Progressive collapse resistance of steel and composite frames under localised fire scenarios near an internal column

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Abstract

Local damages to key structural members may originate from different causes, including localised fires or earthquakes, and, as far as the affected area remains small and the damage is contained, the risk is reduced. However, if the structure does not have the capacity to absorb the damage and bridge over the lost components, progressive collapse may be initiated, with serious consequences for the life of the occupants and the costs of losses. Even if some inherent structural properties, like redundancy and ductility, bring a beneficial contribution to the resistance of structures subjected to such scenarios through the activation of alternate load paths, these properties can be affected when working under elevated temperatures, and thus structural integrity can be at risk.

This paper investigates the cumulative effects of seismic events and elevated temperature on the progressive collapse resistance of two-way frames with steel and composite steel-concrete floors. Numerical models are calibrated against relevant test data. The results show that, even if fire protection is an effective way in increasing the resistance of structural components under elevated temperatures, the failure may propagate due to the attainment of the bearing capacity of the surrounding elements and connections that are still at ambient temperature. Also, the interaction between concrete slabs and steel beams may provide additional capacity to stop the progressive collapse.

Keywords: *Bolted joint, Multi-hazard, Robustness, Progressive collapse, Catenary action*

1. Introduction

The Eurocodes define robustness as a means of ensuring integrity of a structure when subjected to unexpected events, and preventing progressive collapse under extreme loading conditions [1]. All types of private and public buildings may be exposed to extreme events, that can be caused by natural or human-made hazards, e.g., earthquakes, tsunami, hurricanes, fires, explosions, or vehicle impacts. Such events frequently cause localized structural damage, which can propagate and finally be responsible for a progressive collapse (partial or total) of the structure. Progressive collapse may be defined as the mechanism by which local damage triggers a chain of failures, causing a collapse of the building or a considerable part of it [2]. Such extreme events are always assisted by material and human losses [3]. Previous tragic events and the associated outcomes clearly highlighted that society and communities need robust buildings, which can resist local damage without triggering a progressive collapse; this performance objective is even more crucial when critical infrastructure (e.g., power stations, hospitals, passenger terminals) or those open to public (e.g., commercial centres, schools) are under consideration. For such structures, resilience demands, i.e., the ability of a structure to absorb but also recover after extreme events, could be expressed. This demand for resilience goes a step further when compared to the request for robustness, as the capability to rapidly recover the pre-event performance state (or above) in terms of functionality and operation is included [3].

Except for specifically designed protective systems, it is usually impractical for a structure to be designed to resist general collapse caused by severe abnormal loads acting directly on a large portion of it. However, structures can be designed to limit the effects of local collapse originating from different causes, and to prevent or minimize progression of collapse [4]. Extensive research has been made on progressive collapse of steel structures after a fire. Porcari et al. [5] provided a comprehensive literature review about progressive collapse induced by fire in multi-storey steel buildings. They reported that the utilization of performance-based fire design demands is an efficient tool to evaluate buildings for different configuration and to identify the critical parts that may be exposed to collapse under fire loading. Historically, construction codes have concentrated on the need to safeguard main structural steel components from fire effects, by providing a covering layer, which can protect the material from high temperature effects for a limited period. The protection includes, among others, intumescent paint, gypsum board wrap, or spray applied coatings.

Research developed worldwide indicated that the risk of collapse under the effects of fire, explosions and other exceptional events may be sometimes comparable to the conventional ones that have already been addressed in structural engineering design [6]. The study reported in [7] investigated the response of steel frames to close-in detonations. The results indicated that the local failure mechanism and resistance to progressive collapse of steel building frames depend very much on the blast load parameters but also on the level of gravity loads in columns.

Until recently, researchers neglected the fire resistance of joints under the argument that joint capacities at room temperature are higher than those of connected elements and that joints usually experience a more slowly increase in temperature when compared to adjacent column and beams (high concentration of mass). Nevertheless, brittle failures of joint components have been noticed during the cooling phase, for two key reasons: the development of significant tensile forces and the sensitivity of the resistance of welds and bolts to elevated temperature [8]. For this reason, recent researches were conducted on the behaviour of joints subjected to elevated temperature and to combined axial forces and bending moments [9].

In the case of an accidental fire, buildings must maintain stability for a certain period, to ensure that an adequate level of life safety is maintained and economical losses are reduced. The classic structural fire design approach is based on tests performed on individual members under idealized boundary conditions and loadings [10]. It is also acknowledged that the behaviour of a structure during fire is rather different when compared with single elements tested at a standard temperature curve for evaluating fire resistance [5].

Jiang and Li [10] made an analysis of the progressive collapse of a 3D steel frame under fire conditions. An explicit dynamic study was performed to determine the effect of heating location and loading ratio on the collapse mechanism and load redistribution path. They found that the three-dimensional model predicted a distinct collapse mode and the paths of load redistribution when compared to the two-dimensional model. Also, the collapse resistance was affected by the asymmetric load transfer which is not captured in the two-dimensional model. In conclusion, a three-dimensional model is a more accurate approach to evaluate the robustness of structures. Abnormal loads differ notably from the normal loads considered in structural design. Indeed, these loads are associated with low-probability/high-consequence events. On one hand they have a lower probability of occurrence than normal actions, and on the other hand, their potential consequences are much more significant, resulting in huge losses [3].

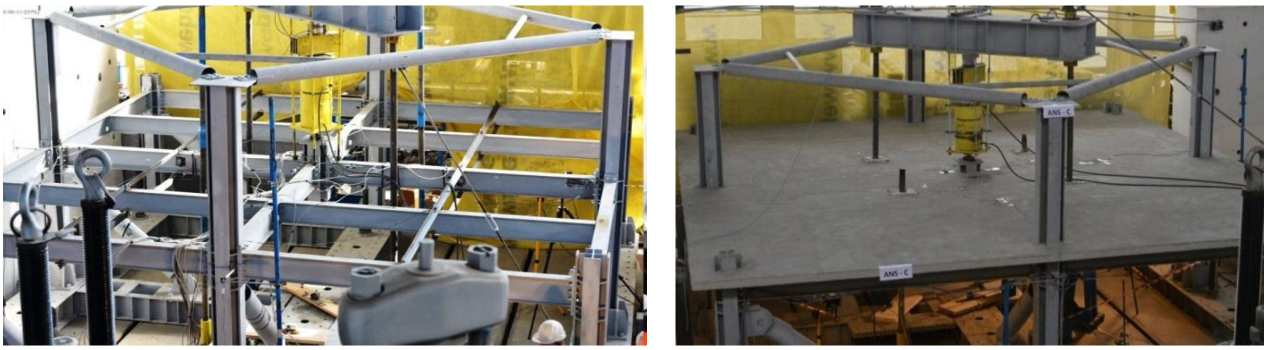
Fire after an earthquake is commonly considered as a low-probability/high-consequence event. In case of post-earthquake fires, the risk of structural collapse is increased because of the damage encountered by the structure during the earthquake, the limited firefighting availability, lack of water supply, extreme traffic congestion and other possible problems during the post-earthquake intervention [2], [11]. In particular, damages associated with fires following earthquakes can be severe. Two historical examples are 1906 San Francisco and 1923 Tokyo earthquakes, when most damages were the result of the subsequent fires [2]. However, the design of steel and composite steel-concrete buildings in Europe addresses separately the fire safety and the seismic safety, without considering the possibility of fire after an earthquake [12]. The procedure for the fatigue assessment of steel building structures subjected to earthquakes, reported in [13], was based on the extension of the classic high-cycle fatigue assessment to cases of low-cycle fatigue. It may serve as a basis for the introduction of a fatigue limit state in the earthquake design of steel structures. It may be also used for the damage assessment of existing steel buildings subjected to past earthquakes. In their study, Suwondo et al. [14] adopted 3D models to investigate the behaviour of composite steel frames damaged by earthquake and fire. The results showed that earthquake reduces the fire resistance of the building and revealed that the collapse may be initiated by the failure of columns.

This study investigates the effect of a localised fire on the progressive collapse resistance of two-way frame substructures with steel and composite steel-concrete floors. The study starts from the results of experimental tests conducted by the authors on the progressive collapse resistance of steel and composite steel-concrete frame [15], [16], and on the response of T-stub elements under fire and combined monotonic after cyclic loading [17], [18], [19]. Several scenarios are considered through numerical simulations: loss of a single internal column at room temperature and loss of the column due to fire with and without considering the fire effects on the adjacent joints. Numerical simulations are performed using a nonlinear static procedure and the SAP2000 software [20].

2. Summary of experimental tests on two-way frame models and joint components

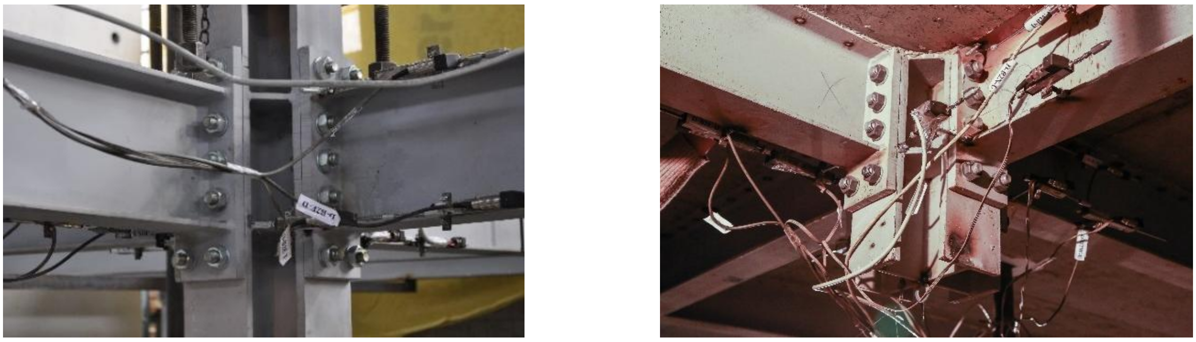
2.1. Two-way steel/composite frame models under column loss

The frame models were extracted from a case-study building with three-bay, four-span, six-storey steel structure with and resisting frames in both orthogonal directions. The substructure selected for the test is located at the first floor of the structure and has one story, two spans, and two bays. Because of laboratory restrictions, the 16.0 m by 16.0 m assembly was downscaled, resulting in a 6.0 m by 6.0 m specimen. The tested system included column, primary and secondary beams, and additional vertical and horizontal braces to simulate the boundary conditions and constraints that develop in the complete case-study structure. The extended endplate bolted beam-to-column connections were designed as fully rigid and fully strength. Fig. 1 *(Continued*a shows the isometric view of the bare steel frame specimen (noted as ANS-M) and test setup. The second system (noted as ANS-C) has the same geometry, except the main and secondary beams act as composite sections (see Fig. 1 *(Continued*b). The number of shear studs was calculated to achieve full shear connection. Fig. 1 *(Continued*c, d show the beam-to-column joints for the two models. Note that structural steel S275 was used for beams, S355 for columns, concrete C20/25 for concrete floor, reinforcement from S420 steel and steel S235J2 + C450 for headed shear studs. More details may be found in [15] and [16].



a) b)

Fig. 1. Views of two way steel specimen (a) and steel-concrete specimen [15], [16]



c) d)

Fig. 1 (Continued)

Force was applied to the top of the central column and gradually increased until the failure of the specimens. The vertical force versus vertical displacement curve for the central column is shown in Fig. 2. The composite beam specimen ANS-C has a larger initial stiffness and yield strength compared with the bare steel specimen ANS-M. After the elastic stage, both substructures show a significant increase of capacity until the peak load is attained. However, the mechanisms contributing to this increase are different. Thus, for the composite specimen, the main source is the bending and compressive arching. In case of bare steel specimen, a catenary force starts to develop under large deformation conditions, leading to a significant increase in the ultimate deformation capacity.

Fig. 3a, b shows the damage status in joints adjacent to the lost internal column and on the opposite side of the beam connected to the lost internal column, for both bare steel and composite frames. As may be seen, the fracture develops at the beam end near the internal column, for both bare steel and composite frames. Also, damage may be seen in the beam ends away from the internal column, but the failure is prevented.

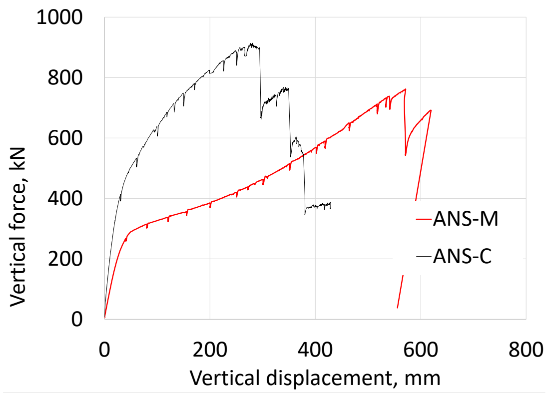


Fig. 2. Vertical force versus vertical displacement of central column, steel and steel-concrete models, experimental tests

a) b)

Fig. 3. Photos after the test: a) steel frame test: joint adjacent to the lost internal column (left) and on the opposite side of the beam (right); b) composite frame test: joint adjacent to the lost internal column (left) and on the opposite side of the beam (right) [15], [16]

2.2. Tests on bolted T-stubs

First tests on bolted T-stubs were performed at room temperature, and considering three quasi-static loading protocols, i.e., monotonic, cyclic, and monotonic after cyclic loading. Geometry of the T-stubs is presented in Fig. 4. The first two types of loading protocols aim at evaluating the maximum capacity of the T-stub under monotonic and cyclic loading. Then, based on the maximum plastic deformation achieved in the cyclic loading, the level of damage is set to low, moderate, and severe, and then the T-stub is tested for a combined protocol, i.e., monotonic after cyclic loading at different intensities. Based on the maximum plastic deformation capacity in the cyclic loading, i.e., eight-time yield displacement, or $8\times d_y$, the three level of damage proposed for the monotonic after cyclic loading were $4\times d_y$, $6\times d_y$, and $8\times d_y$, respectively. More data about geometry, test setup, and test results may be found in [18], [19]. The second series of tests comprised monotonic tests on materials and bolted T-stubs at normal and elevated temperature (542° C) in a steady-state approach, considering quasi-static and high loading rate protocols. For the present study, only the results from quasi-static tests are of interest. Note that T-stubs in both series of tests shared the same geometry and material characteristics. More data about test results may be found in [19]. Fig. 5a shows the force-displacement curves for monotonic after cyclic loading at $8d_y$

(i.e., C_{8dy} + M), while Fig. 5b shows the comparative force-displacement curves for monotonic tests at room and 542° C. As seen from Fig. 5a, when T-stub is heavily damaged (i.e., 8dy), initial stiffness and the normalized plastic deformation capacity in a subsequent monotonic loading reduces significantly when compared with the characteristics of the intact T-stub. Also, as expected, the stiffness, peak strength and ultimate elongation reduce when the T-stub is heated at 542° C (Fig. 5b).

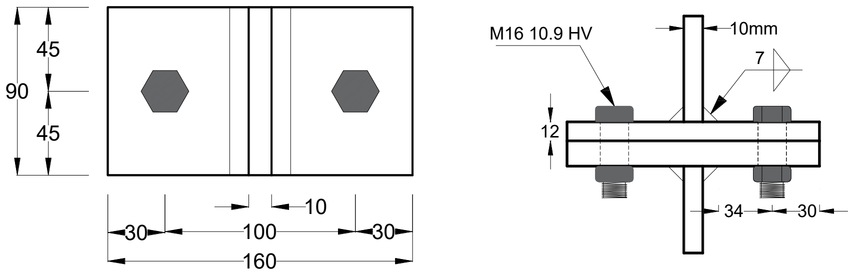
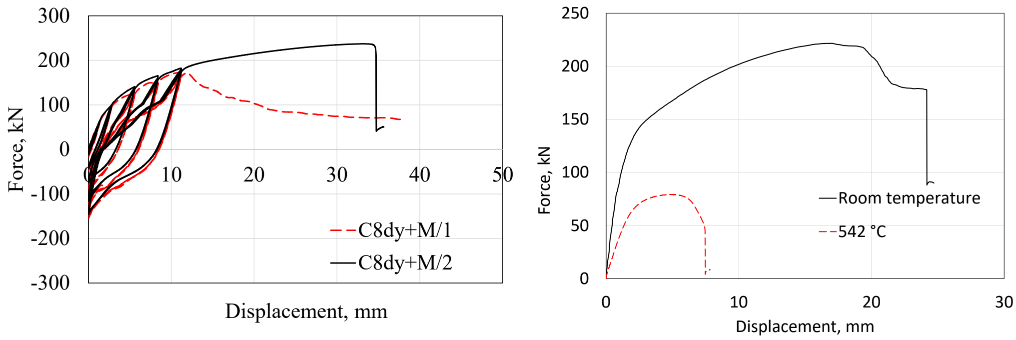


Fig. 4 Geometry of the T-stubs [17], [19]



a) b)

Fig. 5 Force-displacement curves, 12 mm thick flange T-stubs: a) monotonic test vs monotonic after cyclic test at $8\times d_y$; b) monotonic test at room and elevated temperatures

3. Parametric study

3.1. Validation of numerical models

A study was undertaken to investigate the progressive collapse resistance of steel and composite frames under single and multi-hazard scenarios:

- Scenario 1: column loss (e.g., due to localized fire), without thermal effects or seismically induced damage to other elements; this is considered as a reference scenario.
- Scenario 2: column loss (e.g., due to localized fire), without thermal effects to other elements, in the aftermath of an earthquake with three different intensities, i.e., low, moderate and severe plastic deformations at beam ends.
- Scenario 3: localised fire causing the loss of the column and adjacent beam ends
- Scenario 4: localised fire causing the loss of the column and adjacent beam ends in the aftermath of an earthquake with three different intensities, i.e., low, moderate and severe plastic deformations at beam ends.

For Scenario 2, in the first step, the damage due to earthquake is modelled using the residual capacity at the joints, as indicated in Fig. 5a. The seismic damage level is assumed to be low, moderate and severe, i.e., plastic deformations at beam ends is $4\times d_y$, $6\times d_y$, and $8\times d_y$, respectively. Then, in the second step, the internal column is removed and the joint above the lost column is pushed down. For Scenario 3, in the first step, the damage due to fire is modelled for beam ends (see Fig. 5b), then the internal column is removed and the joint above the lost column is pushed down. For Scenario 4, in the first step the damage includes damage due to fire for two beam ends, and damage in the aftermath of an earthquake with three different intensities. Then, in the second step, the internal column is removed and the joint above the lost column is pushed down.

The two-way frame systems, i.e., ANS-M and ANS-C, were modelled using SAP2000 program, see Fig. 6. The numerical models were calibrated against experimental data reported in previous two sections. The properties of the materials were defined based on the results obtained from coupon tests. The modelling of nonlinear behavior for columns was done using concentrated hinge models considering the axial-flexural interaction (P-M_2-M_3). For the beams, fiber-type hinge models were adopted. The later models offer more accuracy than concentrated hinge models, especially for beams under axial load and moment (P-M_3) interaction, which develops at large deformation stage associated with catenary action. The additional features for composite action were modelled using the nonlinear layered shell approach for concrete slab, and multilinear-plastic link elements for shear studs. The numerical analyses were performed using a nonlinear static procedure. The load was applied by pushing down the central column until failure using displacement control.

The overall relationship between the vertical force and the vertical displacement below the central column is shown in Fig. 7. The results in the figures show a very good correlation with the experimental results. All the phenomena that occurred during the test can also be traced on the numerical force–displacement curve, i.e., elastic behavior, plasticity, initiation of catenary force, and failure.

For the multi-hazard scenarios (Scenario 2, Scenario 3, Scenario 4), the damage induced by earthquake and the thermal effects have been modeled using the residual properties of the joints (see Fig. 8a) and the characteristics at elevated temperature (see Fig. 8b). In both cases, the numerical models are accurate and approximate well the entire response.

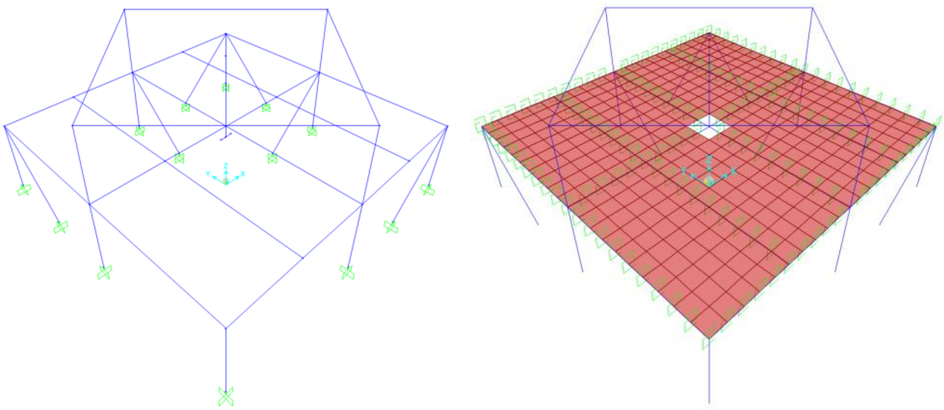


Fig. 6 Numerical models: a) ANS-M model; b) ANS-C model

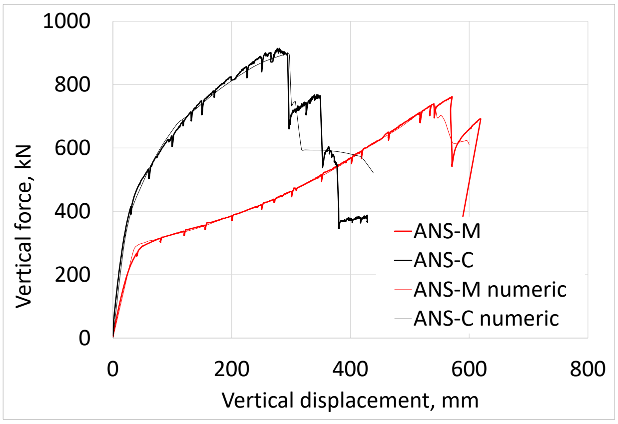


Fig. 7 Force displacement curves in column removal scenario, experimental vs numerical

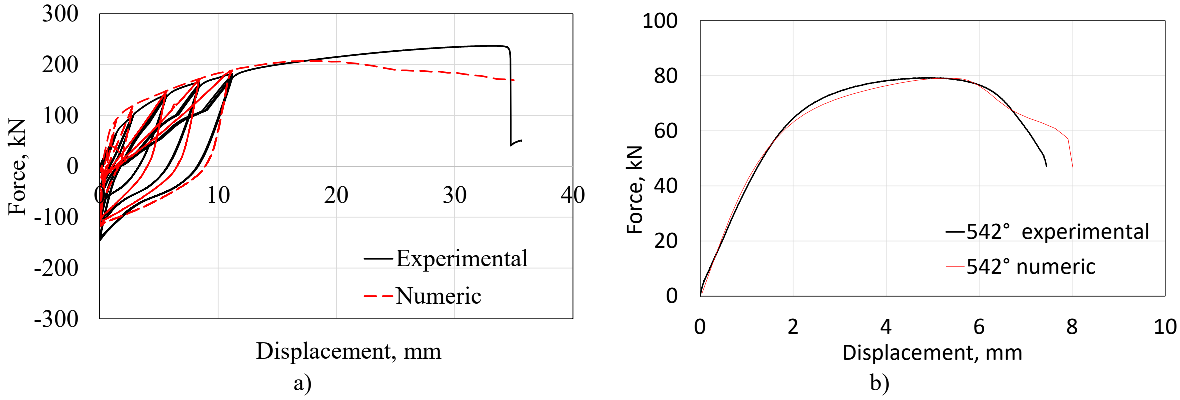


Fig. 8 Force displacement curves, experimental vs numerical: a) T-stub, monotonic after cyclic, 8×d_y; b) T-stub, monotonic test at room vs high temperature

3.2. Results of parametric study

In the following, the results of the parametric study for the 3 multi-hazard scenarios (Scenario 2, Scenario 3, Scenario 4) are described and compared with the reference scenario (Scenario 1). However, only the results for the bare steel system (ANS-M) are further detailed, while the results for the composite system ANS-C will be reported elsewhere.

Fig. 9 compares the results of Scenarios 1 and 2, i.e., the push-over capacity curves for the column loss and for the column loss after earthquake with three intensities (i.e., 4×d_y, 6×d_y, 8×d_y). When the level of plastic deformation (local damage) in the joints caused by the earthquake increases, the initial stiffness, the available ductility and the ultimate resistant capacity decrease. Some hardening effects in the cyclic loading during earthquake, i.e., increase of yield point, are also present, but their effect is limited.

Fig. 10 compares the results of Scenarios 1 and 3, i.e., the push-over capacity curves for the column loss and for the column loss combined with localised fire at adjacent beam ends. A single beam-end weakened by fire leads to some reductions in the vertical load resistance, while the loss of four beam ends has a much higher impact. However, the worst-case-scenarios are those where two or three beam ends are affected by fire, because of the unbalanced response of the central joint.

Fig. 11 compares the results of Scenarios 1 and 4, i.e., cascading scenarios involving localized fire affecting two beam ends in the aftermath of an earthquake with three intensities (i.e., 4×d_y, 6×d_y, 8×d_y), and the loss of the central column. Such scenarios may be considered as very low probability but high consequence events, with a significant impact on the load bearing capacity of the structural system. While the localized fire already weakens the resistance against vertical force, the seismic induced damages further reduce the initial stiffness and available ductility.

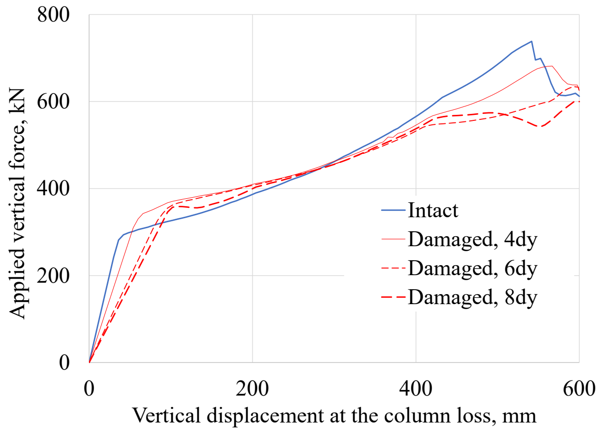


Fig. 9. ANS-M: Applied vertical force vs vertical displacement at the column loss after earthquake, intact and damaged ($4d_y$, $6d_y$, $8d_y$)

Fig. 12 shows the bending moment - axial force (M-N) interaction curves for the central joint of the ANS-M structure, in all four hazard scenarios detailed above. In the single hazard scenario involving column loss, the full plastic bending moment, M_{pl}, is reached, followed by a continuous reduction of the flexural capacity as the axial force increases during catenary stage. At failure, the axial force reaches almost 75 % of the axial beam resistance, N_{pl}. For the multi-hazard scenario involving column loss following a large earthquake (damage level at $8d_y$), the axial force initiates from lower levels of bending moment. At failure, the axial force reaches approximately 40 % of the axial beam resistance. For the other two multi-hazard scenarios, i.e., column loss and fire, and large earthquake (damage level at $8d_y$) followed by fire and column loss, the flexural capacity is severely reduced (almost halved), while the maximum axial force developed in the beam is less than 40 % of the axial beam resistance at room temperature.

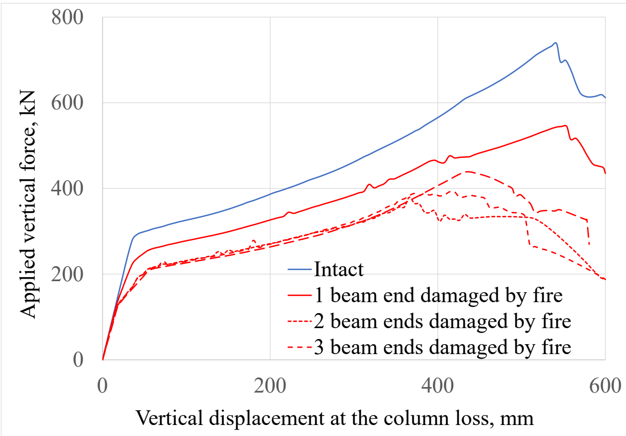


Fig. 10. ANS-M: Applied vertical force vs vertical displacement at the column loss after fire, intact and damaged due to fire (1 beam end, 2 beam ends, 3 beam ends, 4 beam ends)

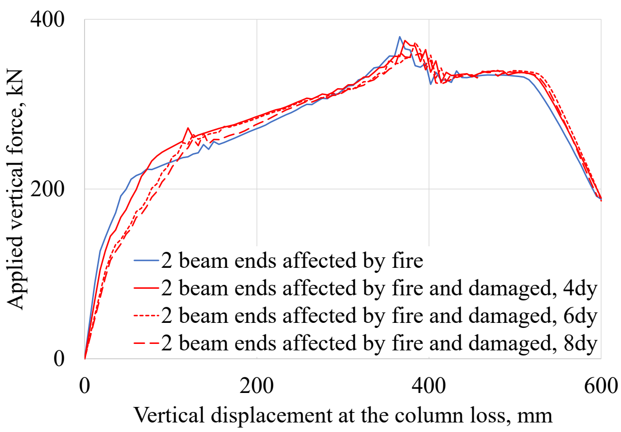


Fig. 11. ANS-M: Applied vertical force vs vertical displacement at the column loss after earthquake, two beam ends affected by fire and damaged due to earthquake (4dy, 6dy, 8dy)

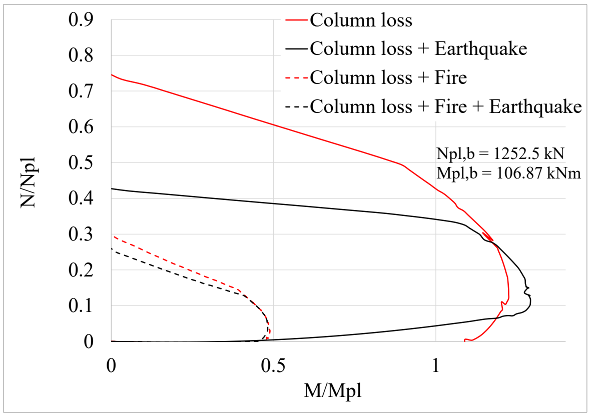


Fig. 12. M-N interaction curves for ANS-M in different hazard scenarios

5. Conclusions

Local damages to key structural members may originate from different causes, including localised fires or large earthquakes. However, as far as the local damages are constrained to a limited area, the risk of progressive collapse is reduced. However, if the structure does not have the capacity to bridge over the damaged components, progressive collapse may be initiated, with serious consequences for the life of the occupants and the associated costs of losses. Even if some inherent structural properties, like redundancy and ductility, bring a beneficial contribution to the resistance of structures subjected to such scenario through the activation of alternate load paths, these properties can be affected when working under elevated temperatures, and thus structural integrity can be at risk.

The parametric study presented in the paper compared the response of a frame substructure made of two-way frames with steel and composite steel-concrete floors to several single and multi-hazard scenarios. The numerical models have been calibrated against relevant experimental results. The results show that, even if fire protection is an effective way in increasing the resistance of structural components under elevated temperatures, the failure may propagate due to the attainment of the bearing capacity of the surrounding elements and connections that are still at ambient temperature. Also, the interaction between concrete slabs and steel beams may provide additional capacity to stop the progressive collapse. The results also shown that even the fire protection is an effective way in increasing the resistance of structural components under elevated temperatures, the failure may propagate due to the attainment of the capacity in the surrounding elements and connections that are still at ambient temperature.

Further studies will consider the interaction between concrete slabs and steel beams in same multi-hazard scenarios. Also, partial-strength joints and partial-interaction composite beams will be considered to investigate their role in provide additional capacity to arrest the progressive collapse and improve the structural robustness.

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