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Progressive collapse triggered by fire induced column loss: Detrimental effect of thermal forces

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Abstract

In progressive collapse analysis, event-independent column loss is commonly used as a design scenario. Yet this scenario does not account for the fire-induced thermal forces that develop in case of a fire. The thermal forces may cause detrimental load redistributions in the structure, notably during the cooling phase. However, as the response of entire structures during the full course of fires until burnout has received little attention, these effects are not well established. The objective of this paper is to analyze the mechanisms of load redistribution in a structural system comprising a column subjected to localized fire, with a focus on the effects of the cooling phase. Numerical simulations by nonlinear finite element method are used, after validation against experimental data. The observed mechanisms result in tension building up in the fire-exposed column and overloading the adjacent columns in compression. Consequently, the damaged vertical member redistributes a force that is larger than the force initially carried. This can lead to failure of vertical members not directly affected by the fire and trigger a progressive collapse. These mechanisms are parametrically studied on a simple system composed of a column and a linear spring. Major parameters influencing the residual tensile force in the fire-exposed column are the maximum reached temperature and the relative stiffness of the remainder of the structure. The analysis of a twenty-story steel frame building under localized fire attacking one ground level perimeter column confirms the development of these mechanisms in a real design. The results have important implications as they question the validity of an event-independent design scenario for capturing the influence of column failure due to fire loading.

Keywords: Progressive collapse; Thermal loads; Localized Fire; Multi-story buildings; Steel frame; Robustness

1. INTRODUCTION

Events such as the Ronan Point Tower partial collapse in 1968 and the World Trade Center collapse in 2001 have highlighted the importance of understanding and preventing the mechanisms of progressive collapse of tall buildings. The key idea is that, in case of an accidental event causing local failure of a structural element, this failure should not spread from element to element eventually resulting in a collapse disproportionate to the original cause [1]. One design approach that can be used to satisfy this requirement is called the alternative load path method [2-3]. This approach to progressive collapse resistance allows local failure to occur when subjected to an extreme load, but ensures that alternative load paths are activated toward a new static equilibrium upon this local failure, therefore preventing the spread of damage and the occurrence of a global collapse. In other words, the structure has enough redundancy to absorb the effect of the loss of one or several structural bearing elements.

The alternative load path method is generally regarded as a threat-independent method. The specific event triggering the local failure is not explicitly considered; instead, one or several key structural elements are removed from the structure and the subsequent ability of the system to bridge over the failed elements and withstand the loads is evaluated [4]. The main advantages of this approach are that, first, it relieves the designer from the necessity to describe the accidental event and the way it will affect the structure and, second, it ensures that, normally, the structure has the ability to survive any accidental event, be it of a foreseeable nature or not. This is why it has been extensively used to investigate progressive collapse of tall buildings due to column loss [5-8].

The loss of a column in a tall building can result from a variety of exceptional events. For instance, the NIST [9] has categorized the potential abnormal load hazards that can trigger progressive collapse as: aircraft impact, design/construction error, fire, gas explosions, accidental overload, hazardous materials, vehicular collision, bomb explosions, etc. Threat-independent sudden column loss has been proven an appropriate design scenario to capture the effect of column failure due to impact or blast [5]. Researchers have worked toward a simplified method to capture the dynamic effects of a column failure occurring over a relatively short time [10-12]. The United Facilities Criteria provisions by the Department of Defense [3] specify that, for nonlinear static analysis of notional member removal, the structural response must be amplified by a dynamic increase factor to account for dynamic effects. This dynamic increase factor for nonlinear static analysis depends on the structure type and plastic rotation limit with a maximum value of 2. However, fire loading is a specific scenario which may not be adequately captured by this approach. The main issue with fire is related to thermal forces, not to dynamic effects. Fire generates thermal forces in a structure, which may lead to a redistribution of loads that differs from the one evaluated from an alternative load path method that considers the removal of the affected column. This is particularly the case when a localized fire affects only one part of the structure, and even more so during the cooling sequence that occurs in a real fire. The mechanisms at stake are studied in details in this paper, notably to assess whether the case of fire-induced column failure may or may not be properly modeled by notional removal of the column as specified in the alternative load path method [9].

It is noteworthy that, in structural fire engineering, the prevailing approach has long been prescriptive, considering isolated member behavior rather than system behavior, and continuously heating standard fire exposure rather than real fires with their heating phase followed by a cooling phase. As a result, the understanding of the behavior of entire structures under real fires remains limited. Since the Cardington fire tests, it has been established that system behavior differs dramatically from isolated member behavior, and that the effects of thermal expansion dominate the response of the structure [13-15]. The NIST final report on the collapse of WTC 7 [16] mentions that the factors contributing to the building failure include the thermal expansion effects and a structural system that was not designed to prevent fire-induced progressive collapse. The NIST report highlighted the current knowledge gaps in structural fire response and formulated a number of recommendations, including the "development of methods for prevention of progressive collapse and for reliable prediction of the potential for complex failures in structural systems subjected to multiple hazards" and the "enhancement of the fire resistance of structures by requiring a performance objective that uncontrolled building fires result in burnout without partial or global (total) collapse" [16]. Consequently, recent research has looked at the behavior of structural assemblies under more sophisticated fire scenarios, analyzing for instance the effects of travelling fires on buildings [17-18], of fires with cooling phases on connections [19-21], or of tanker truck fires on bridges [22-23]. Due to the interaction between members under restrained thermal strains, significant load redistributions are known to occur. The development of residual tension axial force in members after fire exposure has been described by several authors, for instance in beams of multi-story frame structures under single bay fires [24], or in connections [21]. Yet, in previous investigations on the issue of fire induced progressive collapse, the focus was mainly on ultimate failure temperature and load redistribution mechanisms during the heating phase [25-28]. Hence, the mechanisms that develop during the cooling phase in an entire structure subject to a localized fire remain still to be fully understood. Meanwhile, numerical studies of isolated members under fires with heating and cooling phases have shown that failure may occur during or even after the cooling phase [29-30]. The hypothesis tested in this research is to assess whether the load redistributions in a structure during the cooling phase of a fire may threaten stability and lead to progressive collapse. Filling this gap is especially essential as delayed structural collapses, which have been observed in real fire events, pose a specific threat to firefighters [31].

In this context, this paper investigates the behavior of a structural system comprising a column subjected to localized fire. The objective is to analyze the mechanisms of load redistributions that develop in a system during the full course of a fire affecting a part of this system, including the eventuality of progressive collapse provoked by load redistributions in the cooling phase. The mechanisms are analyzed using numerical simulations, supported by experimental evidence. The results have significant implications both at the modeling level and at the design level, which are discussed in the paper.

Section 2 discusses the mechanics of a structure response under a localized fire scenario affecting one column of the system. Section 3 presents a parametric analysis of a system composed of a column and a linear elastic spring representing the surrounding structure. Section 4 presents the simulation of an experimental test previously published in the literature which allows validating the modeling approach to capture a structure response under localized fire. Finally, Section 5 studies numerically the behavior of a multi-story moment resisting frame structure subjected to localized fire, while Section 6 presents the conclusions. The numerical analyses are performed with the nonlinear finite element software SAFIR® [32] developed specifically for modeling the behavior of structures in fire.

2. FORCE REDISTRIBUTIONS IN A STRUCTURE WITH A COLUMN SUBJECTED TO LOCALIZED FIRE

2.1 Scope of the study

This paper analyzes the behavior of statically indeterminate structures in which one column is subjected to fire. The discussions are illustrated on multi-story steel moment resisting frames (MRF) structures, although the principles are valid generally.

Real fires are considered (as opposed to a standard fire), i.e. fires that comprise a heating phase followed by a cooling down phase, at the end of which the temperatures come back to ambient. A standard fire, in contrast, is primarily defined for furnace testing of building elements, and comprises only a heating phase. Yet as, in a real event, the amount of fuel (and oxygen) present in a compartment is necessarily limited, it is natural to assume that the temperatures eventually come to a peak and then decrease, and it is therefore essential to understand the behavior of fire-exposed structures during the cooling phase as well.

In this paper, the studied scenario assumes that only one column is attacked by the fire. This scenario can result from a localized fire occurring in the immediate vicinity of the column [33-34], or from the specific compartmentation of the building. Localized fires are of particular interest in the analysis of fire-induced forces in structures because of the generated differential thermal elongations between members. As the study focuses on the influence of fire-induced failure of a vertical member on progressive collapse, the heating of the beams adjacent to the column is neglected. It is expected that heating of these beams would lead to additional horizontal forces in the structure (due to restrained thermal elongation) but also to a reduction in the degree of axial restraint for the heated column (due to reduction in stiffness of the heated beams). This latter effect would decrease the amount of (vertical) load redistribution during heating and cooling of the column. Therefore from the progressive collapse analysis perspective, the situation where only the column is heated may be the most dangerous because it considers the highest possible restraint stiffness for the column in the considered building.

2.2 Description of the force redistribution mechanisms during a fire

A schematic discussion is used to illustrate the mechanisms of force redistribution. We consider a four-bay, two-story Moment Resisting Frame (MRF) with its central column exposed to fire on the ground floor. The temperature in the member (thermal response) influences the structural behavior (mechanical response). The following hypotheses are adopted to simplify the discussion: there is no buckling; the thermal strains and yield strengths are reversible with temperature; the temperature is uniform in the section. However, these hypotheses are not required for the mechanisms to develop.

Figure 1 shows the schematic evolution of the axial loads in the ground floor columns. The axial loads are represented by arrows at different stages of the fire. The length of the arrow is representative of the magnitude of the force and the direction indicates compression (upward) or tension (downward). Columns are designated from A (left) to E (right) (Figure 1-I). Figure 2 plots the evolution of the axial force in columns B and C, relative to the initial axial force at ambient temperature, as a function of time.

In the first stage of the fire, heating of the central column C leads to thermal expansion of this column (Figure 1-II). However, as this elongation is restrained by the surrounding structure, which remains at ambient temperature, a thermal induced compression force builds up in column C. The amount of the latter force depends on the temperature and stiffness of the heated column as well as on the stiffness of the surrounding structure (the bending stiffness of the beams at the first and second floor, the axial stiffness of the columns). In the heated column, this thermal induced force adds up to the initial compression resulting from the externally applied loads. Meanwhile, the other columns are unloaded by the same amount (in total), because the externally applied loads remain constant during the fire event.

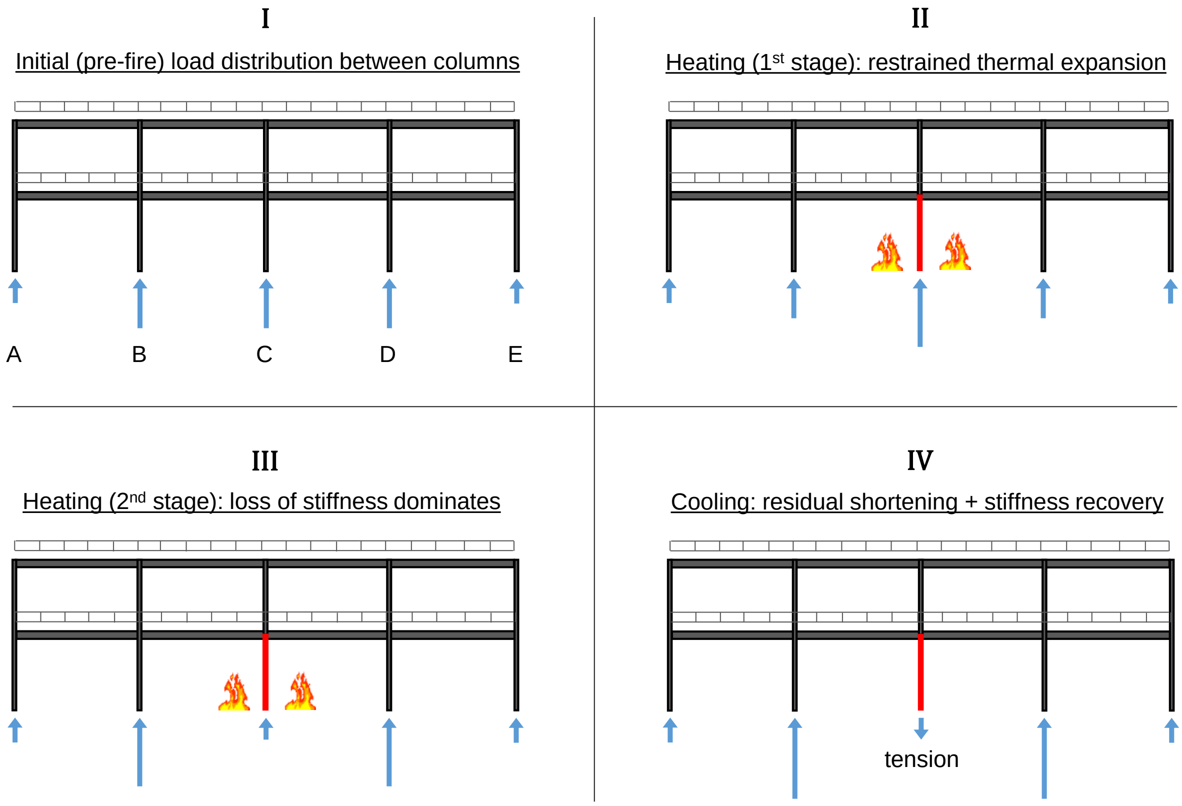

Figure 1. Force redistribution during a fire event affecting a column part of a frame.

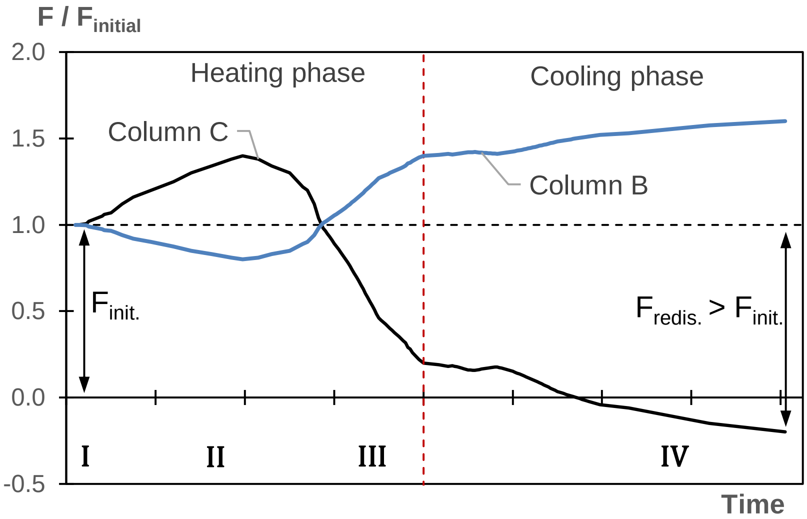

Figure 2. Axial load evolution in the heated column C and the adjacent column B. Numbers I-IV refer to the different stages of Figure 1. F_{init} is the initial axial force in column C. F_{redis} is the total force transferred from column C to the surrounding structure due to the fire. $F_{redis} > F_{init}$ because residual tension adds up to the initial compressive force.

When the heating continues, the decrease in material strength and stiffness in the heated column eventually dominates over the restrained thermal expansion, leading to yielding and unloading of this column. Eventually, the value of the axial force becomes smaller than the initial force carried by the column before the fire (Figure 1-III). Yet, as column C is part of a system (which is assumed to be robust), the load can be redistributed to the surrounding structure. At the end of the heating phase, the weakened column C carries only a fraction of its initial load, with the rest being redistributed to the adjacent columns.

Finally, the fire enters in the cooling down phase. The temperature in column C decreases, though with a certain delay related to thermal inertia. As the thermal expansion is recovered, the column length decreases. Yet, because yielding has developed at elevated temperature, the final length of the column, when thermal expansion is completely recovered, is shorter than prior to the fire. As a result, tension eventually builds up in the column (Figure 1-IV). In other words, column C, after experiencing the heating-cooling sequence, is pulling on the rest of the structure. When the temperature is back to ambient, the column has recovered its strength and stiffness (at least most of it, depending on the maximum reached temperature), so the restrained shortening may generate significant tensile forces. Therefore, column C may eventually transfer a force to the other vertical members which is larger than the load it was initially carrying (Figure 2). Here, the "increase factor" for the transferred force (as typically defined for nonlinear static analysis of notional column removal) does not come from a dynamic effect, but from a thermal effect. As thermal effects are fundamentally different from dynamic effects, the use of the common event-independent column removal approach with "dynamic increase factor" (such as in [3]) is not adequate for fire hazard.

This qualitative discussion shows that fire loading is a specific scenario when it comes to analyzing the influence of a vertical member failure on progressive collapse. The assumption consisting in removing the attacked column and redistributing the gravity loads (while accounting for some dynamic increase factors when using a nonlinear static analysis) to the surrounding structure is inadequate when it comes to fire hazard. In some cases, this assumption may be unconservative, because significant tension can eventually build up in a column subjected to a fire, which generates large additional vertical forces to redistribute to adjacent members, as will be shown in the next sections.

2.3 Role of the surrounding structure stiffness in redistributing the forces

The behavior illustrated by Figure 1 requires the ability for the structure to redistribute the loads from the fire-exposed column to the surrounding structure. In a structure experiencing a column loss, alternative load paths can be activated if there is sufficient resistance and ductility in the structure [35]. A transition from flexural to tensile load transfer happens upon loss of the column [36]. This requires beam-catenary or slab-membrane action to develop for bridging over the failed column and, consequently, provide continuity within structural members [37-40].

The stiffness of the structure surrounding the fire-exposed column is a key parameter. Sun et al. [41] have observed that, in frame-type structures subjected to a column failure, a global collapse happens when the beams have higher sections while a localized collapse occurs when the beams have smaller sections. Indeed, two extreme cases can be considered. On the one hand, if the stiffness of the surrounding structure is infinitely low, the situation is akin to an isolated column. During the fire event, the axial force in the column remains approximately constant (since there is virtually no restraint). When the temperature reaches the critical temperature of the member, the column fails with no possibility to transfer its load. The problem is a ductility one: because the stiffness of the surrounding structure is so low, it would require extremely large displacements (e.g. for the beams in catenary action) to redistribute the loads; yet such displacements cannot be accommodated in practice.

On the other hand, if the stiffness of the surrounding structure is infinitely high, the situation tends to that of a completely restrained column. During the fire event, the displacement at the top of the column remains approximately constant. The axial force varies largely with temperature. When the column stiffness drops as a result of temperature increase, the loads are redistributed to the surrounding structure; there is robustness in the system. However, there might be a resistance problem, as the adjacent columns need to be able to withstand the significant additional loads coming from the heated column.

The question discussed in this paper is that, in case of a fire event, the resistance problem becomes more critical than with other exceptional events due to the effect of the heating-cooling sequence on the column. When the stiffness of the surrounding structure is large, it restrains the thermal contraction of the column in the cooling phase, therefore attracting more loads. Large tensile forces may build up in the fire-exposed column and overload the adjacent columns (Figure 2). It is clear that the stiffer the surrounding structure, the more restraint it brings to the fire-exposed column and thus the more force it can attract during the cooling phase. Indeed, one specificity of fire as a loading case is that it induces restraint forces in a statically indeterminate structure. Besides, these restraint forces can change sign between the heating and cooling phases. By nature, these forces are larger when the stiffness at the restraint is larger.

As a result, excessive stiffness may be detrimental to the safety of the structure in case of a fire event leading to a column loss. Although sufficient stiffness is required for robustness, there may be a counter-productive effect in adding too much stiffness for instance by designing very large transfer beams between columns.

In a real structure, the magnitude of the stiffness restraining the axial displacement of a column is difficult to quantify. Indeed, this stiffness comes from structural as well as non-structural elements, where the former component can be estimated but the latter is largely unknown. If the designer is concerned essentially with the structural behavior at ambient temperature, it is reasonable to assume that this hidden stiffness has a beneficial effect and therefore that it can be conservatively neglected. However, in case of a fire event, this may not be the case. This hidden stiffness brings additional restraint to the column and therefore leads to additional transfer of forces to the surrounding structure. During cooling, larger tensile forces may build up in the fire-exposed column and be redistributed as an additional burden to adjacent columns.

The influence of the stiffness of the surrounding structure as a parameter influencing the axial force evolution in a restrained column is investigated more in detail in the next Section.

3. ANALYSIS OF A SYSTEM COMPRISING A RESTRAINED COLUMN SUBJECTED TO HEATING AND COOLING

3.1 Method

To quantify the behavior discussed in the previous Section, the simple system of Figure 3 is analyzed. In this system, the structure surrounding the fire-exposed column is represented by a spring. The stiffness of the spring can be adjusted to represent different stiffness of the rest of the structure on the column. It is assumed that the spring has a linear behavior, which in fact introduces the hypothesis that the rest of the structure can accommodate the vertical displacements of the column in an elastic state. The column is a HEA 300 section with a yield strength of 355 MPa. Buckling is prevented.

An initial force F is applied to the system; then the column is subjected to heating and cooling. The force F, maintained constant during the fire, is distributed between the spring (F_s) and the column (F_c). The spring stiffness, which remains constant, is noted K_s. The column axial stiffness, which is temperature dependent, is noted K_c. The degree of restraint provided by the rest of the structure is evaluated through the relative stiffness $R = K_s / K_c$. The temperature distribution in the column section is uniform. Temperature creates thermal elongation of the column and a variation of its mechanical properties. The non-linear stress-strain relationship from Eurocode 3 part 1-2 (Figure 3.1 in [42]) is adopted with the corresponding reduction factors for the properties (Table 3.1 in [42]). For the sake of simplicity, the analysis is made in the temperature domain rather than in the time domain. Finally, it is assumed that steel strength is partly irrecoverable: a loss of 0.3 MPa/K is assumed when steel has been heated beyond 600°C.

The initial value of the relative stiffness (at ambient temperature) $R = K_s / K_c$ is varied from 0, i.e. no restraint, to ∞, i.e. a full restraint. The external load applied on the system, F, changes from one simulation to another as a function of R to induce in the column the same initial axial load F_c in all cases. The underlining idea is that the analysis of the global structure at room temperature has given a certain value of the axial force in the column, irrelevant of the amount of restraint. Only the evolution of the axial force during the fire will be influenced by the restraint. Here, the initial value of F_c is equal to 50% of the column capacity at ambient temperature $F_{y,20°C}$.

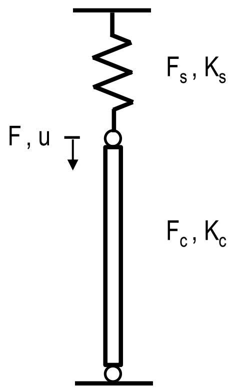

Figure 3. Simple model of a restrained column.

3.2 Results

Figure 4 shows the evolution of the axial force in the column as a function of the temperature in the column. The three plots relate to three different maximum temperatures reached in the column (400°C, 600°C and 800°C) before cooling down to 20°C. For each plot, different degrees of restraint have been considered.

A first simulation is made with no axial restraint at all (R = 0) and a continuously increasing temperature (at a rate of 30K/min) to determine the temperature at failure. The axial force remains constant and the failure is obtained for a temperature $T_{critical}$ of 590°C, see the curve labelled as "R = 0.00 hot". This corresponds to the temperature at which the steel material has lost 50% of its initial yield strength according to Eurocode 3 part 1-2 [42]. Another limit case is the full restraint (R = ∞), which is also analyzed for a continuously increasing temperature. This case leads to a very fast increase of the axial force in the column in the first stage, followed by a progressive transfer of the load from the column to the spring due to yielding of the column. The temperature can then be increased indefinitely, up to the point when the steel in the column has lost all strength and the load is totally carried by the spring, see the curve labelled as "R infinite hot". Of course, this behavior is only realistic provided that the rest of the structure (modeled by the spring) is resistant enough to accommodate this transfer of load. The other curves in Figure 4 are obtained by heating and cooling the column under various levels of restraint.

The following observations can be made:

- In the heating phase, the axial force in the column increases progressively during the heating, reaches a peak, and then decreases. The axial force peak value is larger and occurs earlier (i.e. at a lower temperature) for larger values of the degree of restraint.
- Owing to the presence of the spring, the columns do not "fail", even when the maximum temperature exceeds the critical temperature of the column seen as an isolated element, i.e. 590°C. Of course, the notion of failure is a subject of debate. The axial force in the column may become much lower than its initial value. Yet the consideration of the system makes it possible in the analysis to redistribute the loads and go beyond the temperature of failure of the isolated column.
- In the cooling phase, the column experiences unloading up to a residual axial force when the temperature is back to 20°C. This residual force is lower than the initial force, due to the fact that the column has yielded during the fire and that it has shrunk, except for very low values of maximum temperature and degree of restraint for which the column behavior remained elastic. As a consequence, part of the force that was supported by the column before the fire has been transferred and is supported by the rest of the structure after the fire. The amount of force that is transferred from the column to the spring after the heating-cooling sequence increases with the maximum temperature and degree of restraint.
- Tension can build up in the column when the rest of the structure is sufficiently stiff and the column has been heated to high temperatures. This implies that the force transferred to the rest of the structure exceeds the initial axial force in the column.
- The maximum value of the tensile force that can build up in the column equals the maximum section (yielding) capacity in tension. Such a residual tension results from a very large magnitude of plastic strains developed during restrained heating. In this example, this value is reached when the column has been heated up to 600°C and has an infinite degree of restraint.
- The column heated up to 800°C with a degree of restraint R = 0.20 also reaches yielding in tension after the fire. However, the residual force is lower than $F_{y,20°C}$ because part of the strength has not been recovered, in accordance with the assumption in the material model. Note that, in this situation, the assumption that steel yield strength is not fully recovered when heated beyond 600°C becomes favorable for the structure (it reduces the predicted value of the tensile force) whereas common sense would lead to believe that a loss of residual strength is detrimental. It should of course be carefully ensured that this assumption is valid for the type of steel used.

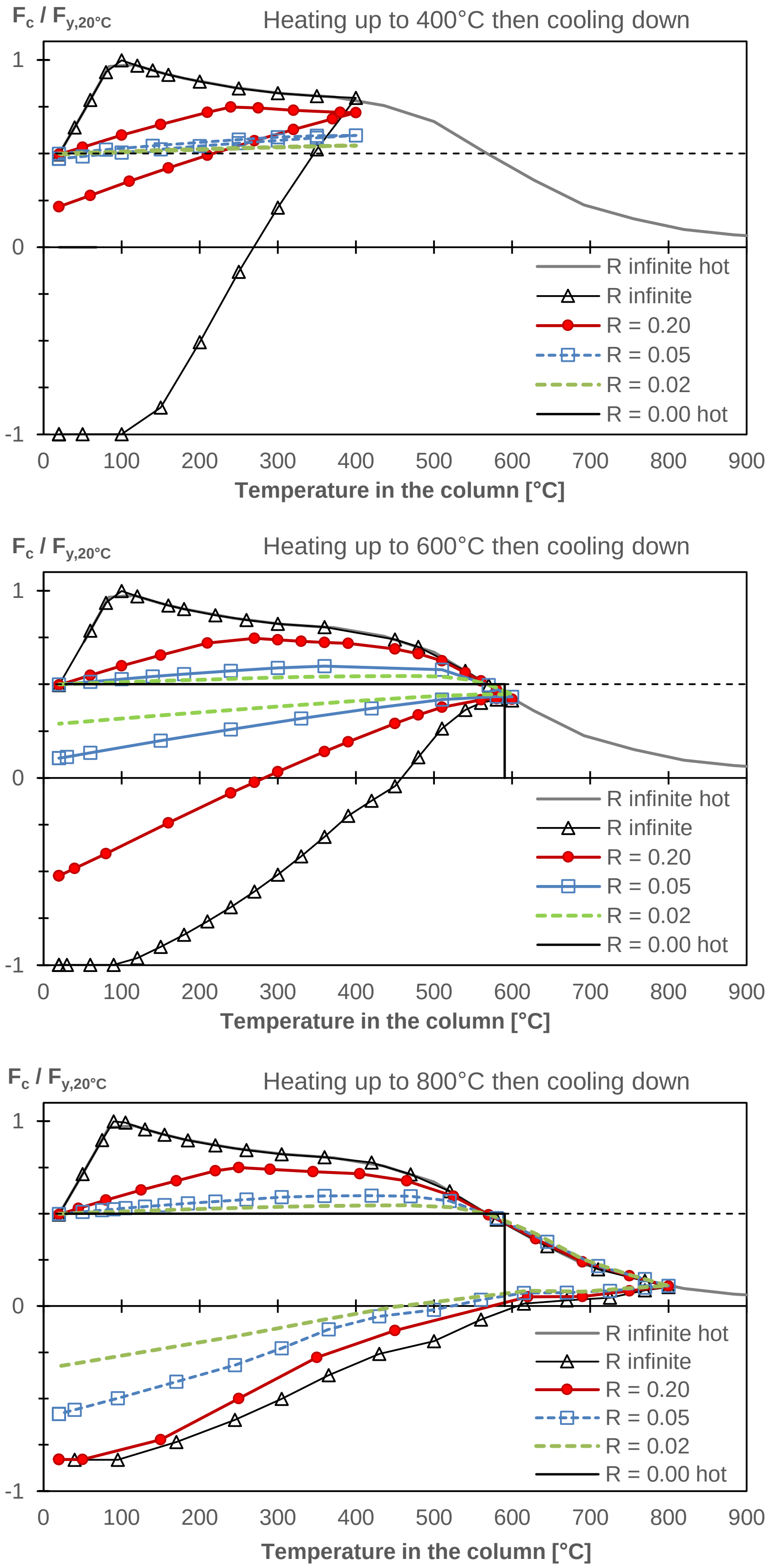

Figure 4. Evolution of the axial load in a restrained column subjected to natural fire: effect of the relative stiffness (R = K_s / K_c) for different maximum steel temperature.

Figure 5 shows the evolution of the axial force in the column as a function of the initial load ratio (LR) in the column, for a relative stiffness of 0.10 and a maximum steel temperature of 800°C. It is observed that higher levels of initial load leads to higher tensile forces in the column after the heating-cooling sequence.

Evidently, in a real structure, the relative stiffness results from the design. Furthermore as stated above, it comes from structural and non-structural elements. Hence the value will differ for every building. The parametric analyses presented in this section provide an indication of the axial load variations in a column for a range of relative stiffness, temperature history and initial load ratio. In Section 5, a prototype MRF structure is studied for which the axial relative stiffness for a ground floor column equals 0.116.

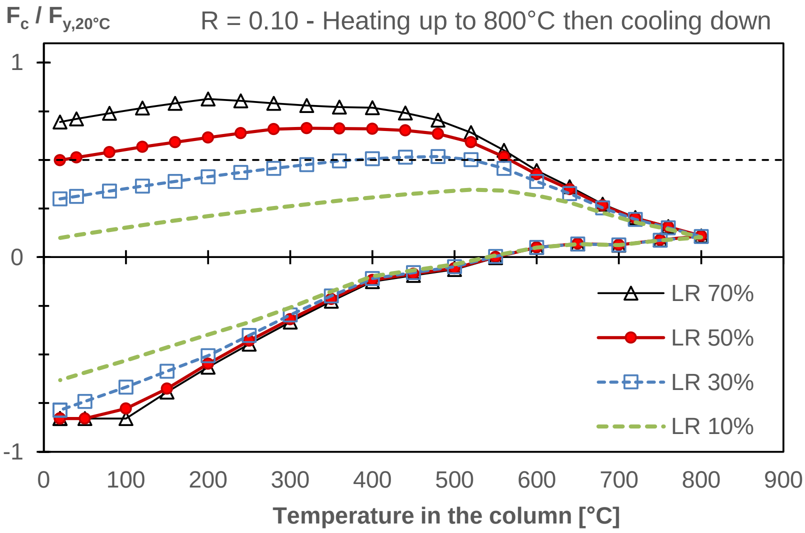

Figure 5. Evolution of the axial load in a restrained column subjected to natural fire: effect of the initial load ratio (LR = F_c / $F_{y,20°C}$).

4. SIMULATION OF AN EXPERIMENTAL TEST ON A REDUCED SCALE STEEL FRAME UNDER LOCALIZED FIRE

4.1 Simulation of the test

In this Section, a numerical simulation of a test conducted in China is performed to support the observations of the previous sections and to validate the numerical modeling approach. The analysis is made using the nonlinear finite element software SAFIR. This software allows modelling the behavior of structures in fire, taking into account material and geometrical non linearities, the thermal elongation, as well as the reduction of strength and stiffness of the materials at elevated temperature [32]. A one-way coupling is assumed from the heat transfer analysis to the thermo-mechanical analysis [43]. The presented simulations make use of two dimensional beam finite elements with three nodes and seven degrees-of-freedom (including the non-linear part of the axial displacement). The non-linear part of the strain component is averaged on the length of the element in order to eliminate shear locking. The beam section is discretized in fibers which are used for the heat transfer analysis and for the mechanical analysis. Longitudinal integration is performed numerically using 2 points of Gauss per element. At each point of Gauss, integration in the section is based on a fiber model. Each fiber in the beam section has its own area, material type, mechanical properties and temperature. The stress-strain material law is solved in every fiber of the section. Therefore, as the stress cannot exceed the temperature-dependent material strength in any fiber, section failure (in the sense of axial load and moment interaction curve) is automatically captured by the software. The effects of thermal gradient, if any, are also accounted for owing to the fiber model. The non-linear material law of Eurocode 3 part 1-2 is adopted to capture yielding and material failure due to excessive strain (a descending branch is considered starting at a strain of 0.15) [42]. The properties of the material law are reduced with temperature according to Eurocode [42]. Buckling is also captured as the software accounts for large displacements and geometrical nonlinearities. Nonlinear implicit dynamic analyses are performed which allows capturing post-critical behavior. The time integration procedure is based on a pure Newton-Raphson scheme, where the stiffness matrix is taken as the tangent matrix, recalculated at every iteration of every time step [32].

Jiang et al. [44] have reported the testing of reduced scale planar moment-resisting steel frames under gravity loads and localized fire attacking one of the columns. The tests brought a particularly valuable contribution as they shed light on the response of the frames during the full course of the fire including in the column post-buckling stage and in the cooling phase.

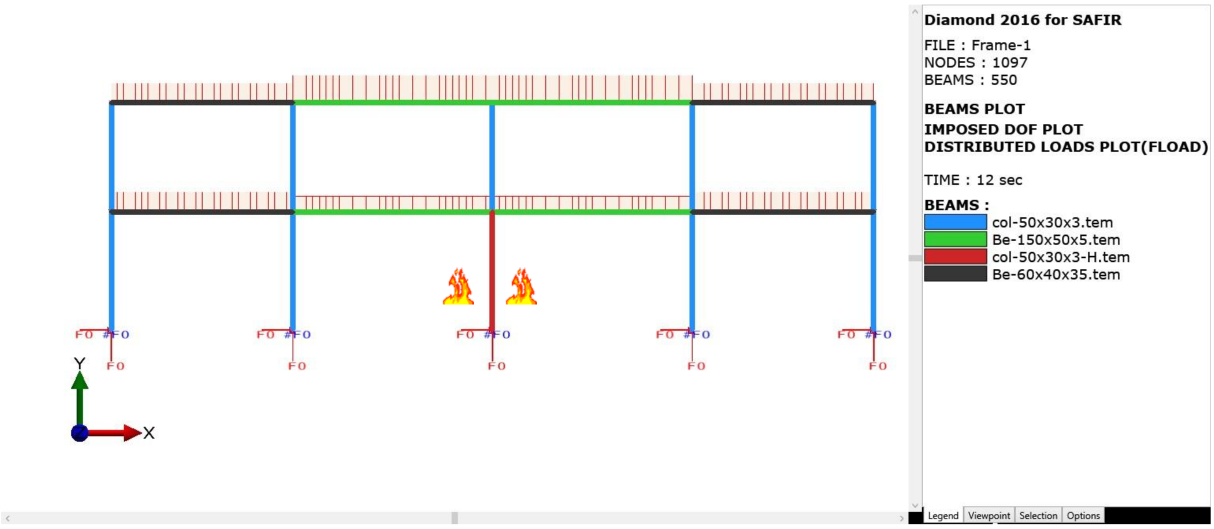

Figure 6. Numerical model of the test Frame 1.

The test frames comprised four bays and two stories, see Figure 6. The two central bays span 2.2 m each while the two side bays span 2.0 m each. The height of the first and second story are 1.3 m and 1.2 m, respectively. The steel members have rectangular tubular sections, with dimensions (in mm) equal to 50x30x3 for the columns and 60x40x3.5 for the beams. Three frames were tested (in Frame 1, the middle bay beams sections are increased to 150x50x5). The central column at the first floor was heated using an electrical furnace. The degree of axial restraint to the heated column, R, varied between 0.023 (Frame 1) and 0.004 (Frames 2-3). The initial axial force in the heated column, F_c, varied between 9.8 kN (Frame 1), 14.0 kN (Frame 2) and 20.9 kN (Frame 3). Temperatures, displacements and strains were measured during the tests. The furnace gas temperature was increased up to 950°C, 829°C and 735°C for Frame 1, 2 and 3, respectively, before turning off the furnace and letting the column cool down. The recorded temperatures in the heated columns were almost uniform in the section. All details of the tests are given in [44-45].

The authors of the tests report that global buckling of the heated column occurred for the three tests. However, the observed behavior was different for Frame 1 compared with Frames 2 and 3. In Frame 1, failure occurred in a quasi-static way with a gradual increase of the vertical deflection and a final shortening which was comparatively small. Conversely, in Frames 2 and 3, failure occurred suddenly; dynamic effects were significant in particular for the most loaded frame (Frame 3). This difference in failure behavior is caused by the different degrees of axial restraint. Frame 1 is stiffer and therefore is able to redistribute the loads from the heated column to the surrounding structure in a gradual way. For this reason, the test on Frame 1 was selected for the numerical simulation.

The test on Frame 1 has been modeled numerically by the finite element method. A transient thermal analysis of the column section was first conducted, using the furnace temperature as boundary condition on the four faces of the column. The steel model for thermal analysis was adopted from Eurocode (see Section 3.4 in [42]). It accounts for the variation in specific heat and thermal conductivity with temperature. The temperature evolution predicted by the model matches very well the test results, see Figure 7. Figure 7 also shows the discretization of the column cross-section in fibers that is used for the thermal analysis. The temperature distribution is plotted at 1800 sec. This distribution is almost uniform due to the boundary conditions (column exposed on four faces) and the low thickness of the plates of the column (3 mm).

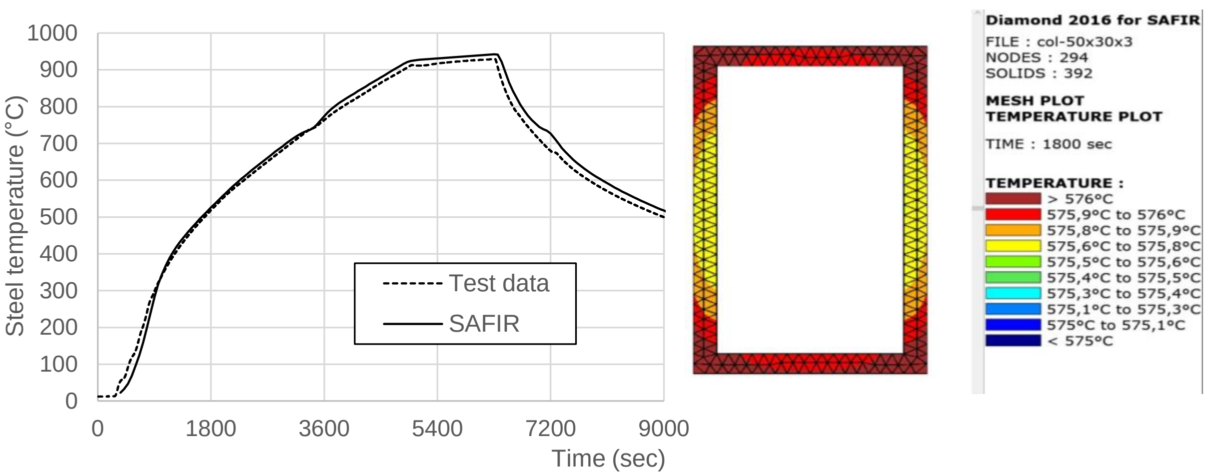

Figure 7. Thermal analysis of the test: Comparison of measured and computed temperatures in the heated column.

Then, a 2D structural analysis was performed using 550 beam FE and the modeling assumptions described above. The material properties reported in [44] were used in the simulation (steel yield strength at ambient temperature equal to 310.8 MPa for the 150x50x5 beam, 290.0 MPa for the 60x40x3.5 beam, and 360.8 MPa for the columns) and reduced with temperature according to Eurocode (see Section 3.3.2 in [42]). Figure 8 shows the evolution of the vertical displacement at the top of the central column as a function of time. The end of the heating phase is indicated by a vertical dashed line. The results indicate that the numerical model can reproduce the experimental behavior during the different stages of the fire. The trend in the evolution of the vertical displacement at the top of the heated column is well captured by the numerical model. The model predicts a somewhat larger expansion of the column, with discrepancies in the values of the predicted displacements of 19%, 31% and 6% at the time of maximum expansion, end of heating, and end of simulation, respectively (calculated as $(d_{model}-d_{test})/d_{test}$, in absolute value). Such discrepancies could possibly be due to a non-uniform heating of the column over its length (near the extremities) during the test, leading to an overall smaller thermal expansion. Most importantly, the model captures the quasi-static buckling of the heated column (linked to the load redistribution to the rest of the structure) and the residual deflection is quite accurately predicted. Since dynamic analyses are run, the simulation can continue beyond the failure of one component to analyze the global structural behavior. This validates the selection of a thermal-mechanical finite element software with a dynamic solver [32] to study progressive collapse of steel frames under fire scenario.

It can be seen in Figure 8 that, during the cooling phase, the frame top displacement continues to move downward. As temperature reduces, the fire-exposed column exhibits thermal contraction. As a result, further load redistribution occurs during the cooling phase, from the fire-exposed column to the adjacent columns.

Figure 9 plots the axial forces in the members before the fire starts and at the end of the simulation, as computed numerically. After the fire, the central column is in tension. The middle bay beams at the first story are in tension as well, having developed catenary action. These observations confirm that the cooling phase may lead to further load redistribution, and hence damage, to members adjacent to a fire-exposed column. Note that for this structure, the amount of tensile force developing in the fire-exposed column is relatively small. This is due to the small degree of axial restraint (0.023). Hence, the redistributed load does not significantly exceed the initial axial load in the central column. As a result, in this case, application of the common alternative load path method would be acceptable to check progressive collapse triggered by fire-induced column loss.

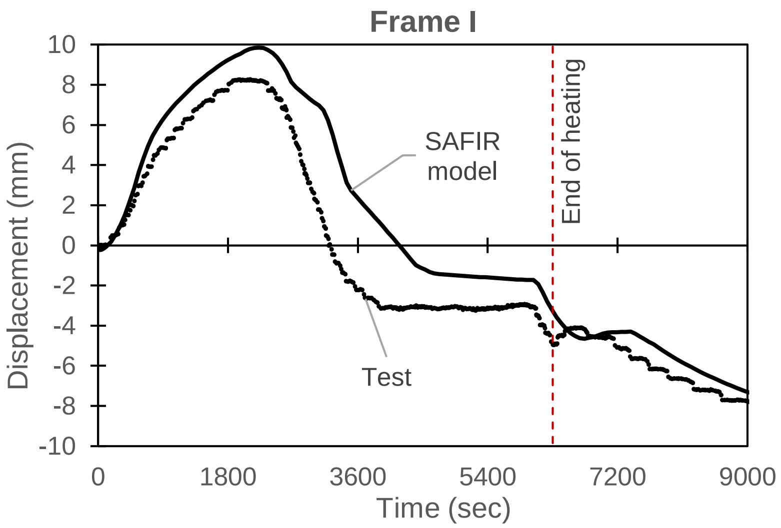

Figure 8. Mechanical analysis of the test: Comparison of measured and computed vertical deflections at the top of Frame 1.

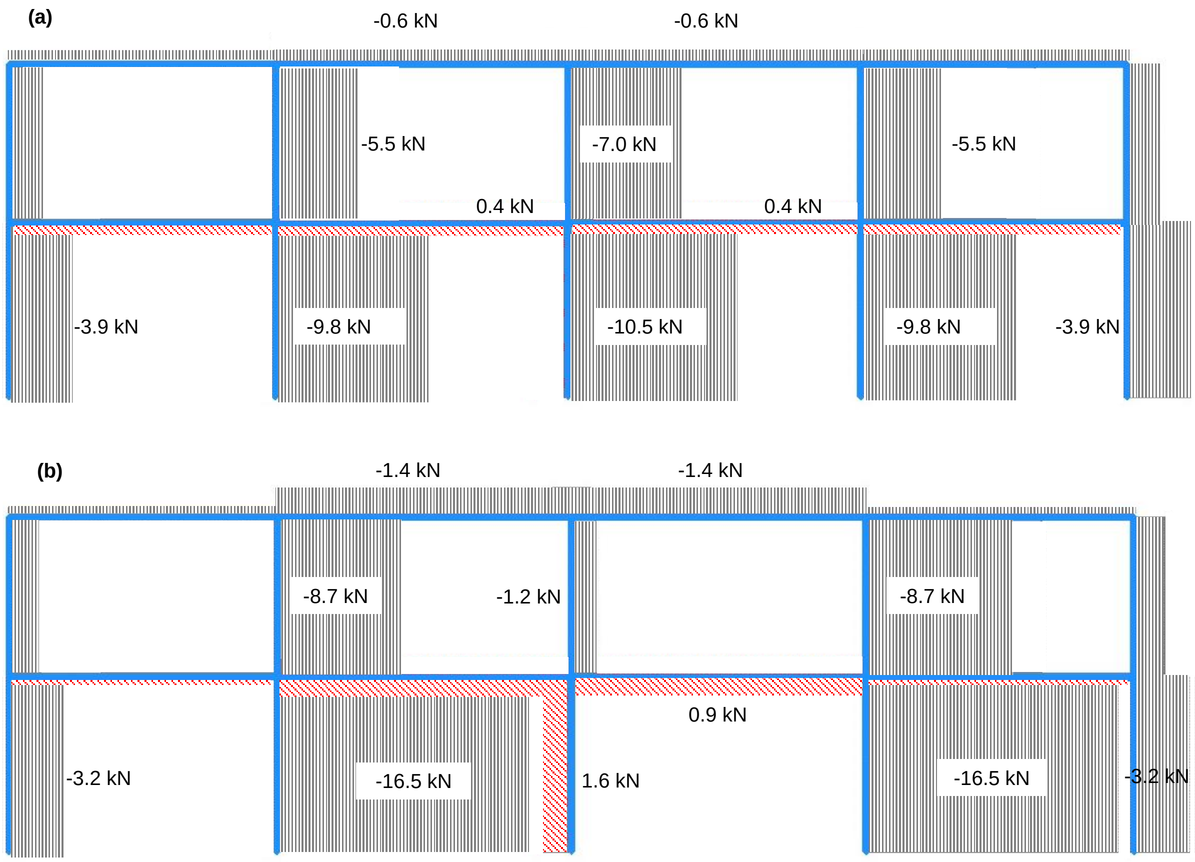

Figure 9. Axial forces in the Frame 1 members (a) before the fire and (b) at the end of the fire. Compressive forces are negative.

4.2 Parametric analysis

The structural frame depicted in Figure 6 is then analyzed under different fire exposures and mechanical loading. Some design changes are made to the tested frame of Section 4.1 in order to consider a configuration closest to a real design. First, the steel members are protected with thermal insulation. A sprayed fire resistive material of 12.7 mm thickness is assumed. The thermal properties of the insulation are based on a study by NIST [46] as implemented in [47]. These properties (density, thermal conductivity, specific heat) vary with temperature. Second, the beam profiles are upgraded to 250x100x5 (mm) rectangular tubular sections with a steel yield strength of 310.8 MPa. The objective of this profile upgrade is to increase the degree of restraining stiffness to the heated column to simulate the effect of more stories. This leads to a degree of axial restraint for the heated column of R = 0.16. From Section 3, it is clear that a higher R leads to more load redistributions during heating-cooling and therefore is more relevant to this study.

The parametric fire model from Eurocode is adopted. The value of the factor Γ in this model was taken as 1.0 here, which makes the heating phase of the time-temperature curve of this natural fire model approximate the standard ISO curve. The only varying parameter is the time that corresponds to the duration of the heating phase, noted DHP [30]. Figure 10 plots the fire curves corresponding to different values of DHP, as well as the numerical model of the protected column for the thermal analysis.

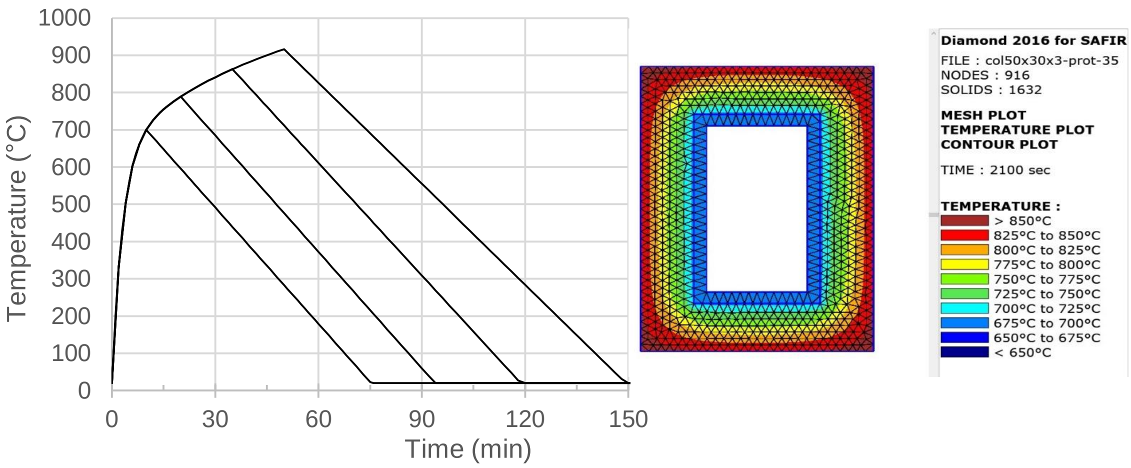

Figure 10. Time-temperature curves based on the Eurocode parametric fire with DHP of 10, 20, 35 and 50 min, and thermal analysis of the protected section.

As shown in Figure 6, only the central ground floor column is subjected to fire. The fires of Figure 10 are applied. The beams of the frame are uniformly loaded with the applied mechanical loads being selected to generate initial axial loads in the column varying between 30% and 50% of the column sectional capacity at ambient temperature. The ratio of the initial axial load in the column and the ambient temperature yielding capacity is written LR. The results are given in Table 1. The spread of failure from the fire-exposed column to the rest of the structure depends on the duration of the fire and the initial applied load ratio. For the more severe cases, a progressive collapse takes place leading to global collapse of the structure during the cooling phase. In other cases, the structure remains stable but a residual vertical deflection is observed after the event, the value of which is given in the table.

DHP / LR	0.3	0.4	0.5
10 min	1 column - 4 mm	1 column - 6 mm	1 column - 8 mm
20 min	1 column - 9 mm	1 column - 10 mm	2 columns - 15 mm
35 min	1 column - 10 mm	2 columns - 17 mm	Global collapse
50 min	1 column - 11 mm	2 columns - 18 mm	Global collapse

Table 1. Parametric analysis of the steel frame with one column subjected to heating-cooling: number of columns failing and residual vertical deflection at the end of the fire.

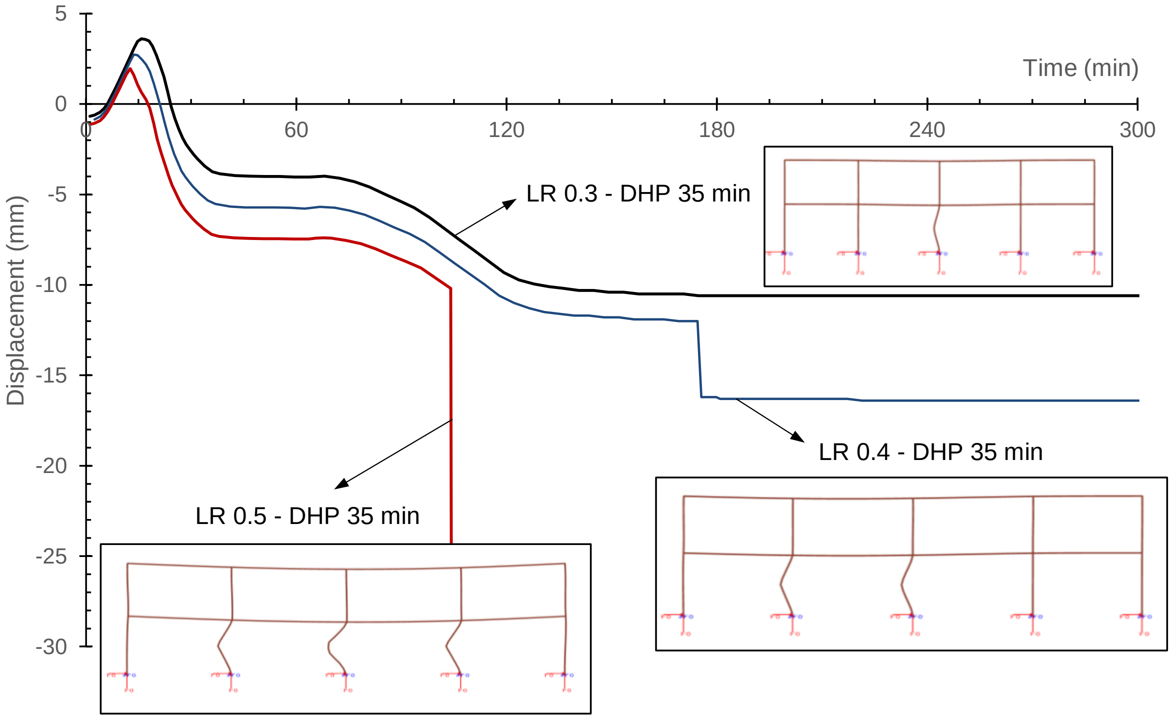

Figure 11. Evolution of the vertical displacement at the top of the central column for a fire with DHP of 35 min and different initial applied loads.

The results for a DHP of 35 min are plotted in Figure 11. Depending on the initial applied load, three different behaviors are observed. The first behavior (LR = 0.3) is the one already described and observed experimentally in Section 4.1. The fire-exposed column shifts from compression to tension but the adjacent columns are able to withstand the additional transferred loads. At the end of the fire, a residual vertical deflection is observed as the exposed column has shortened. The stability was not an issue, but the refurbishment after the event may be an issue given the residual deflection. This behavior occurs for relatively short fires and/or low applied load (see Table 1, results with "1 column"). In the second behavior (LR = 0.4), the failure spreads to one adjacent column. The adjacent column is not heated, but it fails due to an excess of redistributed load from the fire-exposed column that is in tension. Yet, the structure is still able to find an alternative load path and a new static equilibrium, despite the buckling of two columns. This second behavior occurs for intermediate values of fire duration and applied load (see Table 1, results with “2 columns)). It can be seen that there is a sudden change in load path when the second column buckles, as shown by the vertical asymptote in the time-displacement curve. Activation of the new equilibrium at that moment is accompanied by a considerable and sudden increase in tensile force in the beams, typical of a catenary action. The third behavior is observed if the applied load is increased from 0.4 to 0.5, for the same fire with DHP of 35 min. In that case, the two adjacent columns fail simultaneously, and the numerical simulation is unable to find a new equilibrium. Hence, the initial local failure of the fire-exposed column has spread to the two adjacent vertical members. This case is indicated as “global collapse” in Table 1. This is a progressive collapse during the cooling phase of a localized fire.

It must be stressed that the spread of failure occurs during or after the cooling phase. For the fire with DHP of 35 min, the cooling phase starts at 35 min and ends at 120 min. The failure of adjacent columns arise after 105 min for the case with LR = 0.5 and after 175 min for the case with LR = 0.4. This demonstrates the crucial need to consider the effects of heating-cooling sequences on structures, rather than relying on standard fire curves.

5. FINITE ELEMENT ANALYSIS OF A 20-STORY STEEL MRF

This Section analyzes the response of a multi-story MRF building subjected to localized fire attacking one perimeter column at ground level. The objective is to observe the effects of the load redistribution mechanisms in a real structure designed according to the codes in application. Nonlinear finite element analysis with large displacements is used.

The considered building prototype consists of a twenty-story steel building designed in accordance with the FEMA/SAC project [48], for the Boston model building. The building is 30.48 m by 36.58 m in plan, consisting of five bays of 6.096 m (20 ft) in width and six bays of 6.096 m (20 ft) in length. The total height of the building is 80.76 m, divided between a first floor of 5.486 m (18 ft) high and the 19 other floors of 3.962 m (13 ft) high. The structure is composed of perimeter moment-resisting frames to ensure the in-plane stability of the building. The design is controlled by wind loads.

Here, the analysis focuses on the planar response of a five-bay perimeter moment-resisting frame in case of localized fire attacking one of the column. The sections of the beams and columns of the MRF are given in Table 2. For the sake of limiting the number of sections in the model, one single column type is used for a given story (i.e. the section is not reduced for exterior columns). The column bases are considered as fixed. Steel yield strength is 345 MPa. This strength is partly irrecoverable: a loss of 0.3 MPa/K is assumed when steel has been heated beyond 600°C. Steel thermal strains are fully reversible.

Level	Column	Girder
1	W36x485	W33x141
2	W36x485	W33x141
3	W36x485	W33x141
4	W36x393	W33x141
5	W36x393	W33x141
6	W36x359	W24x131
7	W36x359	W24x131
8	W36x359	W24x131
9	W36x359	W24x131
10	W36x300	W24x131
11	W36x300	W24x131
12	W36x300	W24x117
13	W36x300	W24x117
14	W36x280	W24x104
15	W36x280	W24x104
16	W36x210	W24x104
17	W36x210	W21x101
18	W36x150	W18x86
19	W36x150	W18x76
20	W24x131	W12x53

Table 2. Sections of the perimeter moment-resisting frame members [48].

The fire is assumed to attack the third column from the left on the first story. As discussed in Section 2, a simplified scenario is assumed where the fire attacks a single column. This hazard scenario results, for instance, from a localized fire occurring in the immediate vicinity of the column [33-34]. The adopted thermal approach is similar to the one used in [26]. It is noted here that the purpose of this study is to assess the effects of load redistributions within a system due to fire-induced damage to an element, in particular during the cooling phase; it is not to conduct a structural fire design of the prototype building. Therefore, the analysis of other fire scenarios, potentially more severe with respect to overall fire response (such as post-flashover or traveling fires), is outside the scope of the study. Here, the variables of interest are the internal forces in the structure and the temperature in the fire-exposed column; time is irrelevant. Therefore, for the sake of convenience, transient thermal analyses are replaced by a user-defined temperature evolution in the column. Similar conclusions would be reached if the temperatures were obtained from a thermal analysis, e.g. considering a natural fire curve applied to insulated steel members; the results could simply be expressed in the time domain. It is clear that, in a real design, multi-story buildings such as the one under study have thermal protection on the steel profiles. But, as long as the results are expressed as a function of the temperature reached in the steel profile, the mechanism discussed here does not depend on the specific fire scenario or the presence of thermal protection. Justification of the analysis in the temperature domain for steel structures can be found in [49-51]. Note that this approach assumes that the stress-strain material behavior does not depend on time (in line with Eurocode, creep is not given explicit consideration); it also neglects the effects of thermal gradient, which in some cases may be significant [52]. The temperature evolution considered in the analysis is a temperature increase of the steel profile up to 800°C, with a spatially uniform field in the section, followed by a cooling down to 20°C.

The structural analysis is performed using the finite element method [32] and the general assumptions described in the previous Section. The simulation uses 2200 three-noded, two-dimensional beam elements. The cross-section of the beams is discretized in fibers where the properties are evaluated; an integration on the section is then performed to get the stiffness and internal forces. The analysis takes into account geometrical and material non-linearities, including large deflections. Global and members instabilities (buckling) are also accounted for. The effects of initial geometric imperfections are introduced in the model through systems of equivalent horizontal forces in accordance with Eurocode 3 part 5.3 [53]. Thermal expansion is included in the analysis and therefore fire-induced forces are considered in the structure. Columns are continuous and rigid connections are assumed with the beams (moment-resisting frame). As a result, the fire-exposed column is axially and rotationally restrained. Connections are not represented in the model; hence it is implicitly assumed that the connections are able to transfer the forces without failure. The slab contribution is not considered. The structural model is shown in Figure 12.

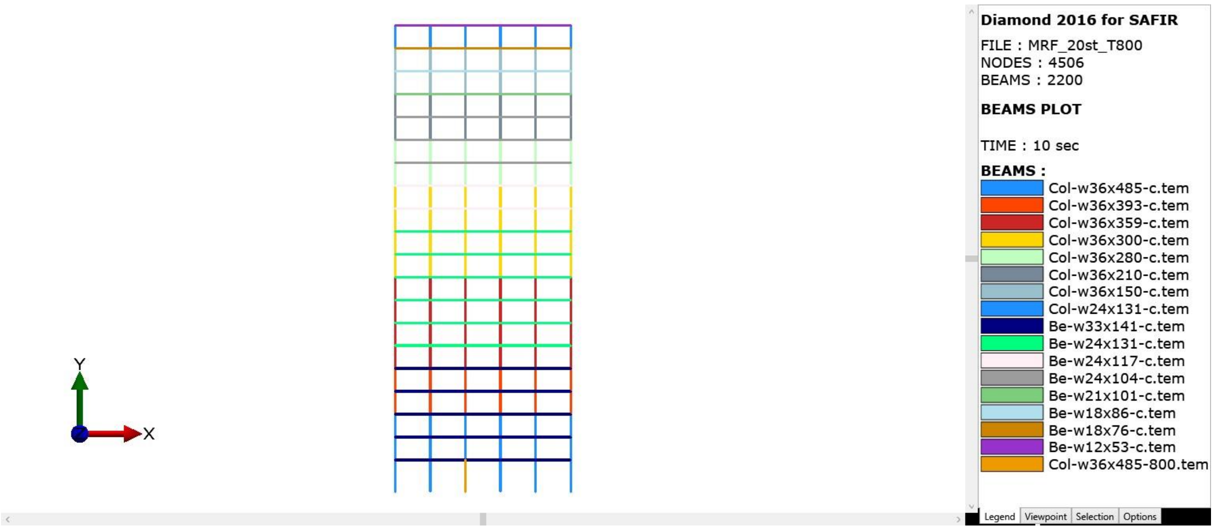

Figure 12. Numerical model of the 20-story steel moment-resisting frame.

The relative stiffness for the axial degree of freedom of the heated column $R = K_s / K_c$ is equal to 11.6%. This evaluation neglects the contribution of non-structural components. The column capacity at ambient temperature, $F_{y,20°C}$, is equal to 31580 kN, as evaluated by analyzing numerically the whole structure with the column loaded until failure. The following floor load distribution is assumed: floor dead load 4.60 kN/m² (96 psf); roof dead load 3.97 kN/m² (83 psf); reduced live load per floor and for roof 0.96 kN/m² (20 psf). In the fire situation, factors of 1.05 and 0.24 are applied to the dead loads and live loads, respectively. This results in a factored distributed load of 5.06 kN/m² on each floor and 4.40 kN/m² on the roof. For the fire-exposed column, the considered load distribution in the fire situation leads to an initial axial force of 1805 kN, or approximately 6% of the capacity at ambient temperature $F_{y,20°C}$. This low load ratio is due to the fact that the perimeter columns are designed to withstand a combination of vertical (gravity) loads and horizontal (wind) loads; therefore the sections of the perimeter columns are significantly stronger than the ones of the interior columns. Besides, the load factors applied in the fire situation are lower than the design load factors.

First, the structure is loaded with the distributed loads. Then, the fire-exposed column is heated up to 800°C, and subsequently cooled down. Figure 13 shows the axial forces and bending moments in the beams and columns at the end of the simulation, i.e. after the temperature has been brought back to 20°C in the column. The fire-exposed column has developed a significant tensile force. By equilibrium, the adjacent columns are subject to increased compressive force. Catenary action and arch effect can be seen in the beams of the various floors. Large positive bending moments develop in the beams due to pulling of the column.

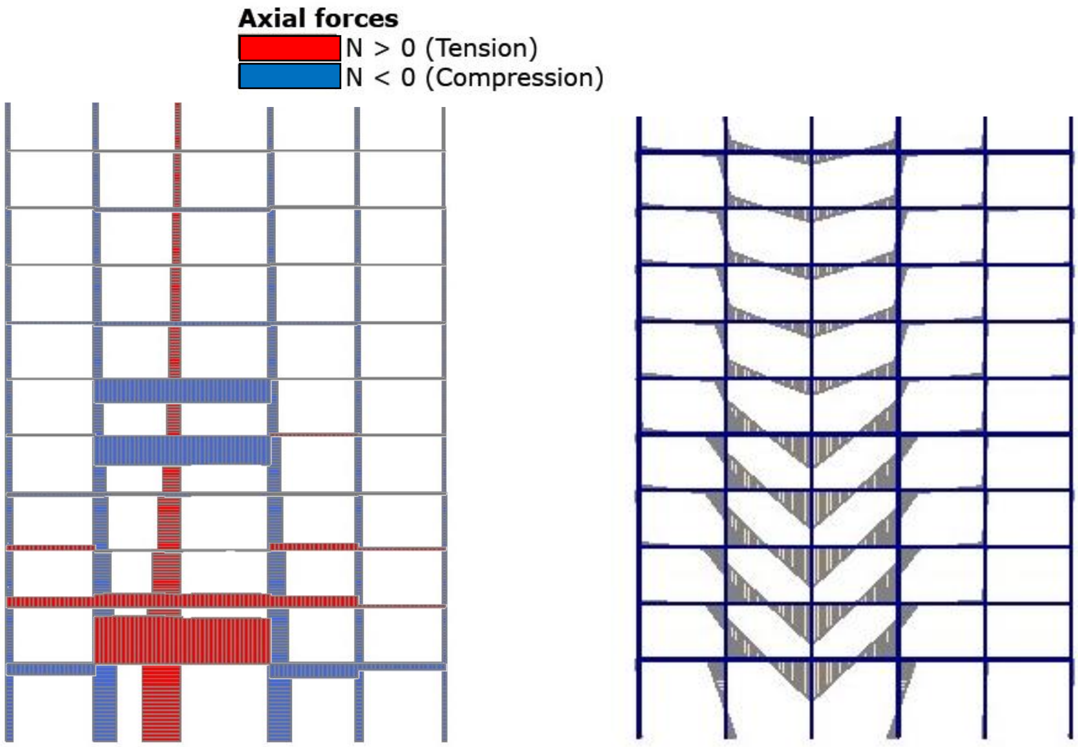

Figure 13. Axial force and bending moment diagrams in the beams and columns at the end of the heating-cooling sequence. Note that, for readability, only the 10 lowest stories are shown, and the scale for axial forces is different for the beams and the columns.

Figure 14 shows the evolution of the axial force (relative to capacity at ambient temperature) in the fire-exposed column and in the adjacent right column. The relative axial force starts at 6% (or 1805 kN) for both columns. When the fire occurs, this force increases in the heated column due to restrained thermal expansion, up to a maximum value of 42% (or 13370 kN) which occurs for a steel temperature of 502°C. In the meantime, the adjacent column is unloaded and even shifts to tension. Then, the axial force in the fire-exposed column drops, first due to a reduction of the properties in heating and then due to the recovery of thermal strains in cooling. At the end of the cooling phase, when the steel temperature is back to 20°C, the tensile force in the fire-exposed column equals 43% of the maximum capacity at ambient temperature. In other words, the column is subjected to a tensile force 7 times larger than the initial compressive force. (Note: it is assumed that the column foundation can withstand the tension. This might not be the case, since not all foundation systems are capable of transferring tension to the ground. If the foundations were not able to transfer tensile forces, the column shortening during cooling would not be restrained and the load transferred to the surrounding structure would be reduced. However, pile foundations for instance do transfer tensile forces, and these systems are common for very high loads and/or poor quality soil.) By equilibrium, the difference has to be supported by the surrounding columns; as a matter of fact, the axial force in the adjacent right column has increased from 6% (1805 kN) to 26% (8147 kN) after the fire event. This is an increase in compressive axial load of 6342 kN for this adjacent column. This mechanism of amplification of the load transfer from the column affected by the hazard to adjacent members, with pulling due to thermal effects, is specific to fire scenario.

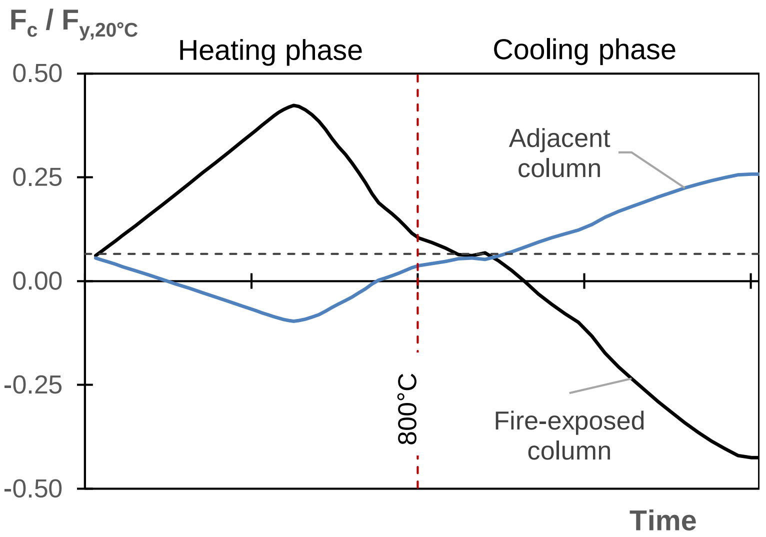

Figure 14. Evolution of the axial force for a fire-exposed column heated up to 800°C and then cooled down.

This result is shown in another manner in Figure 15. The figure illustrates the redistribution of axial forces in the ground level columns as a percentage of the initial axial force in the fire-exposed column. The area of the disks is proportional to the change in axial load in each column. The plots are given at two different times: when the axial force in the heated column drops to zero, and at the end of the cooling phase. It appears clearly that the development of tensile forces in the attacked column during cooling leads to load redistributions that largely exceed the amount of axial force initially carried by the column.

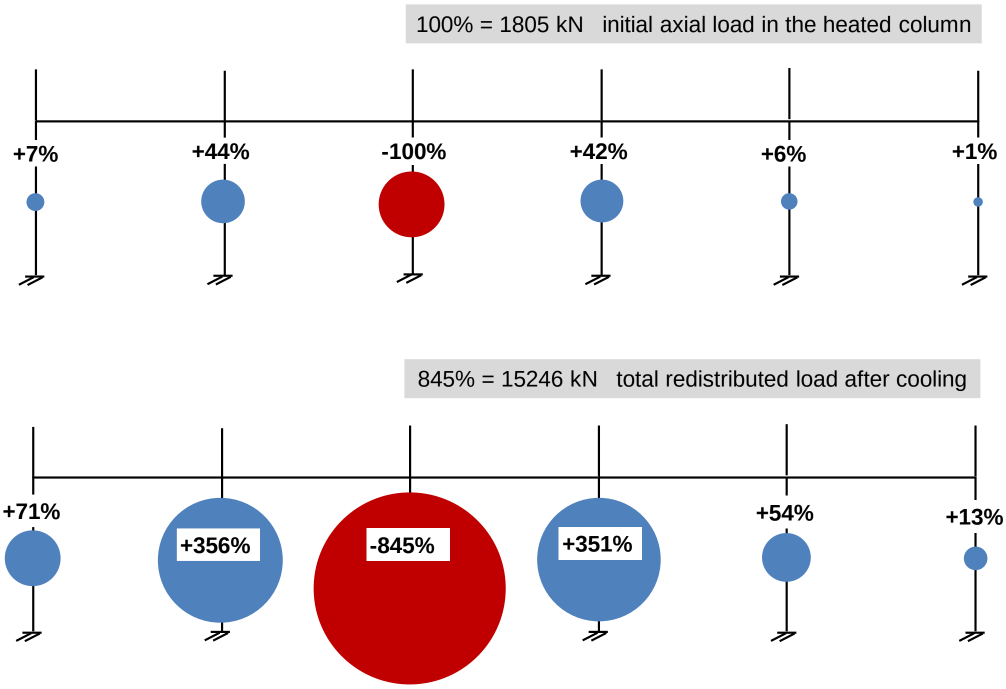

Figure 15. Axial load redistributions in the ground floor columns due to the localized fire; at zero-force in the heated column (top) and at the end of the cooling down phase (bottom).

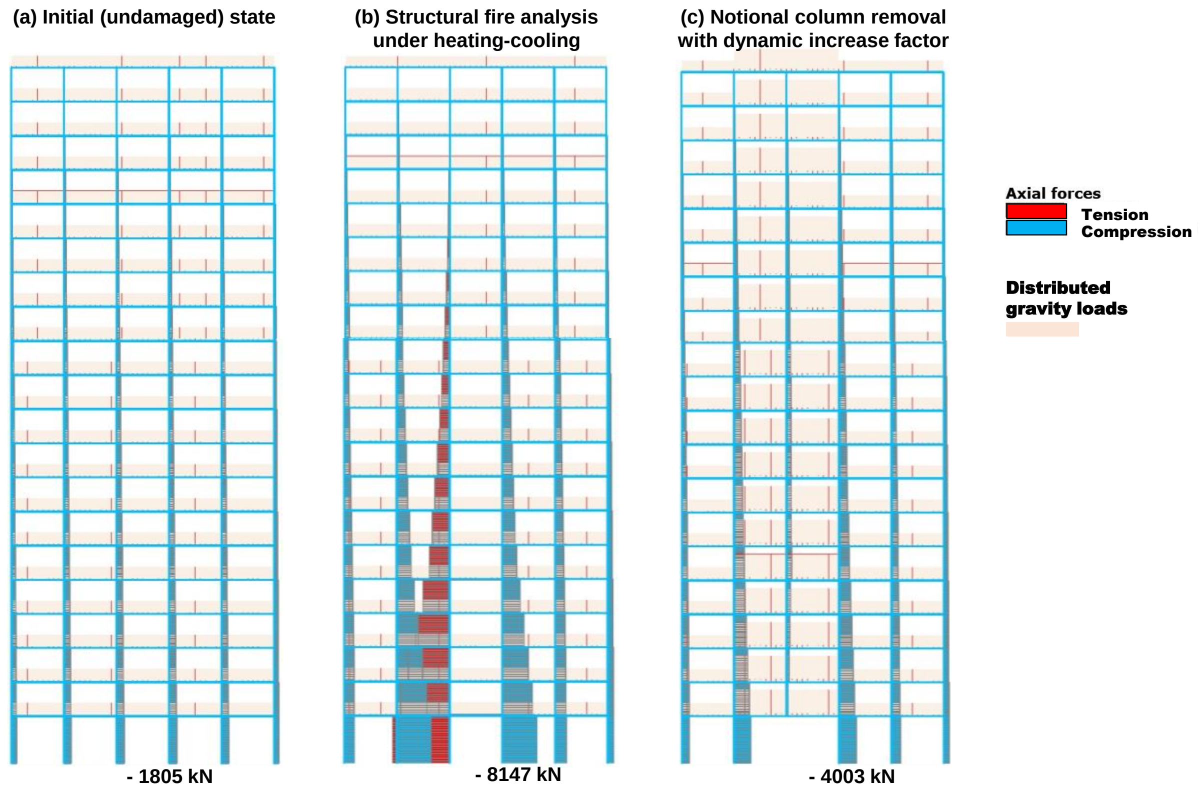

Figure 16. Axial forces in the columns of the 20-story MRF under (a) initial gravity loading in the undamaged state; (b) fire-induced collapse of a ground floor column as analyzed by a nonlinear thermo-mechanical analysis; and (c) alternative load path method with notional column removal and use of a dynamic increase factor of 2 for the gravity loads. The indicated value is for the ground floor column adjacent to the lost column.

The load redistribution evaluated for the fire-induced collapse of the column (as shown in Figure 15) is then compared with the one that would result from the application of the "common" alternative load path method, as developed in a threat-independent approach. The UFC guideline is followed with the adoption of a dynamic increase factor for the nonlinear static analysis [3]. To account for the most severe case, a factor of 2 is selected. This increase factor is applied on the gravity loads to the bays immediately adjacent to the removed column and at all floors above the removed column. Figure 16 compares the axial forces that develop in the columns as evaluated for a fire-induced collapse (Figure 16b) with the ones obtained by the "common" alternative load path method (Figure 16c). Before the loss of the column, the initial compressive force in the adjacent right column is 1805 kN. After the fire, the compressive force in the adjacent column has increased to 8147 kN. However, the "common" alternative load path method predicts a compressive force of 4003 kN after the loss of the column. Therefore, it appears clearly that the thermal effects lead to load redistributions that are much more severe than the ones considered by a scenario of notional column removal.

For the studied MRF, the load redistribution evaluated for a fire-induced column loss did not lead to a progressive collapse of the frame, owing to the low initial level of applied gravity loads. The global stability of the structure was not impacted even though significant load redistributions occur. However, it shows how a fire can lead to overload of an adjacent column by a value (here, +20% of $F_{y,20°C}$) that is significantly larger than the load initially supported by the fire-exposed column (6% of $F_{y,20°C}$). This raises concern about other configurations in which a combination exists of high applied gravity loads and high restraint (i.e. stiffness of the surrounding).

The numerical model focused on the 2D behavior of the frame. In such a building, 3D effects could play a role in the alternative load path after an exceptional event affecting a column. While the concrete slab typically brings additional robustness, it is possible that part of the new vertical forces generated by the fire would be distributed to the interior columns, which are subjected to a higher load ratio (respective to their capacity). Therefore, the studied mechanism could threaten the stability of interior columns; this assumption remains yet to be verified by additional studies.

The results also illustrate the severity of heating-cooling sequences for connections. Although connections were out of the scope of the study, and are not represented in the numerical model, the forces in the beam connected to the fire-exposed column can be plotted over time, to get an insight into the forces that have to go through the connections. Figure 17 plots the evolution of the axial and shear forces and bending moment in the integration point of the beam left of the fire-exposed column that is closest to the column. At ambient temperature, a shear force of 44 kN and a bending moment of 46 kN.m develop under applied gravity loads. The fire then leads to enormous variations of the forces in the beam which, in the real structure, need to be accommodated by the connections. Under these forces, connections may fail, limiting the amount of force transferred to the rest of the structure. Hence, in a progressive collapse analysis, fire loading may also be a severe scenario with respect to the design of connections and this should be further investigated. Numerical procedures for incorporating explicitly the connections in global finite element models of moment resisting frames have been proposed at ambient temperature [54].

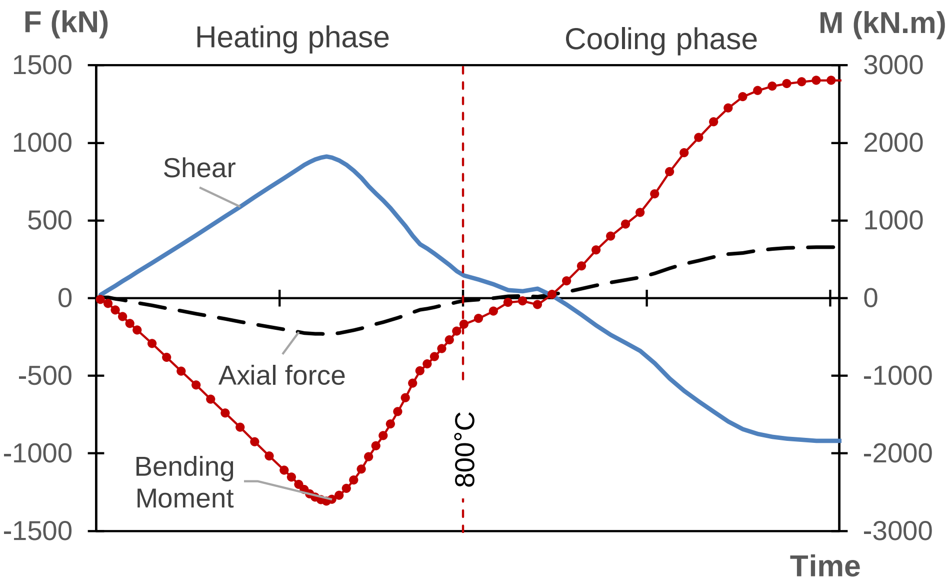

Figure 17. Evolution of forces in the beam connected to the fire-exposed column. A positive axial force denotes tension. A positive (sagging) moment is bending the beam with tension on the bottom.

Finally, the simulation highlights the fact that a fire event significantly modifies the stress state in a structure. This has implications on the assessment of the residual structural reliability after a fire. A fire event that has not led to structural collapse may nevertheless have led to higher demand over capacity ratios in adjacent columns, which impacts the safety of the structure after the event.

6. CONCLUSIONS

This paper has described the load redistribution mechanisms that are observed when a fire affects a column that is part of a structural frame. The mechanisms eventually result in tension building up in the fire-exposed column and overloading the adjacent columns in compression, which could lead to failure of members not directly affected by the fire and trigger a progressive collapse. This suggests that fire scenarios, in particular localized fires affecting part of a structure while the remainder remains cold, lead to specific detrimental effects that are not observed with other exceptional events causing local damage in structures.

The simulation of a structural fire test conducted in China has demonstrated that this behavior is observed experimentally. It also confirmed the suitability of numerical modeling to investigate the behavior of structural systems under real fires. In particular, the software SAFIR is able to capture the response during the heating and cooling phase, including after local failure owing to the dynamic solver.

Parametric numerical analyses have highlighted the key role of the maximum temperature reached in the element and of the relative stiffness between the heated element and the remainder of the structure. The first parameter depends on the fire event; more severe fires cause higher temperatures which in turn lead to higher residual tensile forces in the exposed element. The second parameter depends on the structure; higher relative stiffness leads to higher restraint against thermal strains which eventually lead to higher residual tensile forces in the element. Stiffness is usually considered as favorable as regards robustness as it allows redistributing forces between elements; yet in case of fire, excessive stiffness leads to very high restraint forces. Therefore, hypotheses that are usually considered as conservative, such as neglecting the hidden stiffness due to non-structural building elements, might need to be reconsidered in progressive collapse analyses following a fire scenario. In case of high relative stiffness and maximum temperature, the maximum tensile force that can build up in the element equals the yielding capacity of the section, which represents a very large force to cope with for the surrounding structure.

The results have important implications at the modeling level and at the design level. For modeling, they question the validity of an event-independent design scenario for capturing the influence of column failure due to fire loading. Given the development of tensile forces during cooling, sudden column loss and other column removal techniques seem not adequate to study progressive collapse under a fire scenario. In fire-induced column loss, increase load factors for assessing Alternative Path Method using nonlinear static analysis should be linked to thermal effects (and not solely on dynamic effects). For design, they suggest that some recommendations that are generally valid to improve robustness, such as the use of stiffer beams to redistribute loads between columns, might in certain situations have an adverse effect in case of fire loading, because they amplify the magnitude of the restraint forces.

Further works should focus on structures with relatively high degrees of restraint, high applied gravity loads, and exposure to localized fire scenarios, as this combination is likely to lead to the development of significant tensile forces in fire-exposed members, therefore overloading in compression the non-affected members already subjected to a high level of load. The effects of thermal gradients generated by localized fire exposure, the connections and the role of three-dimensional effects in the redistribution of forces should also be further investigated.

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