KINEMATICS OF THE COLLAPSE OF A WAREHOUSE STRUCTURE UNDER FIRE

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ABSTRACT

The fire resistance of logistics warehouses is of growing interest to the authorities because of the impact on people and goods safety. According to French regulation and requirements, the structures of these buildings have to satisfy, in fire conditions, thermo-mechanical stability as well as some other criteria. Namely, a fire resistance duration well-matched with the evacuation of the staff and a kinematics of collapse driven in such a way to avoid progressive collapse as well as the collapse towards the outside of the building. The kinematics of collapse depends on several parameters and especially on the kind of structural members. This paper presents a study case of the fire resistance analysis of a multi-frame prestressed concrete structure. The fire resistance assessment is determined according to the recommendations of the Eurocode and the kinematics of collapse is studied taking into account the different modes of failure. Analysis of the structural behaviour is based on thermo-mechanical numerical simulations performed using a three-dimensional finite element modelling. The resistance criteria are in accordance with the prescriptions of the Eurocode dedicated to concrete structures (i.e. EN 1992 part 1–2). The analysis shows that the kinematics of failure is mainly driven by the thermo-mechanical behaviour of the connections between structural members whose shear bearing capacity under thermal load should be taken into account. Solutions and recommendations are given in order to avoid collapse towards the outside of the building.

Keywords: warehouse, fire, kinematics of collapse, thermo-mechanical analysis, FEM modelling.

1 INTRODUCTION

In the last twenty years, a change of fire engineering design has occurred, with a real conviction and will for switching from a prescriptive approach to a performance based approach. This is mainly motivated by [1], [2], (1) allocating resources in such a way to meet real needs (cost-effective design) instead of satisfying the redundancy arising from the prescriptive design, (2) promoting innovation by performing rational design for projects which can be considered unconventional to be covered by the prescriptive codes and (3) getting flexible design to satisfy safety criteria other than those prescribed by a given code.

The main difference between the prescriptive approach and the performance based design (PBD) is that, for the first, the fire resistance and reaction to fire of materials are prescribed by mandatory texts in such a manner that the global objectives of security are assumed to be achieved. This approach is mainly based upon feedback and induces several redundancies. For the PBD [3], the development of fire in buildings is determined precisely according to the actual calorific products and the characteristics of the buildings. For warehouses, the French regulation requirements give the objectives of security. They consist of: 1) a fire resistance duration well-matched with the evacuation of the staff, 2) a kinematics of collapse driven in such a way to avoid progressive collapse and 3) collapse towards the outside of the building is not allowed.

The behaviour of structural elements under fire load includes two successive phases; the first is governed by a thermal expansion of the exposed sides inducing increase of lateral displacement, bowing due to the thermal gradient of temperature and internal forces caused by restraints. The second phase begins when materials lose their mechanical resistance and

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the structure shows instability phenomenon when the fire resistance capacity becomes lower than the effects induced by the external loads.

Steel framework warehouses are generally composed of multiple bay frames and due to the characteristics of the steel they experience generally collapse toward the inside of the building [4]. This phenomenon has been demonstrated by surveys of real fires happened in industrial halls [4] and also by modelling [5]–[8]. The main point which contribute to this behaviour is that the joints temperature is lower than which of the current section due to the presence of additional material. This is not the case for concrete structures and especially for prestressed concrete ones [9].

A case study of a prestressed concrete warehouse submitted to fire is analysed in this paper in order assess the kinematics of collapse. A numerical thermo-mechanical analysis is performed and mainly based on the advanced method described in the Eurocode dedicated to concrete (i.e. EN 1992-1–2 [10]) and the finite element code SAFIR [11]. A proposal to avoid the collapse toward the outside of the building complying with a capacity design is given in the last subsection.

2 COLLAPSE KINEMATICS OF REINFORCED CONCRETE WAREHOUSES

2.1 Description of the structure and hypothesis

The study case concerns the compartment A of the logistics warehouse shown in the Fig. 1. The structure is composed of eleven porticoes made of prestressed concrete elements with two spans each (26 and 22 m). The outer columns have a section of 60 x 70 cm^2 and those of the central file a section of 70 x 60 cm^2. The configuration of the beams is the same for all the porticoes (beam I35 cm x 130 cm for the large span and beam I35 cm x 105 cm for the second span). The connection between the column and the beam (see Fig. 4) includes the beam stub, neoprene plates and column head.

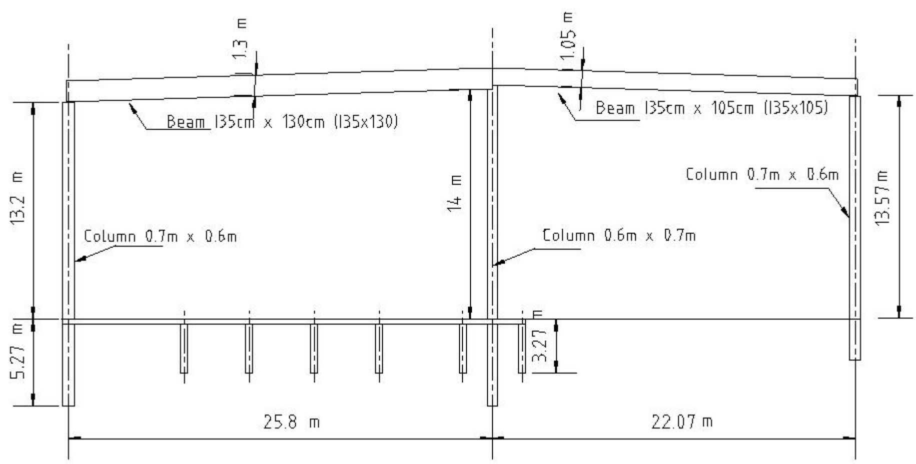


Figure 1: Schematic representation of the structure.

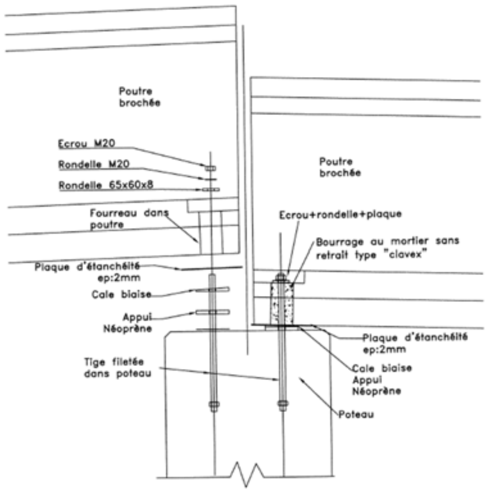


Figure 2: Assembly details between column and prestressed beam.

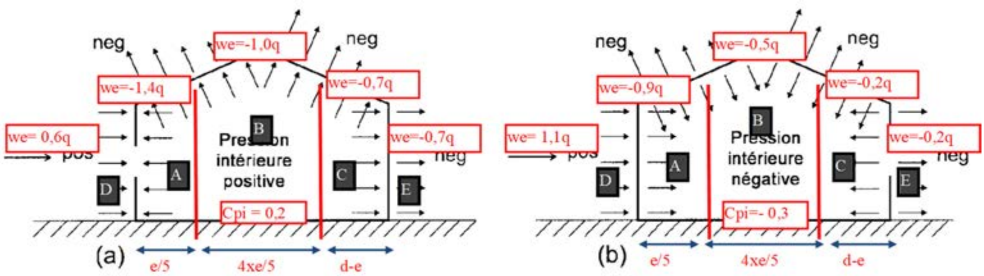


Figure 3: Configuration of wind load acting on the warehouse structure. (a) W_1 configuration; (b) W_2 configuration.

2.1.1 Loads

Live loads are composed of the snow and the wind load. According to the location of the building, the snow load is 45 daN/m². The dynamic pressure acting on the external surface W_e is calculated as a function of the dynamic tip pressure equal to 60 daN/m² and the geometric configuration of the building. All considerations of the Eurocode 1 part 1–4 [12], accounted for, two loading configurations for wind action (W_1 and W_2) have been taken into account in the numerical simulations (Fig. 3).

According to [13], the combinations of loads to account for in the thermo-mechanical analysis are G + 0.2 S; G + 0.2 W_1 and G + 0.2W_2. They are detailed in Table 1.

2.2 Thermal loads

According to the fire development analysis performed in the frame of this work [14], the fire scenarios give two relevant thermal loads as a function of time.

The first one concerns a fire growing in the compartment A (ChA), underneath a central column. In this condition, the latter is submitted to a thermal load described by the continuous curve of Fig. 4. A travelling fire occurs afterwards and the whole structure is concerned by the thermal action represented by the dashed curve of Fig. 4. The second scenario concerns a fire growing in the automated pallet silo (Tr). In this case, the thermal action on the structure is described by the curves given in Fig. 5. The continuous line concerns the central columns and the dashed one concerns the whole structure.

2.3 Thermomechanical modelling

2.3.1 Heat transfer analysis

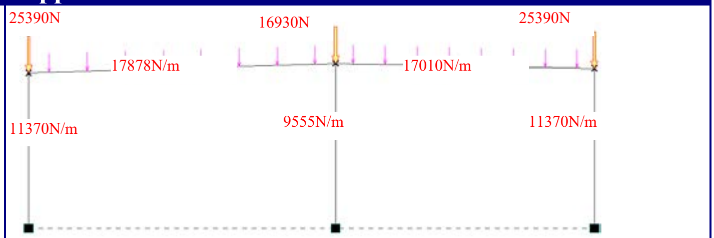
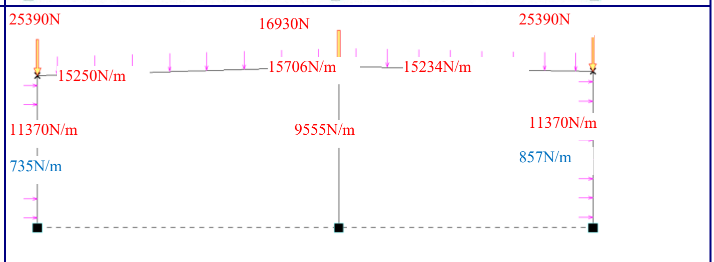
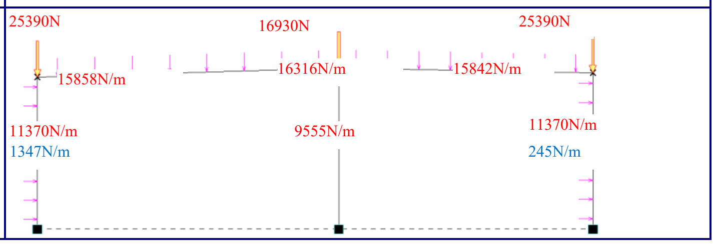
The calculation of the heating of the various elements is carried out using the FE code SAFIR [10]. For each structural element, a 2D heat transfer calculation is carried out. The geometric domain delimited by the current section of each element is discretized and the boundary condition defined by the corresponding thermal action is applied to it (see §2.2). The thermal exchange conditions between the modelled element, the thermal load and the external environment are those prescribed by the Eurocode [12]. The thermophysical characteristics of the different materials are those given by the respective Eurocodes ([10] and [15]).

Two different thermal loads were taken into account according to the curves in Fig. 4 and Fig. 5. The different temperature fields were used afterwards in the fire resistance analysis. The different beams of the compartment are considered exposed on all sides. The central and the outer columns are considered to be exposed to fire on all four sides since the polyurethane of the inner facing has a melting temperature of 170°C which can be reached quickly with the thermal actions taken into account. For illustration purpose, Fig. 7 gives the thermal field at 120 minutes and the curves of Fig. 8 give the temperature evolution as a function of the heating time at a few points of the section.

The outer columns and those of the central file, have a reinforcement along 4 m from the supports consisting of 8 HA32 and 10 HA32. The prestressing strands are continuous throughout the length of each column. Fig. 8 gives the thermal field of an outer column at 120 minutes of thermal loading.

In order to assess the thermomechanical behaviour of the connection in fire conditions, a heat transfer analysis of the assembly zone (Fig. 2) has been performed. In the longitudinal direction, the rod is located at 10 cm from the end of the beam. Given the dimensions of the connection zone and taking into account symmetry consideration, the modelling is limited to the zone close to the connecting rod. A 2D simulation is retained because the end of the beam is far enough from the rod and one can neglect boundary effects in the longitudinal direction. The column is not modelled and its influence on the temperature field is implicitly taken into account by an adiabatic boundary condition. Part (a) of Fig. 9 shows the mesh adopted for the temperature field at 120 minutes given in the part b of the same figure. The Fig. 10 gives the temperature as a function of time at different key nodes given in Fig. 9. One can notice that the temperature reached in the rode (Node 3) becomes significant after 40 minutes, for which the temperature is 400°C and corresponds to the beginning of reduction of the thermomechanical resistance of steel (cf. [15]). At 60 minutes, the temperature of the rod is 550°C, which corresponds to a value of 35% for the reduction factor. It is then relevant to take the resistance of the connection zone in order to assess the fire resistance of structure.

Table 1: Combination of loads.

Load case	Applied loads
Case $P_1 = G + 0.2\ S$ $S = 45 \times 10.2 = 459$ daN/m	
Case $P2 = G + 0.2W_1$ $W_1 = Cp \times q = Cp \times 60 \times 10.2 = 602 \times Cp$ daN/m	
Case $P3 = G + 0.2W_2$	

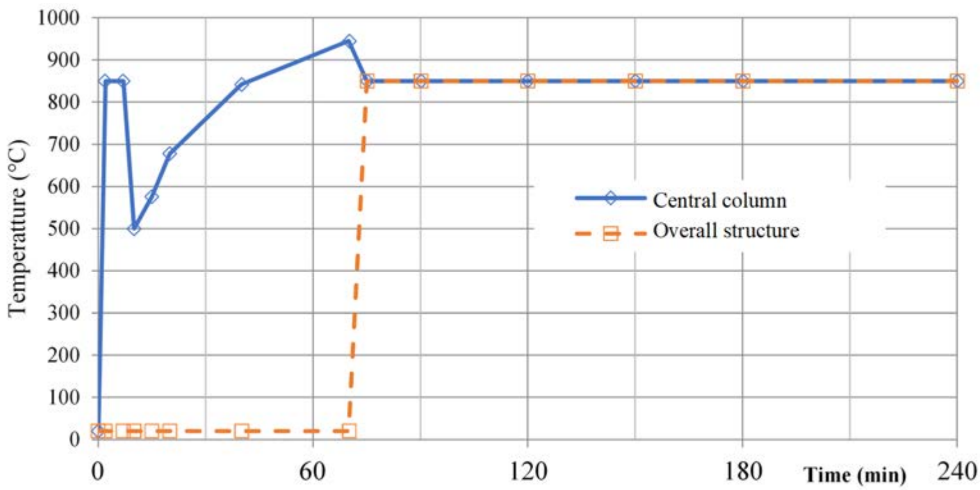


Figure 4: Thermal action applied to the main structure (scenario 1).

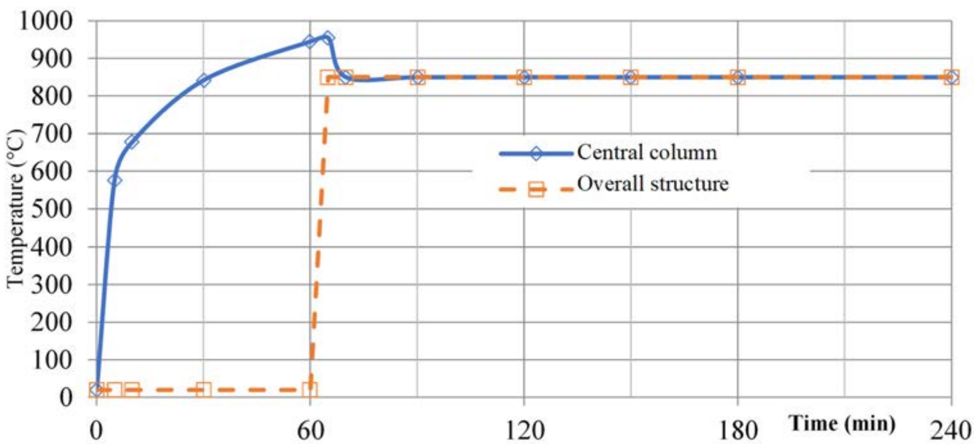


Figure 5: Thermal action applied to the main structure (scenario 2).

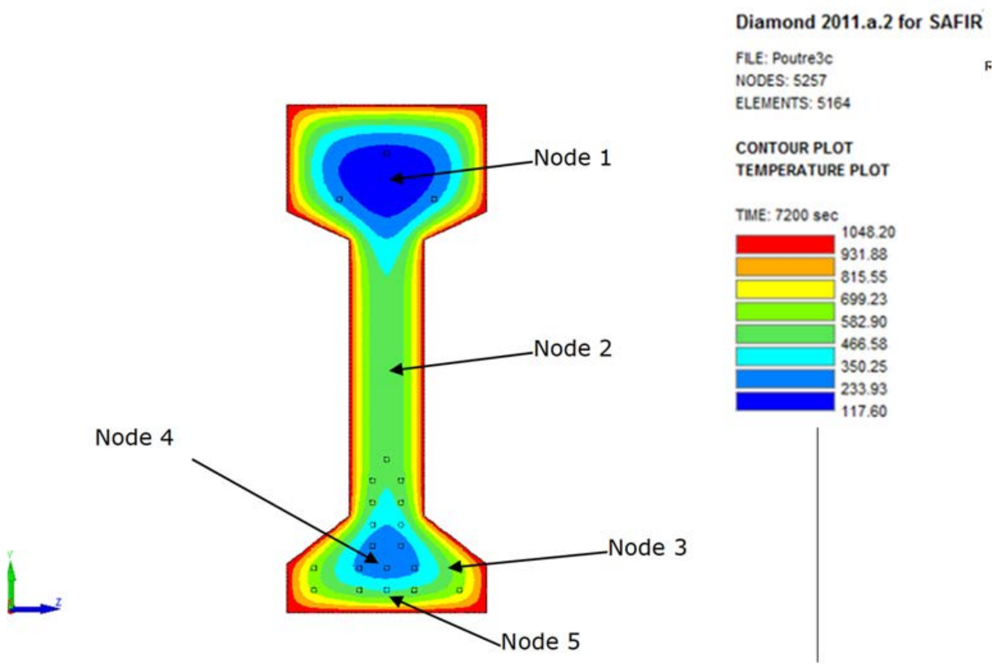


Figure 6: Isotherm of the beam section at 120 minutes.

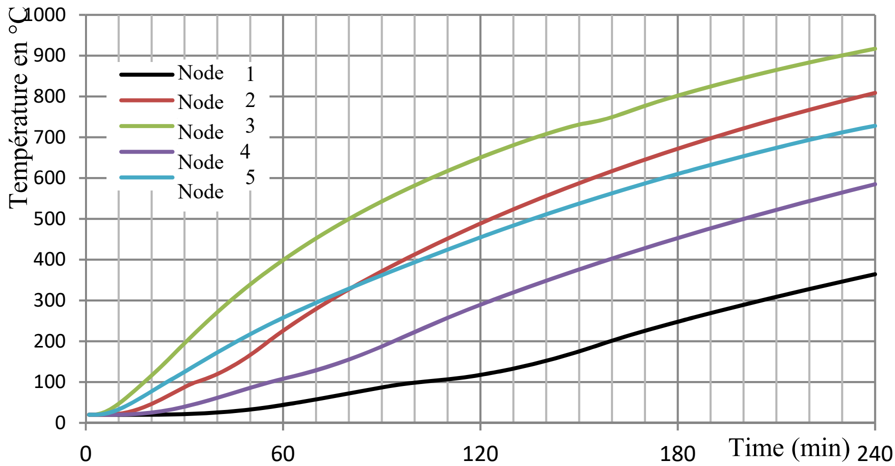


Figure 7: Beam temperature as a function of time.

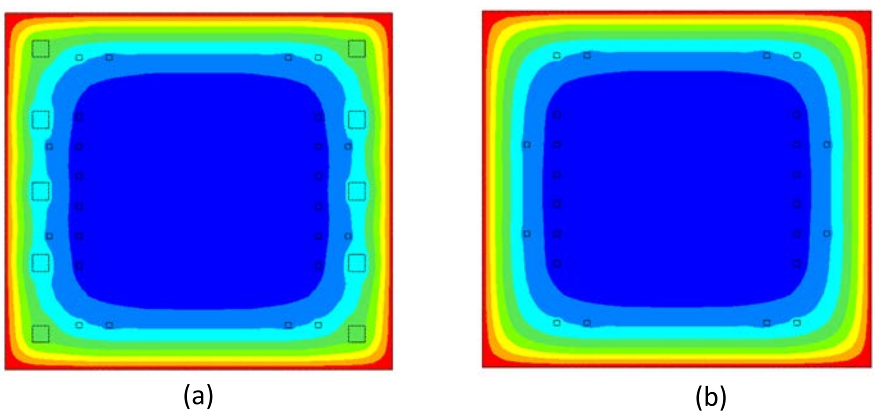


Figure 8: Temperature field of the 60cm x 70 cm column. (a) Lower part of the column; (b) Upper part of the column.

2.4 Thermomechanical analysis

The load cases of Table 1 were combined with the two thermal loads of Fig. 4 and Fig. 5. The results of the various numerical simulations in terms of resistance time, synthesized in Table 2, show that the worst mechanical loading case (giving a lower resistance time) is the case P1 combining the action of snow with the dead load. The load case P3 shows, moreover, a greater displacement at the head of the column.

Fig. 11 shows the deformed shape of the structure for the combination load case P1 just before the failure. Fig. 12 and Fig. 13 show, respectively, the midspan deflection of the two beams and the horizontal displacement at the head of the columns. We deduce that the structure behaviour is relatively the same with the thermal loads taken into account in this study. The displacement at the head of the column remains relatively small given the height of the column. The predominant failure mode is governed by the beams behaviour.

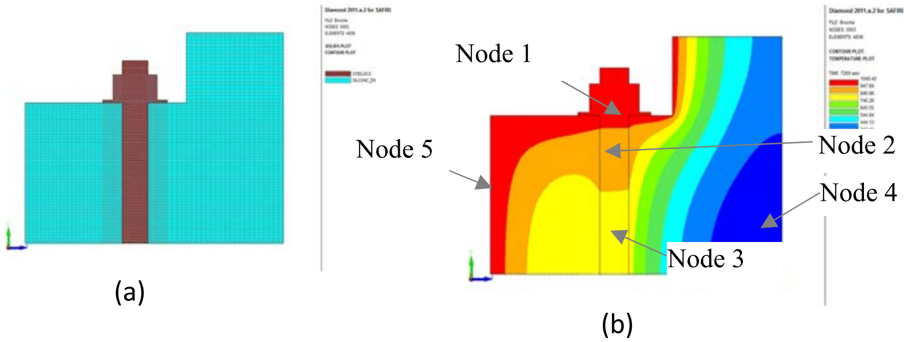


Figure 9: Modelling of the connection zone. (a) Finite element mesh; (b) Temperature field at 120 minutes.

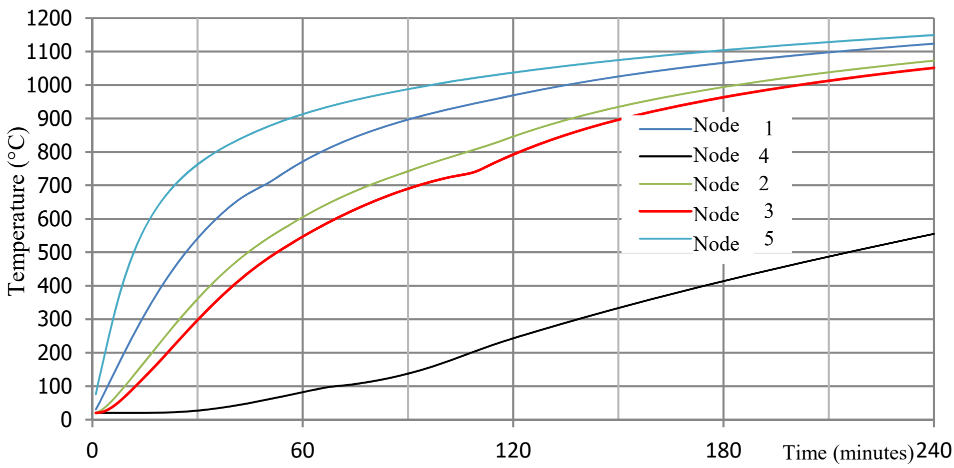


Figure 10: Connection zone, temperature as a function of time at key nodes.

Table 2: Results of numerical simulations.

	Fire in the APS	Fire in the compartment A
Case P1	2 hours 35 min (0.60m)	2 hours 43 min (0.63m)
Case P2	2 hours 42 min (0.69m)	2 hours 45 min (0.66m)
Case P3	2 hours 44 min (0.69m)	2 hours 47 min (0.69m)

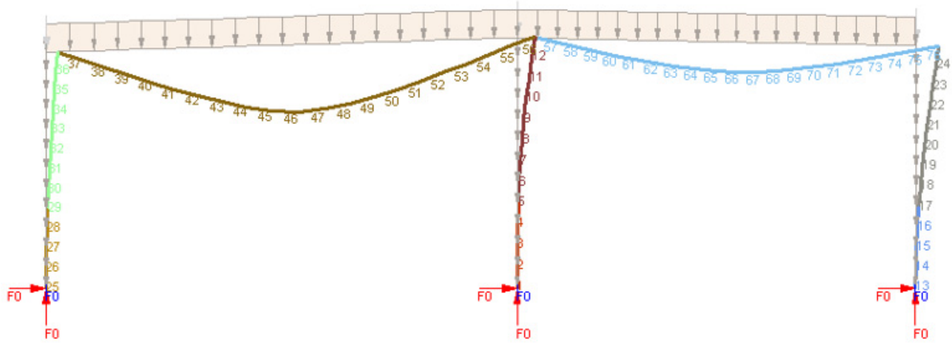


Figure 11: Deformed shape of the portal before the failure.

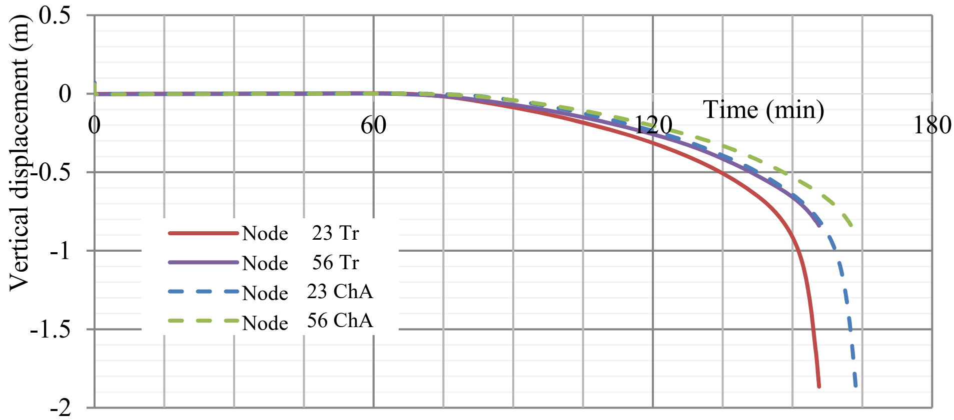


Figure 12: Vertical displacement at the middle of the beams (load case: P1).

2.5 Thermomechanical joints analysis

The behaviour of the structure depends on that of the beam\column connection. The heat transfer analysis shows that the steel characteristics are weakened due to the heating of the rod. The verification of the rods shear mechanical resistance can be performed according

$$\frac{3G + N}{A} < f_y \tag{1}$$

where fy is the resistance of the steel, G is the shear force, A is the section of the rod and N is the prestressing normal force applied to the rod (N is positive in tension).

Under fire load, the shear resistance can be derived from eqn (1) by introducing the reduction factor $k_{y,\theta}$ describing the reduction of the steel resistance as a function of temperature [15]. The safe shear load as a function of the temperature θ, can be written according eqn (2),

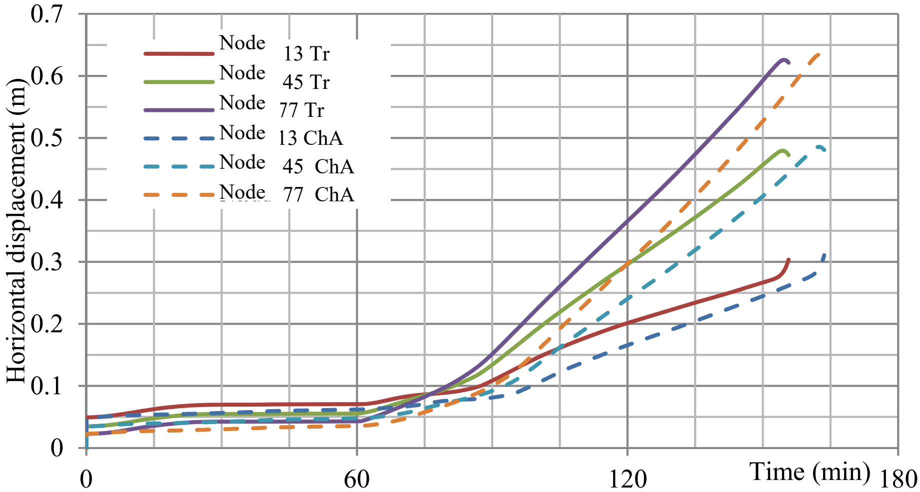


Figure 13: Horizontal displacement of the upper part of columns (load case: P_1).

$$G_{max}(\theta) = \frac{A \cdot k_{y,\theta}(\theta) \cdot f_y - N}{3} \tag{2}$$

The prestressing force N is counted negative for compression forces and we consider it null under thermal load. As a result, for a 20mm diameter steel rod with a yield strength of 480 MPa (grade 6.8), the shear stress is $G_{max}(\theta)$= 50.26 $k_{y,\theta}(\theta)$ (kN). Each beam is fixed to the column by 4 rods (2 at each side). Therefore, the maximum normal force in the beam is 2 × G_{max} and depends on the temperature according to the previous formula. The beam/column connection will be considered efficient if, in each case, Np < G_{max}, with Np the normal force in the beam. Fig. 14 shows the evolution of the lower and upper limits of the shear resistance as a function of time and implicitly as a function of the temperature calculated in the connection (node 3 of Fig. 10).

2.6 Kinematics of collapse

Results of thermomechanical simulations are to be completed by an analysis of the thermomechanical resistance of connections between beams and columns. For this purpose, Fig. 14 shows the evolution of the normal force in the beams as a function of time of numerical simulation according to eqn (2) and the evolution of the temperature of the connecting rod. In the same figure, the time-shear force of the connecting rods is given. From these curves, it can be seen that the failure of the assembly of the large span beam (left span) with the central column occurs at 80 minutes. The connection of the second beam follows and occurs at 88 minutes. From the progressive collapse point of view, the failure of the connection of the first beam will induce a redistribution of the internal forces and generate, among other things, a lower thrust force on the second span of the portal.

In order to analyse the impact of this failure on the remaining span, another modelling has been performed considering only the right hand span and reproducing the forces induced by the left span as a boundary condition applied on the top of the central column (Fig. 15). The Fig. 16 shows the compression force at the remaining span compared to the shear resistance of the beam/column connection. It shows that the connection with the central columns occurs first (after two hours of thermal stress).

Following this event, the outer column will be subjected to its own weight and possibly to a thrust force coming from the beam which rests on the ground and on the head of this pole. The verification of the criterion of no collapse to the outside consists then in performing a fire resistance analysis of the outer column in its deformed state subjected to the addition of these forces. The estimation of the horizontal force gives a value of 24 kN considering a static analysis. A fire resistance duration of 190 minutes was found for this configuration.

In order to avoid this situation and comply with the objectives of security, one solution consists in increasing the steel class of studs of the central columns (and only these columns), so that the failure of the connections will be initiated at the level of the outer columns and will therefore be unloaded. In this case, the calculation shows that the risk of collapse towards the outside is avoided. The Fig. 17 gives an illustration of this principle.

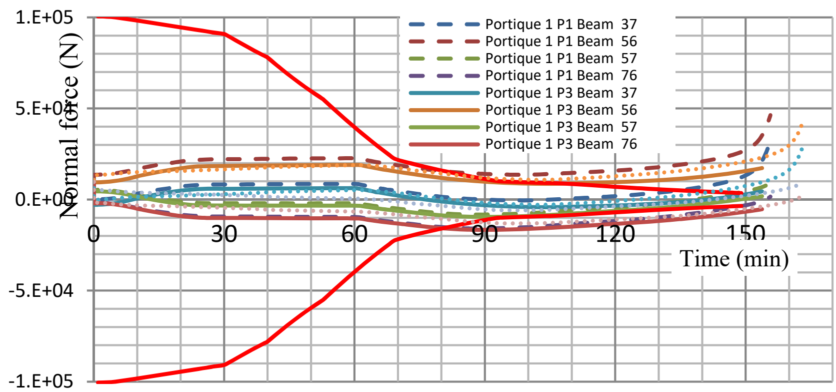


Figure 14: Normal forces as a function of time on the beams and the upper and lower limits of the rod shear resistance.

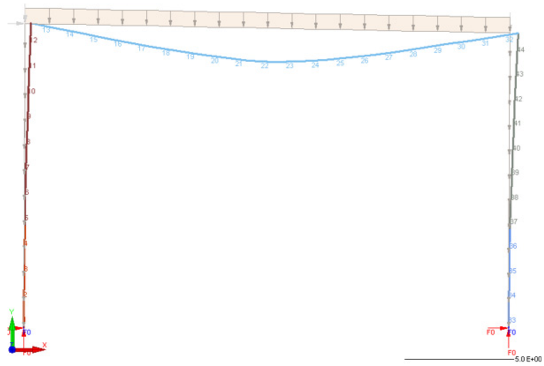


Figure 15: Modelling of the remaining portal bay after the collapse of the bay 1.

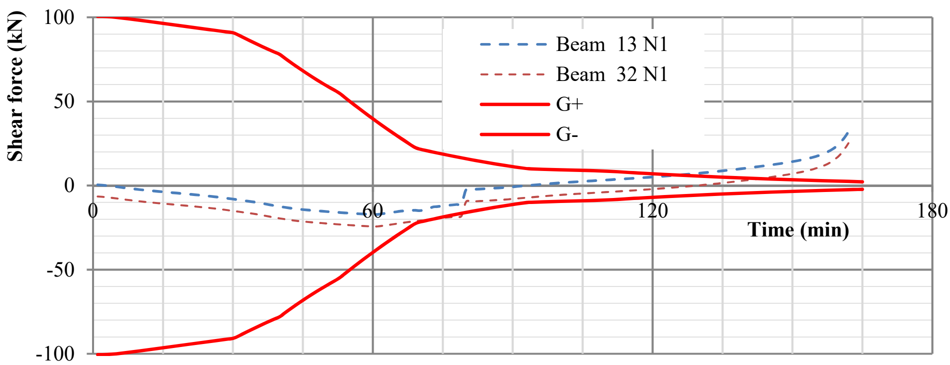


Figure 16: Normal forces in the connections and G^+ and G^- bounds.

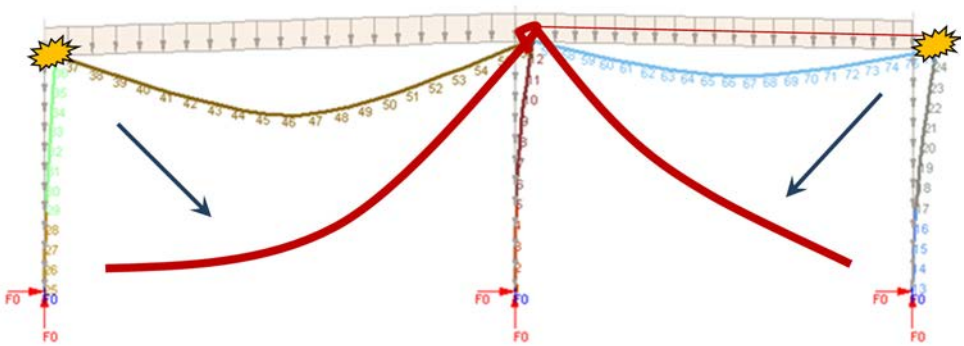


Figure 17: Principle to avoid collapse towards the outside of the building.

3 CONCLUSION

Unlike the steel framework structures, the safety objective of non-collapse towards the outside of reinforced concrete structures subjected to fire is an up to date issue. The case study presented in this paper shows that the thermomechanical behaviour of the beam/column connections has a significant effect on the overall behaviour of the structure. In order to assess a more relevant fire resistance duration or the kinematics of collapse, it is important to take them into account in modelling by a sequential approach (when a more precise approach is not possible). According to the kinematics of collapse, a proposal for a design principle has been made in order to avoid the risk of collapse towards the outside of the building and then to comply with the objective of security in the frame of a performance based design.

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