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PROGRESSIVE COLLAPSE OF STEEL FRAMES SUBJECTED TO PARAMETRIC FIRE CURVES

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ABSTRACT

Extreme events such as terrorist attacks, vehicle impacts, and fires cause local damage to the buildings, which may spread from element to element, inducing progressive collapse. Steel structures are characterised by a high risk of progressive collapse owing to their high vulnerability (*e.g.,* low structural redundancy, joint behaviour), high hazard, and exposure (*e.g.,* they are often employed for large industrial or strategic buildings occupied by a large number of people). Therefore, steel structures should be carefully designed to mitigate the risk of progressive collapse. Despite being a rare event, fire may lead to the progressive collapse of the structure, and this paper investigates the progressive collapse of a seismically designed 9-storey steel moment-resisting frame subjected to different parametric fire scenarios and various loading levels. The response of critical components was validated against experimental data and standard prescriptions to accurately simulate both beam-type (*i.e.,* ductile plastic hinges and catenary action) and column-type (*i.e.,* non-ductile buckling) mechanisms. Various parametric fires and scenarios affecting ground-storey columns were considered. It is shown that the fire affecting one column may induce buckling of the adjacent ones (*i.e.,* column-type mechanism). This is in contrast with the typical considerations on progressive collapse at ambient temperature, which assume beam-type mechanisms. The mechanisms of collapse are described in detail and the influence of the fire parameters and the loading levels are outlined.

Keywords: Progressive collapse; Parametric fires; Steel structures; Risk mitigation

1 INTRODUCTION

Extreme events may cause local damage to building structures, which may spread from element to element, leading to the collapse of the entire structure or a large part of it [1], *i.e.,* progressive collapse. Among these extreme events, short-duration events, like blasts and impacts, cause sudden damage to the structure involving dynamic actions that may severely stress the structure. Conversely, long-duration events, such as fires, damage the structure with slow-developing actions, which can typically be investigated with static analyses. A general simplified threat-independent approach for short-duration events involves static analyses in which the damaged elements are removed and factors to account for the dynamic effects, *i.e.,* Dynamic Increase Factors (DIF), are applied to the gravity loads. However, recent studies have shown that such an approach is not appropriate for fire-induced progressive collapse (long-duration events) [2] and that the structural response needs to be carefully investigated, explicitly simulating the effects of fire. Indeed, the degradation of the mechanical properties with temperature and thermal expansion cannot be accurately accounted for with a notational element removal scenario. Moreover, fire might be critical as progressive collapse may be caused by mechanisms significantly different from those in other situations.

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For instance, columns may lose their lateral restraints due to fire-induced floor collapse, as presumably occurred in the World Trade Center [3], or experience additional tension forces in the cooling phase [2,4].

Steel structures have a higher risk of progressive collapse compared to other building types. Indeed, being risk defined as the convolution of vulnerability, hazard, and exposure, the three components of risk are highly present in steel frames. The vulnerability of steel structures is high as they are usually designed with a low level of redundancy, with sections optimised for specific design actions. The joints may not be able to withstand the additional tie forces. The hazard and the exposure are high as steel structures are often employed for large strategic or industrial buildings occupied by a large number of people or devoted to the provision of fundamental services. In this context, investigating fire-induced progressive collapse is very relevant for steel structures, and extensive studies are necessary to effectively design resilient buildings and propose actions to mitigate the progressive collapse risk.

This paper addresses the problem of fire-induced progressive collapse of steel moment-resisting frame (MRF) by performing an extensive parametric study. A 9-storey steel MRF is considered for case study purposes. Different scenarios involving several parametric fires affecting ground-storey columns were investigated. The intensity of the gravity loads was varied to explore the response of the structure under different loading levels. The case study structure was modelled in OpenSees [5,6], with particular attention to components significantly affecting the response under fire and large inelastic deformations. The response of beams and columns was validated against experimental data and European design standards [7,8] to accurately simulate beam-type and column-type collapse mechanisms. Global and local imperfections were included as well to capture the potential loss of stability of the structure. The governing collapsing mechanisms are described in detail for representative scenarios. In contrast with the typically considered beam-type ductile mechanisms in progressive collapse analyses of steel frames, the investigated structures show a progressive collapse initiated by the buckling of the columns adjacent to the heated one (*i.e.,* column-type mechanism). It is also demonstrated that the seismic design confers to the investigated frame a sufficient capacity to resist progressive collapse when the fire combination is considered, while by increasing the loads by 10% or 20%, progressive collapse occurs depending on the fire scenario. The present study provides significant insights toward the understanding and knowledge of fire-induced progressive collapse of steel MRF. In particular, the influence of each single fire parameter and the loading level are assessed.

2 NUMERICAL STUDY

2.1 Case Study Structure

Figure 1 shows the 9-storey seismically designed MRF considered for case study purposes. This MRF was already examined in previous research focusing on progressive collapse [9,10], and detailed information can be found in Gerasimidis *et al.* [11]. The structure is characterised by an inter-storey height of 3 m and a total height of 27 m. The plane frames consist of 4 bays with 5 m spans, with the steel sections oriented along the major axis. The bay span in the transversal direction was considered equal to 7 m. Beam-column joints were designed as rigid, full-strength welded joints. Steel S235 was used for the steel components (*i.e.,* yield strength f_y = 235 MPa, Young's modulus E = 210000 MPa, and Poisson ratio v = 0.3). Table 1 summarises the steel cross-sections for columns and beams. The seismic design was performed according to the Eurocode recommendations [7,12,13] for a building located in Greece with a horizontal peak ground acceleration equal to 0.16g. The structure was considered unprotected against fire, which generally would not be true for a multi-storey building. However, this aspect does not alter the results, given that steel structures can be assessed in the temperature domain. The focus is not on the time of collapse but on identifying the mechanisms leading to progressive collapse. Thus, the time indicated in the figures must not be considered a measure of assessing the vulnerability of the case study.

Table 1. MRF design. (Adapted from Gerasimidis *et al.* [11])

Storey	1	2	3	4	5	6	7	8	9
Columns	HE500B			HE280B			HE220B		
Beams	IPE550	IPE500		IPE450			IPE400		IPE360

A Dead Load (DL) equal to 5.0 kN/m^2 was applied on all floors, consisting of 3.0 kN/m^2 for the self-weight of the concrete slab and 2.0 kN/m^2 to account for the non-structural permanent components. The additional DL for the self-weight of beams and columns was applied directly to these elements. The building was assumed to be an office; hence, a Live Load (LL) of 2.00 kN/m^2 was applied on all floors. The only exception is the roof, which considered a Snow Load (SL) equal to 0.69 kN/m^2 based on Eurocode guidelines [12] for the Greek climate region in Zone III, 200 m of altitude and standard conditions. The analyses were performed considering the fire load combination defined according to the EN1990 [14] reported in Eq. 1 as follows.

$$q_d = 1.0\text{DL} + 0.3\text{LL} + 0.0\text{SL} \tag{1}$$

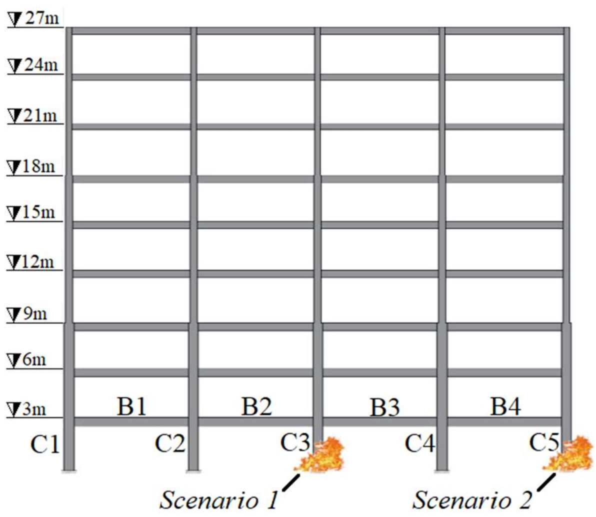


Figure 1. Case study steel moment-resisting frame (MRF) and fire scenarios.

2.2 Fire scenarios and parametric analysis

Two scenarios were considered to investigate the effects of fire on the progressive collapse resistance of the frame. Columns are typically identified as the elements that are more critical to structural stability and whose failure maximises the progressive collapse risk. Hence, fire was assumed to affect the central ground-storey column in Scenario 1 and a perimetral ground-storey column in Scenario 2. Figure 1 shows the frame and the fire-affected columns for the two scenarios.

For each scenario, a parametric analysis was performed to identify the parameters governing the progressive collapse of the MRF. Several parametric fires were defined according to EN 1991-1-2 [15], considering an opening factor of 0.044 m$^{1/2}$ for Scenario 1 and 0.051 m$^{1/2}$ for Scenario 2. Ventilation-controlled parametric fires were defined by varying the thermal inertia b and the fire load density $q_{\text{f,d}}$ within the range b = 900-1500 J/m^2 s$^{1/2}$ K and $q_{\text{f,d}}$ = 500-1000 MJ/m^2. Figure 2a and Figure 2b show the influence of these parameters on the gas temperatures for Scenario 1.

Finally, the variability of the loads was considered in the parametric analysis. Different loading levels, due to possible alternative load configurations were adopted, varying the value of the load factor λ. The load factor λ was defined as follows:

$$\lambda = \frac{\sum_{\text{i}=1}^{\text{n}} R_{\text{i}}}{Q_{\text{tg}}} \tag{2}$$

where $\sum_{i=1}^{n} R_{\mathrm{i}}$ is the sum of the n vertical base reaction forces of the frame, and Q_{tg} is the load target the structure is supposed to bear according to the load combination in Eq. 1. The load factor λ was varied from 1.0 to 1.3. Table 2 summarises the parameters varied in the numerical simulation.

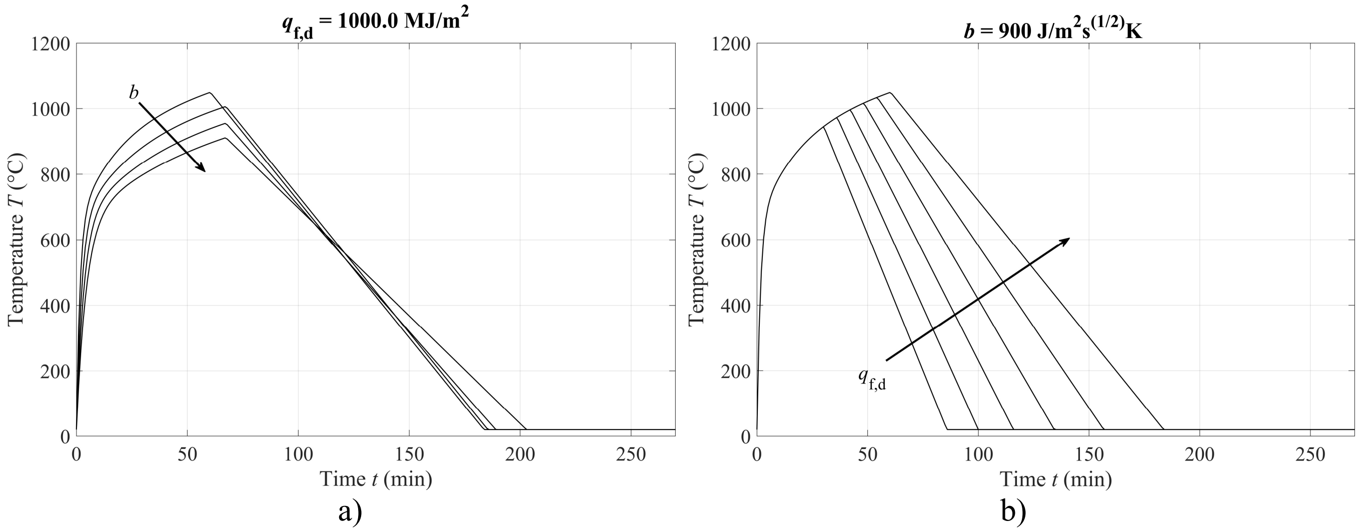


Figure 2. Parametric fire curves for Scenario 1: a) Influence of b; b) Influence of $q_{\mathrm{f,d}}$.

Table 2. Configurations varied in numerical simulation

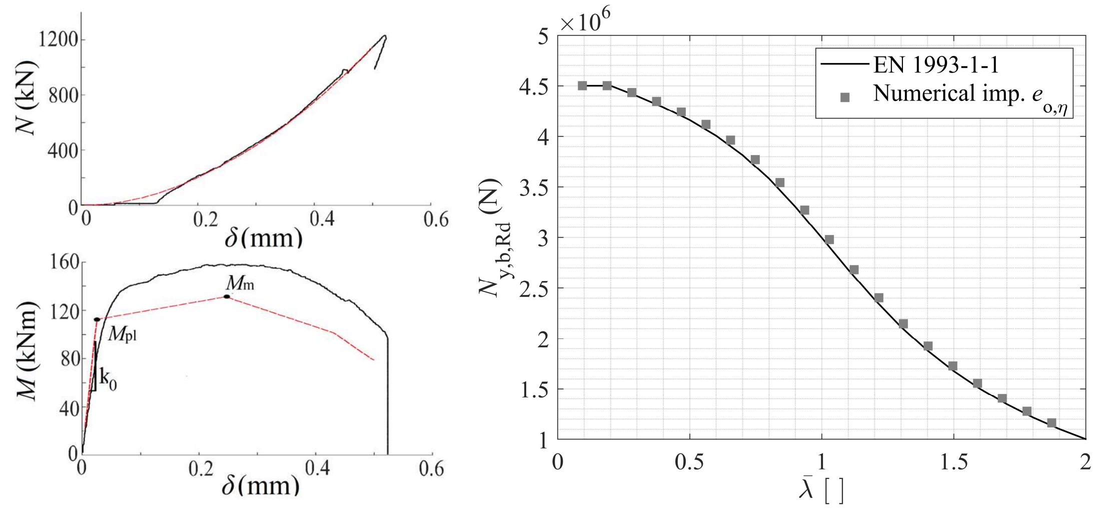
Fire scenarios	Fire curves	Load factors
1) Fire on the central ground-storey column	b = 900 - 1500 J/m^2 s$^{1/2}$ K	λ = 1.0 - 1.3
2) Fire on a perimetral ground-storey column	$q_{\mathrm{f,d}}$ = 500 - 1000 MJ/m^2	

2.3 Numerical model

3D numerical models of the plane MRF were developed in OpenSees [5] and OpenSees for fire [6]. The ground-storey columns were fixed in the x-direction and hinged in the y-direction. Lateral restraints were introduced on the beam-column joints of each floor to prevent a global out-of-plane loss of stability and account for the lateral stiffness of the transversal beams and the slab. Similarly, it was assumed that the slab provides effective restraints for lateral-torsional buckling of the columns. Conversely, the possible positive contribution of the slab to the progressive collapse resistance was conservatively neglected. The possible loss of stability of the structure and the buckling of columns were considered based on the EN1993-1-1 [7] recommendations. A global equivalent imperfection in terms of initial column rotation Φ was applied to the structure, and both in-plane and out-of-plane local imperfections with a local magnitude e_{o} were considered. Masses were concentrated on the floor and at half the columns' length.

A distributed plasticity approach was considered for columns. The elastic shear stiffness of the columns was included using the 'Section Aggregator' command. A lumped plasticity approach was used for beams, concentrating plastic hinges at the beams' ends. Plastic hinges were modelled with the 'Parallel Plastic Hinge' (PPH) approach proposed by Lee *et al.* [16]. This approach considers flexural and axial behaviour via two springs. It is, therefore, particularly suited for progressive collapse scenarios, in which both flexural and catenary actions contribute to the resisting mechanism. Beam-column joints were modelled through two independent flexural springs connected to two orthogonal rigid links, whose extension is representative of the height of the column and beam steel sections. The flexural springs were employed to define a 'Scissor Model' [17] to simulate the deformability of the column web panel and flanges. The main parameters of this model were determined according to Charney and Downs [18].

OpenSees for fire was extensively validated against experimental results and was shown to be able to correctly capture the behaviour of heated columns [6]. Conversely, the response of beams and columns at ambient temperature was validated and calibrated against experimental results. A brief description of the validation is provided here, while detailed information can be found in Possidente *et al.* [10]. Figure 3a shows the validation of the PPH model against the experimental results provided by Dinu *et al.* [19]. In such tests, a beam-column connection was investigated by testing a steel subassembly subjected to a column removal scenario. Figure 3a shows a good agreement in the evolution of the axial force with the vertical displacement δ, measured at the actuator simulating the column removal. A conservative behaviour is observed for the flexural response, which is consistent with the plastic bending moment capacity M_{pl} of beams. A similar model was implemented in the FE model of the entire structure and calibrated based on the dimensions of the connections. Figure 3b shows the validation of the numerical model for column buckling. The capability of the model to simulate buckling is essential for progressive collapse scenarios and was carefully checked in the present study. For this purpose, GMNIA (Geometrically and materially nonlinear analysis with imperfections included)m analyses were performed to reproduce the buckling resistance provided in EN1993-1-1 [7] of compressed members $N_{y,b,Rd}$. Indeed, various research studies [20,21] showed that a good match between numerical results and buckling curves could be achieved for the strong axis buckling by introducing the imperfection $e_{o,\eta} = k\alpha(\bar{\lambda}\text{-}0.2)$, indicated also in the ENV1993-1-1 [8]. In such an equation, $k=W_{el}/A$ is the kernel radius, defined as the ratio between the elastic section modulus W_{el} for the strong axis and the area A of the section, while α and $\bar{\lambda}$ are the imperfection factor and the non-dimensional slenderness according to EN1993-1-1 [7], respectively. Analyses were performed by varying the length of simply supported columns made of HE400B section with steel S235 (*i.e.,* same as the ground-storey columns of the case study structure). Lateral restraints were introduced to avoid buckling on the weak axis, and each column was modelled with 6 fibre elements. Figure 3b shows that the numerical results are in good agreement with the buckling curve. Hence, the model was deemed sufficiently accurate to reproduce the effect of buckling, and the same discretisation was employed in the FE model of the entire structure.



a) b)

Figure 3. Model validation. a) Experimental validation for the PPH; b) Numerical validation for the columns.

2.4 Analysis procedure

The analyses were performed with OpenSees and involved both thermal and mechanical analyses. First, thermal analyses were performed to determine the temperature inside the steel sections [6]. Analyses were carried out for the columns heated in the two scenarios, considering all the different fire curves. The same temperature distribution was assumed along the column height.

The mechanical analyses were performed in two steps: *1)* the gravity loads according to the selected load factor λ were applied to the MRF. As in this stage, the structure remained essentially in the elastic range, the results of the analyses are not shown in detail in the following section; *2)* the temperature evolution obtained from the thermal analyses was applied to the heated columns, and the structural response of the MRF was investigated. Dynamic analyses were carried out to capture the final collapsing phase and understand the collapse mechanism. Fire analyses were performed considering both scenarios.

3 RESULTS

For brevity, the results are shown in detail only for a few representative cases of Scenario 1. Figure 4 shows the relevant results for a load factor λ=1.1 and the parametric curve defined with b=1500 J/m^2 s$^{1/2}$ K, and $q_{f,d}$=500 MJ/m^2. The axial force in the ground-storey columns and the average steel temperature $T_{s,avg}$ in the heated column are shown in Figure 4a. The axial force in the heated column (*i.e.,* column C3) increases owing to the restrained thermal expansions until it reaches the buckling capacity at about 250°C. After that, the capacity is reduced due to the degradation of the mechanical properties caused by the further temperature increase, and the column is unloaded. The axial force reduction continues in the cooling phase, where tension forces appear due to thermal contraction. Throughout the analysis, the loads previously carried by the heated column are redistributed to the adjacent columns C2 and C4, while a very small portion of the loads is redistributed to the perimetral columns C1 and C5. The columns C2 and C4 experience buckling in the cooling phase when the average steel temperatures $T_{s,avg}$ are lower than 250°C and the axial force reaches the buckling resistance $N_{b,Rd,MN}$, calculated incorporating the effects of the bending moments according to EN1993-1-1 [7]. Nevertheless, the MRF does not collapse as the catenary mechanism activates, and the slight load increase induced by restrained thermal contraction is effectively redistributed to the perimetral columns. Figure 4b shows that the beams are not critical as the bending moments in the beams above the heated column (*i.e.,* beams B2 and B3) remain below the maximum bending moment M_m defined in the PPH model.

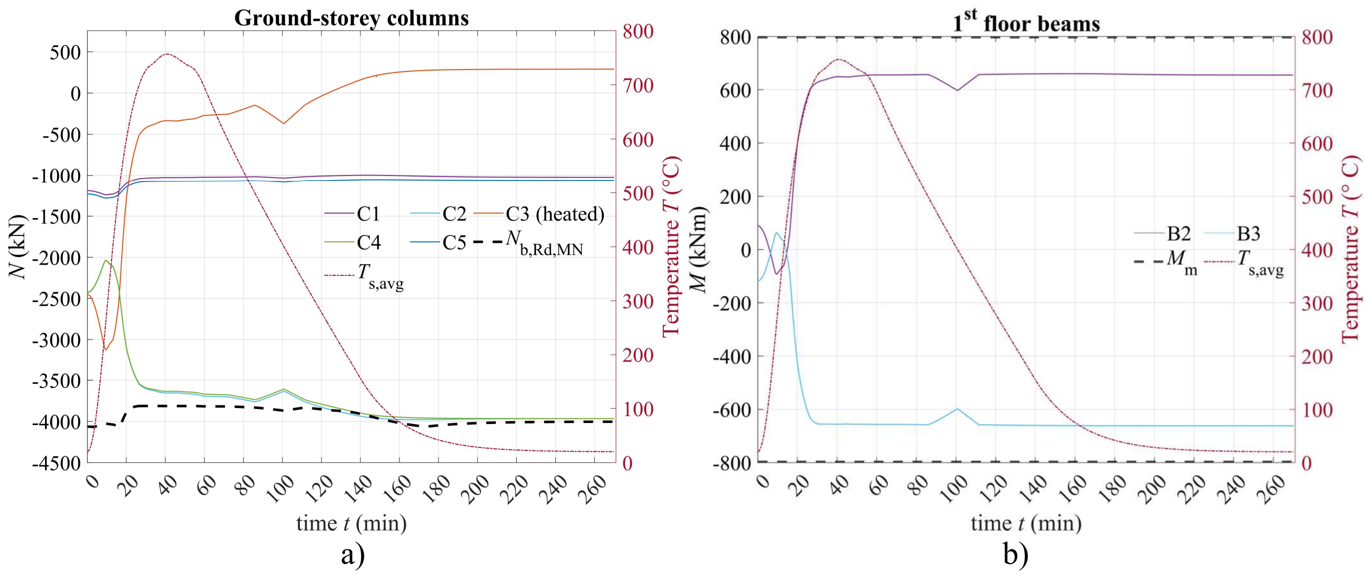
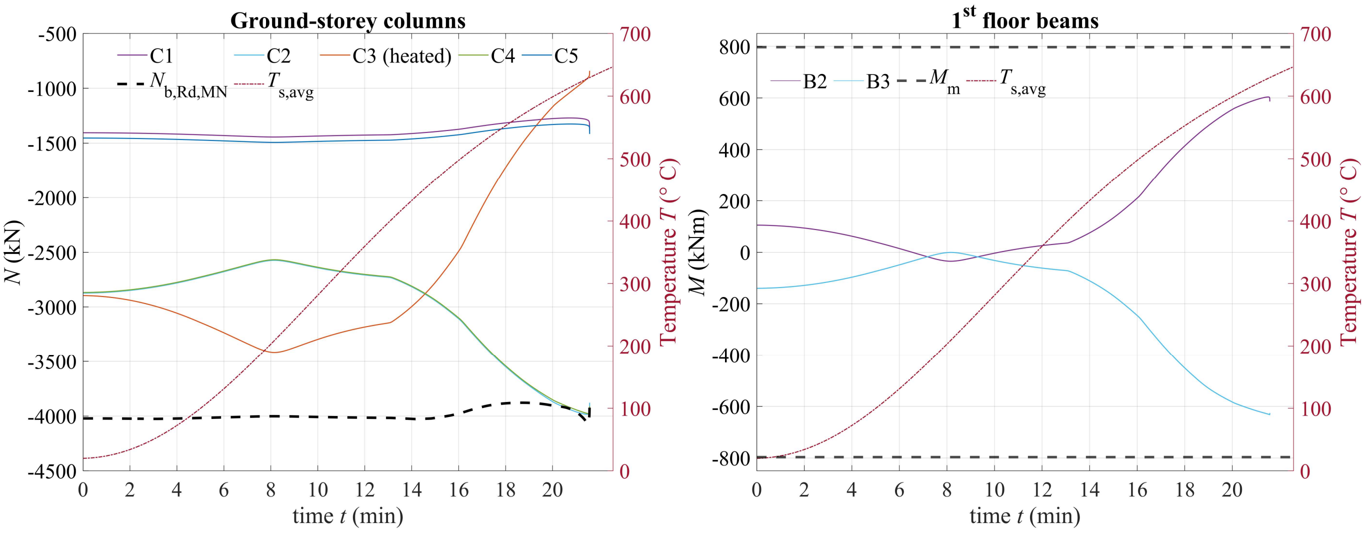


Figure 4. Scenario 1, b=1500 J/m^2 s$^{1/2}$ K, $q_{f,d}$=500 MJ/m^2, λ=1.1: a) Axial forces in the ground-storey columns; b) Bending moment in the beams above the heated column.

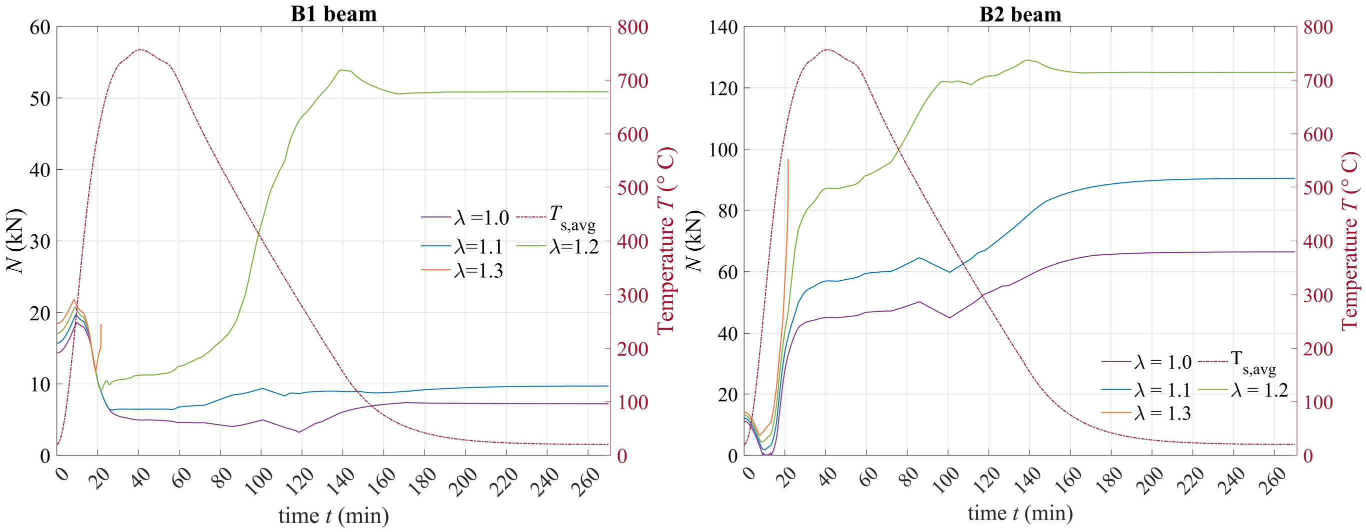
Similarly, Figure 5 shows the axial force in the ground-storey columns and the bending moment in the beams B2 and B3 in the case of λ=1.3. It can be observed that the increase in the load factor causes the collapse of the structure. Figure 5a shows that the axial force in the heated column is reached at about 210°C and that the load redistribution triggers column buckling in the adjacent columns at about 630°C. In this case, the out-of-plane runaway deflections caused by buckling of columns C2 and C4 cannot be effectively prevented, causing the collapse of the MRF in the heating phase. As shown in Figure 5b, the beams are not critical as the bending moment in the beams B2 and B3 is well below the maximum moment M_m. Hence, the buckling of columns C2 and C4 initiates the progressive collapse of the frame.



a) b)

Figure 5. Scenario 1, b=1500 J/m^2 s$^{1/2}$ K, $q_{f,d}$=500 MJ/m^2, λ=1.3: a) Axial forces in the ground-storey columns; b) Bending moment in the beams above the heated column.

Figure 6a and b compare the axial forces in beams B1 and B2 to track the development of catenary actions for all load factors considered in the parametric study. The measured tension forces are very small and well below the axial capacity of the beams. For λ<1.3, the beam's tension forces effectively redistribute the gravity loads during the entire fire exposure, even when buckling occurs in the C2 column for λ=1.1 and λ=1.2. Conversely, for λ=1.3, the load redistribution mechanism stops being effective at about 630°C, as the higher loads induce a more rapid increase in the tension force. In this situation, a further temperature increase in column C3 induces a deformed configuration that, combined with the gravity loads, causes the loss of stability of the MRF.



a) b)

Figure 6. Scenario 1, b=1500 J/m^2 s$^{1/2}$ K, $q_{f,d}$=500 MJ/m^2: a) Axial forces in the B1 column; b) Axial forces in the B2 beam.

Figure 7 shows the results of the parametric analysis in terms of time of collapse, *i.e.*, the time of fire exposure before the collapse. As aforementioned, considerations in the domain of the temperature were provided as the time of collapse would differ from the one provided if a more typical situation for high building with protected steel elements were to be considered. Nevertheless, it seems relevant to assess the influence of the different parameters on the time of collapse. Trends of the collapse-retarding effect of the parameters remain valid in the case of protected steel elements. Hence, despite the structure will be protected, Figure 7 gives insight on the influence of b and $q_{f,d}$. The time of collapse was defined as the time at which no converged solution could be found in the numerical analyses due to a loss of stability of the structure.

Figure 7a shows that for Scenario 1, the collapse is observed only for λ>1.1. In particular, λ=1.2 entails collapse times spanning from 22 to 38 minutes, while for λ=1.3 collapse occurs between 14 to 22 minutes. All collapses were observed in the heating phase, as when buckling occurred in the columns adjacent to the heated one during the cooling phase, it was always possible to redistribute loads to the perimetral columns. Hence, the fire load $q_{f,d}$ shows no impact on the time of collapse. Indeed, for heating curves with the same value of thermal inertia b, collapse is triggered in the heating phase by exceeding a critical average steel temperature value $T_{s,avg}$, and therefore, the only possible effect of reducing $q_{f,d}$ is to avoid collapse by preventing from reaching such critical temperature in the steel section (see Figure2b). This is the case when $q_{f,d}$ is reduced from 600 to 500 for b=1500 J/m^2 s$^{1/2}$ K and λ=1.2. Figure 7b shows that Scenario 2, which involves the heating of column C1, is more severe and collapse occurs already for a load factor λ=1.1. Similar considerations as the ones for Scenario 1 apply in this situation. The collapse times span from 21 to 37 minutes for λ=1.3, 15 to 23 minutes for λ=1.2 and 13 to 20 minutes for λ=1.3.

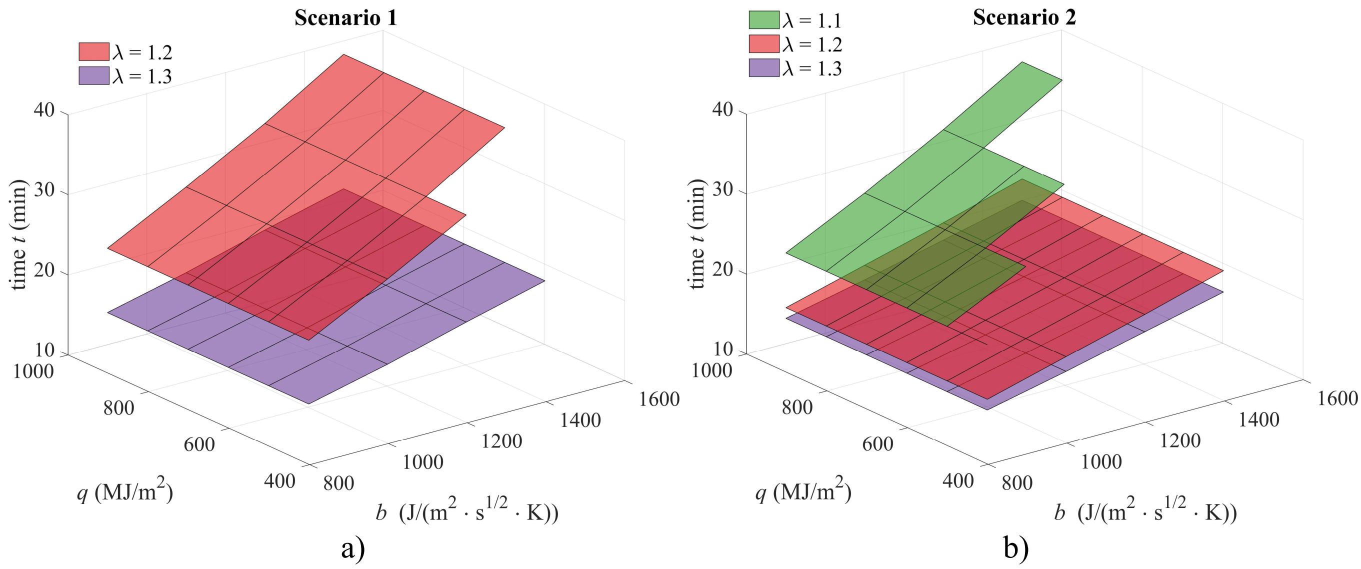


Figure 7. Time of collapse: a) Scenario 1; b) Scenario 2.

In general, it can be concluded that for the loads considered in this study, the fire combination is not sufficient to induce the collapse of the structure, as no collapse was observed for λ=1.0. Hence, the seismic design of the structure seems adequate to provide sufficient capacity to the structure also in fire situation. Nevertheless, the DL and the LL on the structure may change owing, for instance, to the use of the structures for different purposes or the reorganisation of internal non-structural components. In this case, a careful check of the design is required as the fire combination may become critical for progressive collapse. Finally, it is noteworthy that preliminary analyses performed with the alternate path method (APM) showed that for a progressive collapse load combination according to the recommendations provided in the UFC [22], the MRF is not able to withstand the removal of the central or perimetral column, owing to the dynamic effects induced by the sudden removal of the column [23].

4 CONCLUSIONS

This paper presents and discusses the results of a parametric study on the fire-induced progressive collapse of steel structures. Non-linear thermomechanical analyses were performed to investigate the progressive collapse in a 9-storey Moment Resisting Frame (MRF) considering various fire scenarios and parametric fire curves affecting the ground-storey columns. To better capture the progressive collapse response of the MRF, and accurately simulate both beam-type (ductile plastic hinges and catenary action) and column-type (non-ductile buckling) mechanisms, beam and columns were validated against experimental data and design standards. The study shows that the structure is sufficiently robust to withstand the fire scenarios when the fire combination is considered. However, by increasing the gravity loads by up to 30%, *i.e.,* employing load factors λ up to 1.3, progressive collapse occurs depending on the scenario and the fire curve. In particular, progressive collapse may be triggered with a loading level $\lambda \geq 1.2$ when the central column is heated and $\lambda=1.1$ when the perimetral column is heated. As collapse occurred in the heating phase, less severe parametric curves, *i.e.*, with higher b or lower $q_{f,d}$, may hinder from attaining the critical steel temperature $T_{s,avg}$ causing collapse, *e.g.,* for $\lambda=1.1$ in Scenario 1 and $\lambda=1.2$ in Scenario 2. The thermal inertia b, may also have the effect of postponing the time of the collapse. In contrast with the typical consideration based on the ductile behaviour of beams, the progressive collapse was always triggered by the buckling of the columns adjacent to the heated column. It is noteworthy that the attainment of the buckling capacity in such columns did not always induce the collapse of the structure. Indeed, especially when buckling occurs during the cooling phase of the fire curves, the MRF shows sufficient capacity to further redistribute the loads from the columns adjacent to the heated one to the other elements. This situation is rare in the case of higher load factors, in which the higher loads prevent effectively redistributing the force previously carried by the buckled columns through catenary action in the beams. In future applications, different steel structures and scenarios will be investigated, to generalise results. Among the others, MRFs more sensitive to the beam mechanism will be analysed. These analyses aim to further investigate the redistribution capacity of the frames, particularly in the case of multiple buckled columns, and explore the potential for progressive collapse during the cooling phase.

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