ABAQUS. It was found out that the critical scenarios are (i) the post-fire residual behaviour of the connections between walls and (ii) the possibility of spalling during fire exposure. Fire resistance tests, under preparation, will validate the numerical models presented in this paper, and study the above-mentioned phenomena.

1 Introduction

Modular construction represents a significant evolution with respect to traditional *in-situ* building practices. Modules (*i.e.* parts of the structure) are produced in controlled industrial environments, often already integrating key infrastructure systems such as electrical wiring, plumbing, and sewage installations. This prefabrication process significantly reduces construction time, making speed one of the primary advantages of modular construction (compared to conventional techniques). As so, over the past decade, modular prefabricated solutions for residential buildings have seen significant advancements, also driven by challenges such as labour shortages and the rising costs of traditional on-site construction. The industrial production of structural elements facilitates the adoption of optimized geometries, enabling more efficient material usage. While this optimization often results in lighter and slenderer structural members, which is advantageous in most scenarios, it may also increase vulnerabilities under fire conditions. The fire performance of the connections between these structural members is particularly critical, as it profoundly influences the overall fire resistance of prefabricated reinforced concrete buildings. Furthermore, these connections often determine the extent of post-fire repairs required, not only to maintain structural integrity, but also to ensure reliable performance under potential future seismic events.

This paper presents a numerical study conducted for the evaluation of the fire vulnerability of a modular reinforced concrete (RC) solution, currently under development, as part of an ongoing project (see Section 2). The temperature field resulting from the application of a standard fire curve (ISO 834) was determined through a transient thermal analysis. Subsequently, a mechanical analysis was performed taking the results from the previous thermal analysis as input. The regulatory requirements were assessed, and the critical points that require further experimental investigation were identified.

2 Project description

The "R2UTechnologies | Modular System" project aims to develop and industrialise a modular construction concept that addresses contemporary challenges with a focus on sustainability. Its implementation involves an extensive network of partners and beneficiary entities. Specifically, this paper refers to ongoing activities at "Instituto Superior Técnico" of the University of Lisbon, with the main objectives being: (i) evaluating the fire safety of the structural elements within the modular RC housing system, in compliance with the (Portuguese) legal framework (RJ-SCIE, Decree-Law No. 220/2008, as amended by Decree-Law No. 224/2015), based on 'REI' criteria (load-bearing capacity, R; integrity, E; and thermal insulation, I); and (ii) proposing protection systems and/or geometric or material modifications to the elements when more demanding requirements need to be met. These modular buildings are intended for residential use (senior housing, student residences, single-family, or multi-family homes), classified as 'Type I' use, according to the above-mentioned documents. The typology is customisable by assembling different combinations of modules, accommodating up to eight floors (ground floor plus seven stories), which, in the most unfavourable case, corresponds to a '3rd risk category'. In accordance with regulatory provisions (RT-SCIE), the load-bearing and compartmentalising elements must therefore have a minimum fire resistance of 'REI90' under standard fire conditions, covering the requirements for stability (R), integrity (E), and insulation (I). To achieve this: (i) the 'R' criterion is evaluated through mechanical analysis; (ii) the 'E' criterion must be ensured using fire-resistant doors and windows, as well as fire-rated service installations; and (iii) the 'I' criterion is assessed based on the temperature increase on the unexposed surface, following EC2, Part 1-2 (2010).

3 Concrete fire resistance

3.1 General aspects

The challenges faced by structural and supporting elements under fire conditions are primarily associated with stability (resistance, *i.e.*, the preservation of the load-bearing capacity), integrity (referring to the presence of holes, cracks, or gaps that may allow the passage of smoke and/or flames) and insulation (which involves controlling the temperature rise). As detailed on the previous section, the specific criteria to be met depend on the construction typology. The fire resistance rating of a building, typically ranging from 15 to 360 minutes, is defined by the time it takes for one of these criteria to fail, during a standard fire test.

The fire action is a design accidental situation in which, in addition to the permanent and variable ("mechanical") actions, the fire thermal load is considered. During a fire event there is energy release in the form of heat, and the three processes of heat transfer are conduction, convection and radiation [1].

The performance and temperature field in each structural element during a fire is inherently dependent on the characteristics of the thermal action. The use of natural fire models allows the load-bearing capacity of the structural components to be designed for the entire duration of the fire, including the cooling phase. Due to thermal inertia, concrete components experience delayed heating during the fire's cooling phase, which can lead to the development of tensile stresses. Furthermore, as stated in [2], after reaching the peak temperature followed by a cooling phase, the strength of concrete falls below the value specified at the maximum temperature in EC2-Part1.2. Thus, considering natural fire scenarios would guarantee a more realistic analysis [3]. Lennon [4] provides guidance on the choice of the parameters that impact on post-flashover fire behaviour. However, due to the complexity of simulating such scenarios, the application of standardised fire models (most excluding the cooling phase) remains a common practice in fire engineering. Indeed, according to [4], the results from standard fire resistance tests are generally a conservative approach. In this context, the ISO 834 curve is a widely used standardised reference, and was employed in this study. The structural response, in terms of temperature field, depends on the thermophysical properties of the materials, initial and boundary conditions, and the geometry of the element.

3.2 Numerical and experimental aspects

When analysing structures exposed to fire, it is essential to account for the evolution of the thermo-physical and mechanical properties of materials as a function of temperature, given the wide range of temperatures they may experience. However, the scarcity of literature on the variation of thermophysical properties of insulation materials with temperature is noteworthy, with only limited data available. For instance, Kontoleon [5] simulated the evolution of thermophysical properties for an EPS system until 300 ºC, while Duarte et al. [6] investigated the behaviour of PUR and PET foams up to 650 ºC. Furthermore, the assumed emissivity values may significantly impact the results, as highlighted in [7]. Given the above, experimental validation of the numerical models is critically important.

It is also worth to mention that, when using combustible insulation core materials (*e.g.*, EPS), the dynamics of the transient thermal problem vary according to the reached temperature on these elements. Beyond decomposition temperature, processes such as liquefaction, vaporization, and thermal decomposition of the combustible thermal insulation layer become evident. Therefore, the heat transfer problem evolves from: (i) conduction through combustible solid sections (the thermal insulation layers) until their complete decomposition (melting and gasification), generating an air cavity, therefore to (ii) free convection and longwave radiation among both concrete wythes in contact with the core insulation [5].

Additionally, due to the accidental nature of fire, the system does not achieve a steady-state condition. Consequently, the solution to the heat conduction equation always depends on the initial temperature, assigned to each node of the model.

The radiation emitted by the fire (shortwave), is a prescribed flux categorised as a *Neumann* boundary condition. This flux is subsequently re-emitted by the surfaces heated by the fire in the form of longwave radiation, also referred to as re-radiation [8] , which is a flux dependent on the difference of the surface and ambient temperatures. As in structural engineering practice the fire action is commonly defined by a temperature-time curve (*e.g.*, the ISO 834 standard), thus the thermal boundary conditions (BC) are typically simplified to the *Robin* condition, as described by Lucherini [9]. Further considerations on this topic are explored in [10]. This BC is defined according to Eq. (1):

$$\textit{Robin}\text{ condition (3}^{\text{rd}}\text{ type BC):}\qquad -k\frac{\partial T}{\partial n} = q_c + q_r = q \qquad \text{in } \Gamma_{cr} \tag{1}$$

in which k is the thermal conductivity of the material [W/(m K)], n is the normal vector to the surface, and Γ_{cr} represents the surface exposed to heat exchange, specifically: (i) by convection, Eq. (2), with the flux (q_c) given by Newton's law, where h_c is the convection coefficient in [W/(m²·K)] and T_a is the air temperature; and (ii) by longwave thermal radiation (q_r), Eq. (3), given by Stefan-Boltzmann's law, where ε is the surface emissivity and σ is the Stefan-Boltzmann constant.

$$q_c = h_c\,(T - T_a) \tag{2}$$

$$q_r = \varepsilon\sigma(T^4 - T_a^4),\ \sigma = 5.669\times10^{-8}\ \text{W/ (m}^2\ \text{K}^4) \tag{3}$$

Under fire exposure, the structure experiences a complex thermal response. Temperature gradients develop, potentially resulting in significant thermal stresses, depending on the restrain conditions [11] (mechanical boundary conditions). To numerically simulate this phenomenon, the most common coupling between thermal and mechanical responses is sequential coupling. In this method, the temperature distribution influences the mechanical response, but the reverse is not true – for instance, phenomena such as cracking are not considered to affect thermal conductivity. Given this limitation, it is essential to experimentally evaluate the significance of the singularities that are not accounted for in the numerical analysis.

Also, it is important to highlight that certain fire-related phenomena are still difficult to simulate numerically, and, therefore, require experimental investigation. One such phenomenon is spalling [12], which refers to the delamination of the concrete cover of structural elements due to exposure to high temperatures, and, especially, increasing with high heating rates. Spalling causes a loss of the effective load-bearing cross-section and exposes the reinforcement to the fire, which can compromise the integrity of the rebars. This phenomenon results from a complex interaction of factors, such as the type and age of the concrete, permeability, heating rate, aggregate type and size, moisture content, presence of cracks, fibres, applied load, etc. (fib Bulletin 38). The underlying mechanism, however, is the increase in pore pressure due to water vapor within the concrete, making it particularly critical in high density concrete. To mitigate the effects of spalling, the application of additional reinforcement can be considered. To prevent explosive spalling, non-metallic polymer fibres can be used (*e.g.* polypropylene fibres [13]), promoting the formation of micro-channels that allow water vapour to escape. Due to the complexity of this phenomenon, its numerical prediction is still theoretically unfeasible, making experimental verification essential.

In walls composed of different layers of concrete and/or other materials, exposure to high temperatures can lead to the separation of these layers, compromising the residual load-bearing capacity of the wall. This phenomenon is exacerbated by the differing thermal expansion of the materials, and facilitated by the changes in the pressure of evaporated water. Such conditions can lead to the formation of microcracks, potentially causing layer detachment. To characterise the integrity of the interface, and the residual strength of the system, a detailed experimental evaluation is required.

Considering the above, a subsequent experimental campaign (after the numerical analysis) is mandatory, with the following objectives: (i) validate the numerical models (with respect to the used mechanical and thermophysical properties assumed for the materials, as well as the boundary condition parameters); and (ii) evaluate the importance of phenomena that cannot be determined numerically. However, experimental campaigns are costly (both in time and financial terms). Thus, numerical thermal analysis is a useful tool for identifying critical scenarios where testing is mandatory. These tests should be performed on the most safety-challenged typologies, identified not only by the temperature of the rebars embedded in the concrete but also by the temperature gradient near the exposed face.

Also in this context, it is important to emphasize the significance of full-scale tests, as they offer unparalleled insights into the structural behaviour under fire conditions. These tests are relevant for assessing failure mechanisms, such as connection failure. Additionally, they address potential discrepancies between standardised fire exposure and the thermal conditions experienced during real fires. Moreover, full-scale tests remain the most reliable approach for evaluating the spalling behaviour of concrete structures, provided that the tested element is representative of the actual structure [12]. In this situation, when comparing displacement measurements from experimental tests with numerical model results, it is necessary to include transient creep strain in the constitutive relationships for concrete at elevated temperatures [12], [14]. Accurate prediction of these strains is necessary to estimate progressive collapse (more relevant in steel framed buildings [15]). However, due to the relatively short duration of fire tests, classical material creep ε_{creep} is generally neglected [2].

Finally, after a fire, it is crucial to determine whether the structure requires repairs before it can be safely reused. Post-fire assessment of reinforced concrete structures involves evaluating whether to retain, repair or demolish the structure, particularly following moderate fires. The structure must be assessed for its safety under the fundamental regulatory ultimate limit state (ULS) and serviceability (SLS) load combinations, as well as for potential future seismic actions. To determine the need for repairs, it is necessary to consider the residual properties of the concrete expected at room temperature (in accordance with EC4 – part 1.2), after exposure to the temperature profiles attained during fire, and evaluate its structural integrity for subsequent use. It is worth noticing that the time elapsed after exposure to high temperatures significantly influences the residual strength [2]. For preliminary conclusions, a damage classification based on the temperature distribution of concrete members was proposed in [16].

4 Numerical modelling

4.1 Thermal analysis

The developed thermal analyses focused on two cases: (i) a wall section in a standard area (away from connections) and (ii) a connection zone (one of the possible connection systems under development in this project). The typical wall section features a sandwich configuration with insulation placed inside the walls (Fig. 1a), which is ideal for the use of EPS (one of the most cost-effective options). Due to the flammability of EPS, its use on the exterior would require the application of an additional non-combustible coating for insulation (*e.g.* gypsum-based; fibre-reinforced cementitious mortar, etc). In the connection zone, the walls consist of two layers: 5 cm of exterior UHDC (*Ultra-High Durability Concrete*) and 15 cm of interior LCLWAC (*Low Cement Lightweight Aggregate Concrete*). The connection between adjacent walls is composed of metal components bolted together, to which four metal rebars are welded. These rebars are embedded in the walls, transmitting forces through adhesion to the concrete (see Fig. 1b). The thermophysical properties used in this analysis are described in [17].

Regarding the applied action, the most common scenario was simulated (fire in only one room): therefore, one face was exposed to ambient temperature (T_{air} = 20 ºC) and the other to the ISO 834 curve. Since the incident flux on the exposed faces depends on the temperature difference at the boundary, *Robin* BC were considered, accounting for heat exchange through both long wave radiation and convection.

Due to the section's geometry and the uniform nature of the considered thermal action, heat transfer predominantly occurs in the direction normal to the faces, making a one-dimensional analysis theoretically sufficient. However, given the relatively low computational cost of the thermal analysis (with only one unknown per node), a two-dimensional (2D) model was adopted for the standard area. This approach allows for potential future extensions of the analysis to more complex and irregular scenarios. Fig. 1a shows the adopted finite element mesh: discretisation with second-order quadrilateral elements (8 nodes) and 10 equally spaced elements across the thickness (a total of 2,735 nodes and 800 elements). The lateral faces of the model were considered adiabatic boundaries (no heat flux across the boundary, *i.e.*, $q = 0$), thus acting as axes of symmetry for the wall's temperature field (as represented in Fig. 1a). Consequently, the simulated wall can be (thermally) considered infinitely long.

In the connection zone, given the dimension of the reinforcing rebars, it was considered that they could influence the temperature distribution in the wall, so they were included in the model. For this reason, a 3D analysis was conducted, despite the heat flow being predominantly one-dimensional (*i.e.* through the thickness). Fig. 1b and Fig. 1c illustrate the connection zone represented in the 3D model. Each wall module is 2.4 m long and 3.0 m high. The analysed wall segment measures 500 mm in height and 900 mm in length. This dimension is sufficient to ensure that the concrete surrounding the connection is modelled up to a region where the influence of the reinforcing bars on the heat transfer process (and consequently, on the obtained temperature distribution) becomes negligible. The model utilised hexahedral elements with 8 nodes (first-order), combined with linear elements for the reinforcement bars. The rebars were modelled as embedded constraints to ensure interaction compatibility between the reinforcement and the concrete elements. Contact conditions were applied at all material interfaces, and the steel plate within the connection, located in the outer tooth of the wall, was considered thermally insulated (ceramic fibre blankets will be installed over the steel connection). A discretisation of 20 equally spaced elements across the thickness was adopted, resulting in a mesh with 102,302 nodes and 89,725 elements. The lateral faces of the model were considered adiabatic boundaries, effectively allowing the wall to be (thermally) considered as infinitely long beyond these faces. The top and bottom faces were also considered adiabatic, since the simulated wall section is sufficiently distant from the floors to be considered unaffected by this boundary condition.

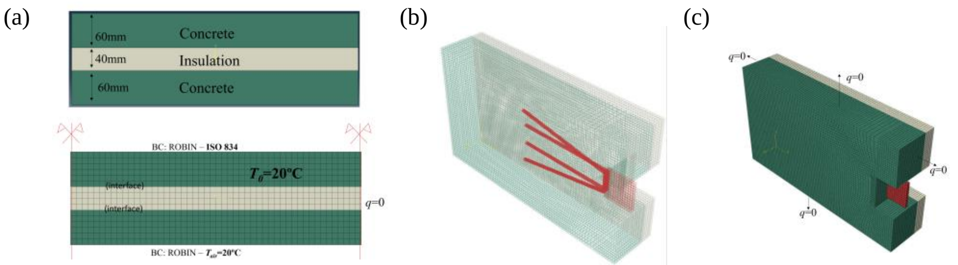

Fig. 1. Finite Element model and boundary conditions of: (a) standard area; and connection zone (b) interior; (c) exterior.

Both models were developed using ABAQUS Standard software. A transient analysis was conducted starting from the initial temperature of the materials ($T_0 = T_{air} = 20$ ºC) and extended up to 6 hours of exposure to the standard ISO 834 curve. An implicit (incremental and iterative) solution was employed for time integration, with a calculation step ranging from $1E^{-3}$ s to 10 s, allowing a maximum temperature variation of 10 ºC per increment.

Fig. 2a and Fig. 2b depict the temperature field of the studied standard wall, indicating that 'Criterion I' (average temperature increase at the unexposed surface <140 ºC / local increase <180 ºC) is clearly satisfied up to 90 minutes of fire exposure. After 90 mins of exposure, the temperature within the EPS core (insulation) ranges between 190 ºC and 440 ºC. EPS decomposes for temperatures between 350 ºC and 500 ºC [18]. Therefore, modifications to the thermal model would be required (to simulate the air cavity after decomposition) if criterion I was not met with such a significant margin.

Fig. 3a and Fig. 3b present the temperature field of the studied connection zone. By analysing the thermal gradient close to the exposed surface (depth <25mm), it is possible to assess the risk of spalling. Maluk [19] concluded that the thermal gradient triggering this phenomenon (in ordinary concrete considered in that study) is approximately ~11 ºC/mm. Since the obtained gradient in the present study is high (~20 ºC/mm), it is considered that the risk of spalling during fire exposure is significant. Additionally, the thickness of the wall that is subjected to temperatures above 300 ºC is approximately 40 mm, *i.e.*, greater than the concrete cover thickness (25 mm). Therefore, this temperature profile suggests the presence of non-negligible damage in the concrete (classified as "damage state 3" in [16]), which may require significant post-fire repair. Evidently, thermal analysis does not explicitly allow for assessing the reduction in load-bearing capacity. However, it highlights phenomena worthy of further studies. In particular, it is recommended experimental verification of spalling, as well as to assess the post-fire residual capacity of the wall.

Indeed, a key advantage of the coupling method employed between analyses (sequential) is its ability to optimise computational resources by enabling more targeted mechanical analyses, as from the thermal analysis it is possible to identify the conditions/scenarios/structural elements that are most at risk. As a result, the subsequent (and more computationally demanding) mechanical analysis can be focused on these critical scenarios, thereby leading to a more efficient overall assessment process. Therefore, regarding the load-bearing capacity of the connections: during the fire, this is anticipated not to be a critical aspect, as the connections mechanical demand (*i.e.* the load level) is practically negligible for this accidental situation. Furthermore, it is observed that after 90 minutes of exposure, the maximum temperature in the rebars embedded in the concrete is only 112 ºC (as shown in Fig. 3c), meaning that the mechanical properties of the steel remain unchanged. Given these considerations, the subsequent mechanical analysis will focus solely on the load-bearing capacity of the concrete wall itself, rather than the connections. Consequently, it is unnecessary to discretise the rebars as solid elements (which would be required if evaluating their interface behaviour). Instead, modelling the rebars as embedded elements, as done in the thermal analysis, was considered sufficient.

In the experimental studies to be conducted as part of this project, the focus will be on the residual (post-fire) capacity of these connections, particularly to assess the need for post-fire repair to ensure adequate seismic performance.

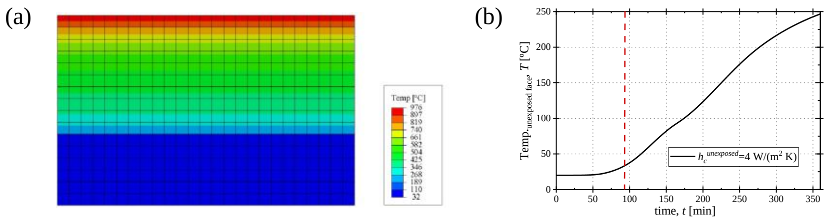

Fig. 2. Standard wall: (a) 2D FE model, thermal field after 90 minutes of exposure; and (b) temperature evolution at the unexposed face.

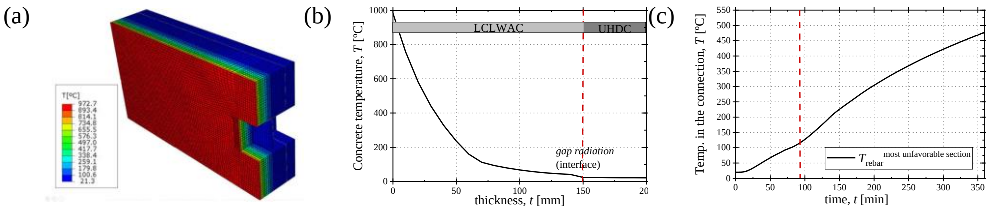

Fig. 3. Connection zone: (a) 3D FE model, thermal field after 90 mins of exposure and (b) across the thickness; (c) temperature evolution in the embedded rebar (shown in Fig. 1b).

The verification and validation of the model are both complementary and fundamental processes in the application of numerical methods, particularly the Finite Element Method (FEM). These processes evaluate the accuracy of a FEM model in simulating a specific physical scenario [20]. Code verification assesses the precision of the numerical algorithms used to approximate the solution of differential equations. This step also identifies errors arising from inadequate mesh refinement (evidenced by the model's sensitivity to the adopted discretization and its suitability for the phenomenon under study). The verification of the models developed in the present study was achieved by comparing the results with the ones obtained using another FEM program (PAT [21]). Subsequently, it is desirable to validate the material properties considered in the numerical model for the specific scenario being simulated – namely, to assess whether if the phase change of the combustible insulation, which was not accounted for in the present numerical model (due to the safety margin with respect to the 'I' criteria), would have a significant impact. This requires *in situ* observations to validate, or iteratively calibrate, the thermo-physical parameters used in the model. The experimental validation phase will be conducted at a later stage of this project. Nevertheless, it is important to bear in mind that at the early stages of fire exposure, achieving consistency between the experimental and numerical temperature distribution is more challenging (as explained in [22]). This difficulty arises not only because the results are still in the most dependent phase on the initial conditions, but especially due to phenomena such as steam and moisture migration, crack formation, spalling, and the detachment of surface layers. These considerations should be accounted when validating the numerical model by comparing results.

4.2 Mechanical analysis

Following the thermal analysis, a sequentially coupled mechanical analysis was conducted to assess the load-bearing capacity of the wall and its connections under fire conditions. Using the temperature distributions obtained from the thermal analysis, the mechanical model was developed to focus on the connection zone.

Before addressing the mechanical BCs, it should be noted that, following the thermal model, only a fraction of the wall was considered. This simplification hinders the representation of the actual mechanical BCs that the wall would experience when integrated into the real structure. Therefore, the simulated BCs are a simplification, as is typical in preliminary analyses, but will allow the critical scenarios to be identified; this model is intended to be subsequently improved, namely considering the full height of the building, to accurately simulate the curvature induced by the temperature gradient.

Regarding the simulated BCs, the wall was assumed to be vertically fixed at the base (Y translation restricted at each node of base), while a symmetry axis was considered on its length (X translation restricted at each node of the left side, as shown in Fig. 4a. To prevent rigid body motion, the Z translation was also restrained (at a single node, at the corner of the base, allowing for Poisson's expansion). No horizontal (X direction) restraint was considered on the side corresponding to the connection (right side), as the joint is "dry" (*i.e.* not involving any *in situ* mortar pouring), therefore the nodes between adjacent walls will not be in contact due to the clearance in the joint. As previously mentioned, the restriction caused by the walls above was not included in the model. In reality, however, these walls not only restrict the expansion of the simulated portion, but also constrain its curvature, resulting in a deformed shape that differs from that of the (simulated) free-end condition. These simplified BCs provide significant structural freedom to the model, resulting in minimal expected stresses from the thermal gradient. Although this simplification, the model allows for the simulation of internal thermal stresses generated by the restraints resulting from the different expansions due to the varying coefficients of thermal expansion of the materials (steel rebars and concrete layers) across the thickness of the wall.

Ties constraints were applied between the different parts of the model, to guarantee displacement continuity at the nodes from adjacent parts. In this way, it is possible to consider distinct mesh refinements between concrete and steel, as represented in Fig. 4b.

(a) (b)

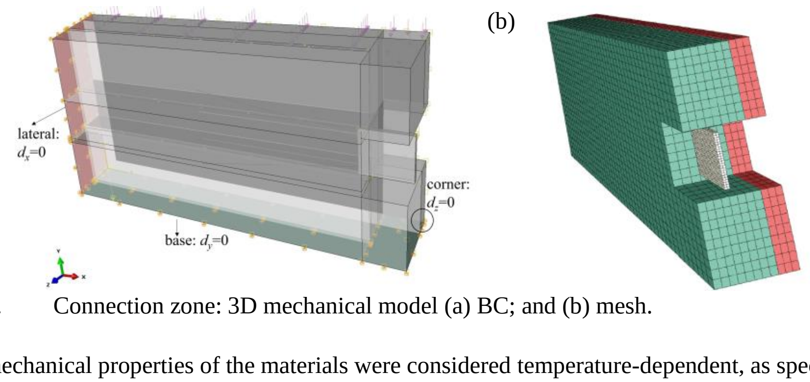

Fig. 4. Connection zone: 3D mechanical model (a) BC; and (b) mesh.

The mechanical properties of the materials were considered temperature-dependent, as specified in the relevant regulations (EC2, Part 1-2, and EC3, Part 1-2).

In terms of load combinations used for the definition of the load level applied to the wall, the accidental design situation, as defined in EC0 (2011), was adopted, corresponding to the quasi-permanent combination for variable actions. This resulted in a vertical load applied to the model of approximately 10 kN/m² (per floor), introduced in the first step of the analysis. Subsequently, the previously determined temperature distribution was used as input for the mechanical model. However, due to the high degree of refinement applied in the thermal model, the corresponding mechanical model would be computationally demanding (now involving three unknowns per node), resulting in prolonged analysis times. To reduce the computational costs, a less refined mesh (with only 11 elements through thickness, as shown in Fig. 4b) was adopted; still, it was confirmed that the differences in terms of temperature distributions when compared to the more refined mesh described in section 4.1 were not significant (as observed in Fig. 5). This approach is feasible because ABAQUS manages the interpolation of thermal results in the mechanical analysis when the meshes are not coincident.

Regarding tensile stresses in the reinforcement of the connection after 90 minutes of fire exposure (mostly induced by the thermal gradient), a maximum value of only 22 MPa was obtained (see Fig. 6). This stress level remains significantly below the material's strength at the corresponding rebar temperature (around 100 °C, as discussed in section 4.1).

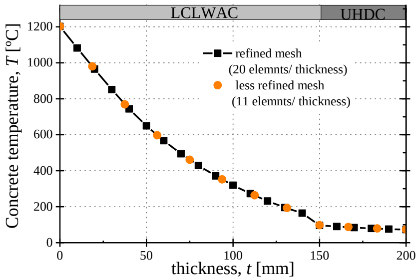

Fig. 5. Temperatures through the thickness after 360 mins of exposure to ISO834 (mesh sensitivity analysis)

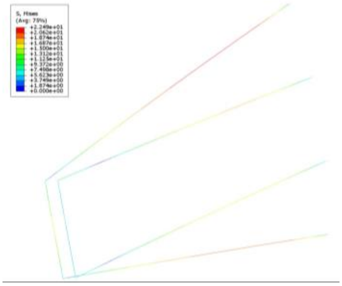

Fig. 6. Stress distribution in the steel rebars after 90 minutes of exposure to the ISO 834 fire curve.

Regarding the concrete, evaluating its load-bearing capacity involves verifying whether, on average (through thickness), the material's remaining strength at each temperature exceeds the applied stress. It was observed that, in highly localised areas, the concrete strength at certain temperatures is exceeded (see Fig. 7). However, these occurrences are limited, and do not affect the overall load-bearing capacity of the wall, as these stresses will be redistributed to the stiffer (*i.e.* less damaged) layers of the wall. In general, the compressive stress remains below 20 MPa, for the first 90 minutes of fire exposure, which is always below the remaining strength of the material for that temperature. A representative wall thickness profile at the base is shown in Fig. 8. This figure illustrates the temperature gradient across the thickness of the wall after 90 minutes of exposure to the ISO 834 fire curve, together with the corresponding remaining concrete strength through the thickness of the wall. It was observed that near the exposed surface, where the concrete is significantly damaged, it no longer contributes to the load-bearing capacity. Additionally, this figure also shows that the compressive stress (positive in the direction indicated on the graph) remains significantly below the material's strength. The load-bearing capacity is therefore confirmed with a substantial safety margin.

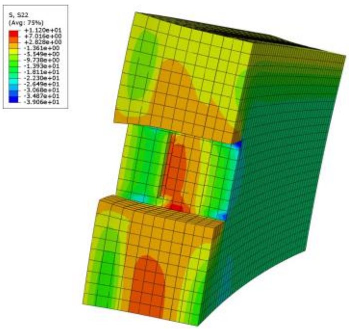

Fig. 7. Stress distribution in the concrete wall after 90 minutes of exposure to ISO 834 fire curve

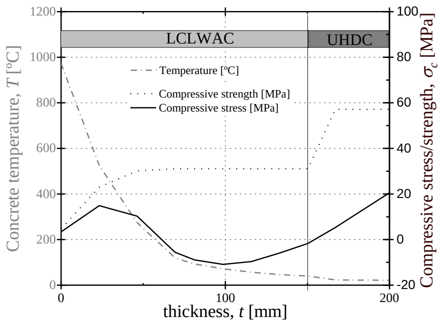

Fig. 8. Thickness: stress and temperature field after 90 mins exposure to ISO834

Based on the results outlined above, it can be concluded that the load capacity of the wall has been verified and is clearly ensured for up to 90 minutes of fire exposure. However, the residual (post-fire) capacity still needs to be assessed, as this is expected to represent the most critical scenario. The verification of residual properties for other ultimate limit states (persistent and transient situations, as well as seismic situations) is crucial, as the load-bearing capacity was only assessed for the accidental scenario (considering the relevant fire load combination). Regarding the connections, experimental testing is required to account for potential loss of bond between concrete and the embedded steel rebars due to concrete cracking and/or spalling that may occur during fire exposure.

5 Conclusions

This study presented a numerical analysis to evaluate the fire vulnerability of a modular reinforced concrete solution. The thermal analysis confirmed that the walls under development satisfy Criterion I for a duration of 90 minutes. Although Criterion R cannot be explicitly verified using thermal models alone, the thermal results strongly suggested that the load-bearing capacity would be met. Therefore, a straightforward thermal analysis proved to be a valuable tool, as it highlighted critical scenarios and phenomena that require experimental investigation, thereby enabling mechanical analysis and experimental efforts to focus on the most potentially problematic aspects.

A sequentially coupled analysis was conducted (*i.e.*, thermal followed by mechanical), and the results demonstrated that: (i) the acting stresses in the reinforcement rebars were negligible (which was even less problematic given that they were exposed to temperatures that did not compromise their mechanical properties); (ii) furthermore, the load-bearing capacity of the wall for 90 min of fire exposure was verified with a substantial safety margin, affirming the structural reliability of the modular solution under fire conditions.

From the FEA presented here, it was found out that the critical scenarios requiring detailed experimental verification are (i) the residual behaviour of the connections and (ii) the assessment of the possibility of spalling, ensuring a more robust understanding of the fire performance of the modular RC solution under development. These fire resistance tests will also allow for the validation of the numerical models presented in this paper. Once the numerical models have been validated, it will be possible to conduct parametric studies deemed relevant to identify the most efficient solution.

Finally, a full-scale fire resistance test will be carried out on a complete housing module. This test aims to verify the overall compliance of the structure with the ‘REI’ criteria, complementing the ‘element-by-element’ checks previously performed. In this test, the thermal action will be applied by igniting and burning wooden elements, with the fire load density defined according to the module’s usage class and risk level. The test will evaluate the adequacy of the studied passive fire protection measures, particularly in the connection areas, and assess the potential need for active measures (*e.g.*, sprinklers).

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