PROGRESSIVE COLLAPSE ASSESSMENT OF STEEL STRUCTURES UNDER FIRE

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1. ABSTRACT

This paper presents an assessment framework for structural behavior under static and time-dependent fire scenarios. The proposed method is an effort to associate the characteristics of the initial cause (*i.e.* the elevated temperature due to structural fire) with the building's response. The effect of the time-series used to perform a transient thermal analysis on the overall performance of the building is indicated. Relative applications of the method and further improvements are discussed.

Key words: steel, fire, progressive collapse, collapse resistance

2. INTRODUCTION

The mechanical properties of structural steel allow the bridging of large spans without the need for particularly large element sizes, compared to alternatives such as reinforced concrete. However, the capacity of steel decreases considerably under elevated temperatures. Current guidelines such as Eurocode 3 (EN 1993-1-2) [1] take into consideration the deterioration of steel's properties by reducing appropriately its stiffness and yielding strength for increased temperatures. This drop to the material's capacity affects the overall performance of steel buildings when structural fire occurs, which might lead to partial collapse of the building over the affected bays. This has lead numerous researchers to investigate the performance of steel structures or individual components under elevated temperatures [2-14].

In practice, fire protection can be achieved on three levels: (a) actively, (b) passively or (c) using fire preventing measures. Active fire protection includes all the systems incorporated in order to put out a fire when it starts or slow down its spread, such as sprinklers, fire extinguishers (manual or automatic), gaseous clean agents, firefighting foam systems, etc. All aforementioned are often installed in areas where risk of fire is increased or an intense concentration of people takes place (public buildings, dormitories, restaurants, etc.). Passive fire protection includes all measures that were designed in order to contain the fire within a specific area or slow down its spread. Contrary to active fire protection measures, passive fire protection does not require a trigger in order to act, but are part of the building in its everyday use, such as fire-resistant doors and walls. Finally fire prevention, includes all measures applied in order to reduce the risk or fire ignition.

During structural design, steel elements are protected from fire as well as corrosion using special fire resistant paint which can sustain particularly high temperatures. This paint slows down the heating of the steel section, providing enough time for response in a steel building on fire, before its structural integrity is at risk. Alternatively, steel sections can be embedded in fire-resistant materials such as concrete (e.g. concrete-encased steel-concrete composite columns), which also contribute to the element load bearing capacity and stiffness.

The large number of accidents and casualties of firefighters on duty has lead agencies and research institutions such as the National Institute of Standards and Technology in the U.S.A. to research alternative techniques for the protection of civilians, as well as firefighters from fire-related accidents. One of the main fields of current research is the development of methods for real-time assessment of the potential for collapse under fire. In the following sections, an assessment framework is presented in order to (a) estimate the potential for progressive collapse based on the thermal conditions considered and (b) define a retrofit scheme for steel buildings in order to improve their performance under structural fire conditions.

3. STRUCTURAL PERFORMANCE UNDER STATIC FIRE SCENARIOS

The proposed method is presented I parallel with an application example. In particular, the collapse resistance of a typical residential building is assessed. It is a six-storey steel building with five bays per horizontal direction (Figure 1). All columns have the same orientation: their web is parallel to global x-direction. A single section is considered for all columns and beams (HE280B and IPE270 correspondingly) for simplification purposes. Beam to column connections are simplified as well: (a) in x-direction, they are considered fully moment restrained, in order to form moment resisting frames, (b) in y-direction they are modeled as simple supports (hinges). The required lateral stiffness is provided by bracings that are installed in the middle bays at each face of the building. Column bases are considered fixed supports (*i.e.* they are able to receive the full moment applied without substantial deformation).

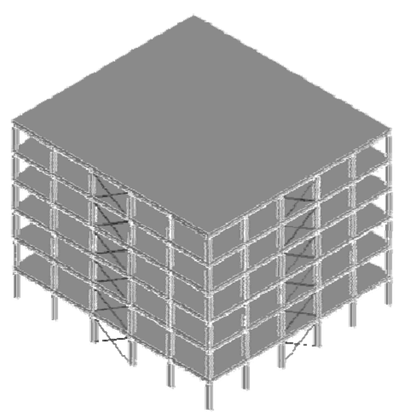


Fig. 1 Typical six-storey building modeled.

A structural fire is initiated that cannot be controlled for a prolonged period of time (e.g. one hour), so eventually it spreads to a large number of bays on multiple storeys. The scenario considered is time-related, as the thermal conditions are altered in time. Hence, in order to acquire more accurate information by the simulation of the event, the temperature conditions as a function of time (fire pattern) at all affected compartments inside the building should be defined and a transient thermal analysis should be performed. For the purposes of this work, three distinct time captions (static fire scenarios) were considered: (a) one soon after the outburst of the fire, (b) one at an intermediate point of time that the fire has spread to neighboring compartments and (c) at the time the fire brigade has arrived, but that the fire has spread to multiple storeys of the building, while the temperature is particularly increased.

In the first static fire scenario considered (SFS1), the fire initiated at a corner bay of the building, as indicated in Figure 2a. The temperature at the particular bay has increased substantially, but it did not reach its peak. Also, the fire is limited within a single room (*i.e.* the area defined by the four neighboring columns), so very limited structural elements are affected. In the intermediate fire scenario (SFS2), the fire has spread outside the room and a larger area is affected (Figure 2b). The temperature at the starting point has further increased and affects the neighboring bays where the temperature is comparatively lower, but elevated enough in order to affect the stiffness and capacity of load bearing elements. Finally, in the last fire scenario simulated (SFS3), the temperature at the starting point has reached a maximum point, but the fire has spread even more, covering three out of five bays of the building on the ground floor, while reaching as high as the third storey of the building as illustrated in Figure 2c.

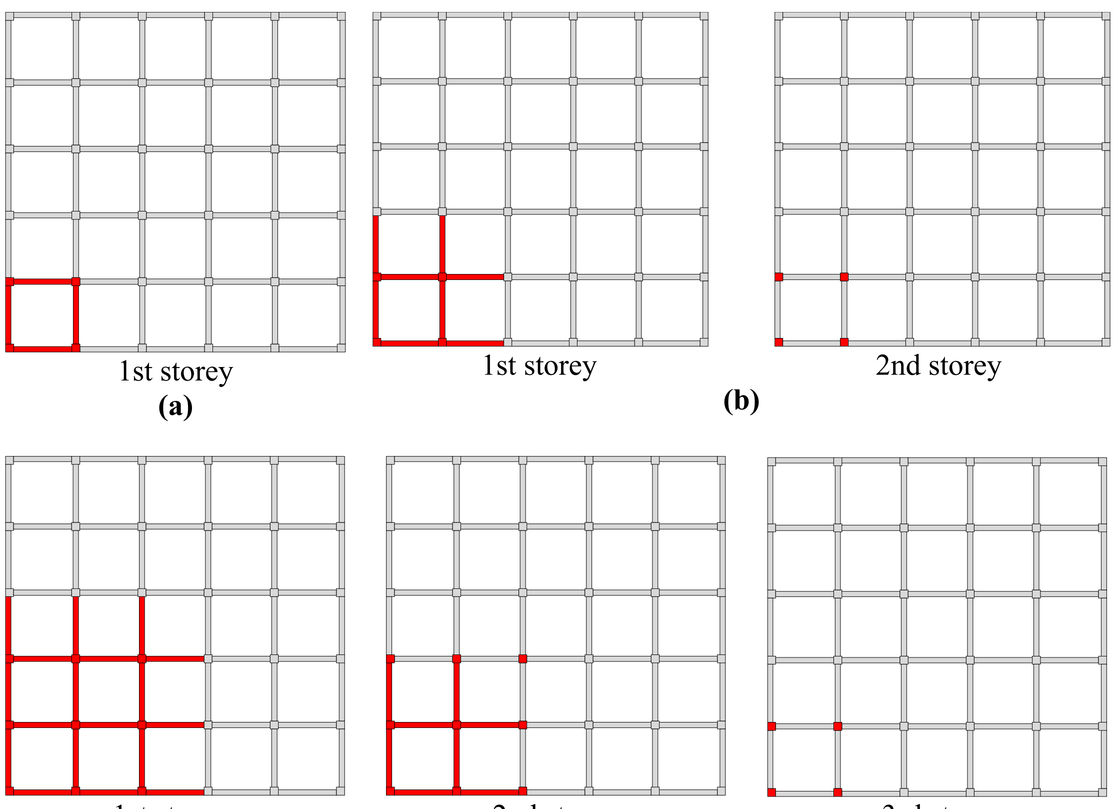


Fig. 2 Static fire scenarios simulated (a) locally elevated temperature (SFS1), (b) intermediate thermal conditions (SFS2) and (c) peak temperature reached (SFS3).

The simulations were performed using the OpenSEES [15] software. The building is simulated using three-dimensional distributed plasticity elements (fiber elements). Beam-column and column base connections are simplified as explained previously. The gravitational loads are modeled as distributed loads on the beams. If the slab is properly designed, it can contribute to the formulation of an alternate path for the loads to be transferred to the neighboring elements. Leading to further reduction of the deflections occurring due to loss of the columns' capacity. In this work, such a design is not considered, so its contribution to the load bearing mechanism is expected to be particularly reduced compared to that of the beams, so it is omitted.

Three pushdown analyses were performed in order to assess the response of the structure. In each analysis, the material properties of each structural element that is affected by the elevated temperatures were altered based on the provisions of EN 1993-1-2 [1]. As progressive collapse resistance criterion, the maximum plastic rotation at the end of the steel beams was taken. Even though the plastic rotation of a single beam might be within acceptable limits, the added effect of multiple beams in a row might result in large vertical displacement particular nodes (e.g. the height of a single storey). Due to the expected large displacements in particularly elevated temperatures, the maximum vertical displacement of the nodes at the affected areas was also recorded, as it would be indicative of partial collapse at a specific area of the building. The results are illustrated in Figure 3 in a plot of the temperature at the starting point of the fire versus the maximum recorded vertical drift of a single beam (representing the plastic rotation at the base of the beam).

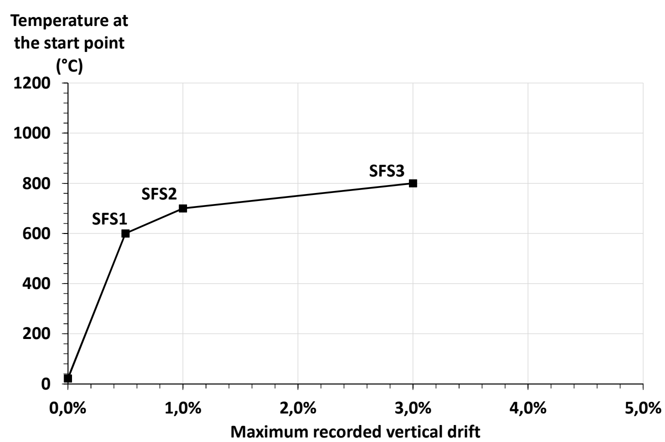


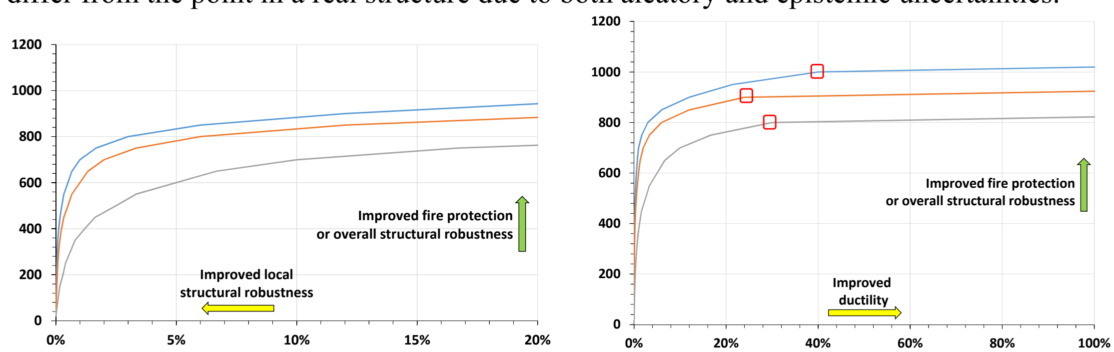
Fig. 3. Maximum recorder vertical drift for each of the three simulated static fire scenarios.

The form of the curve defined in Figure 3 is indicative of the scenarios considered. The maximum recorded vertical drift is particularly smaller than those defined when buildings are evaluated against progressive collapse due to e.g. an explosion, where the affected elements are considered to have failed and are removed from the model. It is of particular interest that in the performed analyses, in the most severe scenario (*i.e.* SFS3), the vertical drift does not exceed the limit defined by UFC 4-023-03 [16] for High Requirements on progressive collapse resistance, that is 10% (corresponding to 6° plastic rotation at the base of the beam).

In the first scenario, where the number of elements affected is small, the evident effect on the structural stability is also very limited, if not nonexistent, as the structural elements' capacity is not particularly reduced, while all neighboring structural elements currently not affected by the fire contribute to the alternate load path developing. When the behavior of more elements deteriorates due to spread of fire to multiple compartments of the building, columns weaken even more, and are not able to receive the load from the beams which are also weakened by the elevated temperatures. This results in a more abrupt increase of the beams' deflection which is now evident. Finally, as the fire reaches its peak value (*i.e.* 800°C in this example), elements in close vicinity with the fire source have lost a major proportion of their capacity and are ineffective. Loads need to be received by neighboring elements only, which are also weakened and are those which develop large vertical drifts. Even though partial collapse might not be assumed, it is evident that severe structural damage has already occurred. At this point, the collapse potential needs to be estimated, as partial collapse of the building might be imminent.

Considering the aforementioned, if the structural performance is not acceptable (e.g. high potential for collapse at not very high temperatures), alternatives might be sought in order to improve its behavior. During initial design, alternatives such as encasement of columns within reinforced concrete might be more attractive to designers, as it allows to take advantage of interaction of two materials in the composite section, resulting in reduces structural cost as well. When existing buildings are retrofitted, the same techniques are available as well, but due to constructional difficulty their only their efficiency as fire protection measures should be considered. Alternative solutions could include the installation of active and passive fire protection measures, as well as the strengthening of particular elements which are expected to contribute to the development of an alternate load path when various load bearing elements have weakened substantially due to elevated temperatures.

Using a temperature versus collapse resistance criterion curves (Figure 4), one can acquire a satisfactory illustration of the building's inherit fire-resistance. In order to improve the performance of the building, it should be able to sustain higher temperatures without substantial increase to the collapse potential. Strengthening of the structural elements that are directly affected, can displace the curve closer to the vertical axis. This does not necessarily lead to increased temperatures that result in local collapse, as the particular elements typically fail due to buckling before the temperature reaches its peak, while the remaining structural elements receive the loads. Increasing the overall robustness of the building allows for the development of multiple alternate load paths and so increases the structural damage the building may sustain. Consequently, the elements affected by the fire might fail, but the building does not collapse thanks to these mechanisms. Of particular interest is also the point which indicated the collapse of the structural model. This might differ from the point in a real structure due to both aleatory and epistemic uncertainties.



(a) **(b)**

Fig. 4. Maximum recorded vertical drift versus temperature at the affected elements (a) displacement of the curve for improved design, (b) displacement of the maximum vertical drift at collapse.

Nevertheless, it is indicative of the overall resistance of the building. The closer this point is to the vertical axis, the less the overall ductility of the building is. Comparison between the performance of the building under an actual fire scenario and the simulated performance can be used in order to decide whether it is more preferable to repair the building or demolish it.

4. STRUCTURAL PERFORMANCE UNDER TIME-DEPENDENT FIRE SCENARIOS

In actual structural fires, collapse has occurred even under practically stable thermal conditions, after an adequate amount of time passed. This is mainly due to deterioration mechanisms that require a certain amount of time to take place. Fire-resistant paints and sprays can sustain fire for some time and slow down the heating of the sections they protect. However, as the temperature rises, their resistance reduces, until they burn out or melt away. Additionally, various connection components reach close to failure, but this does not take place until a micro-vibration within the structure occurs, which results in exceedence of their capacity. Such incidents are not easily modeled, but their occurrence potential increases with time.

Hence, in order to assess the performance of the building more accurately, a suitable time-dependent fire pattern needs to be determined. This time-dependent fire pattern includes the thermal conditions in all affected structural elements as a function of time. Additionally, it might include functions to adjust the internal stresses of the elements based on the elapsed time. For illustration purposes, an indicative fire pattern is selected which considers a fast outburst of structural fire, which slows down as the temperature rises (Figure 5). The time-versus-temperature data of this pattern are combined with the data of vertical drift versus temperature used for the most resilient building in Figure 4 (blue curve). The results are illustrated in Figure 6.

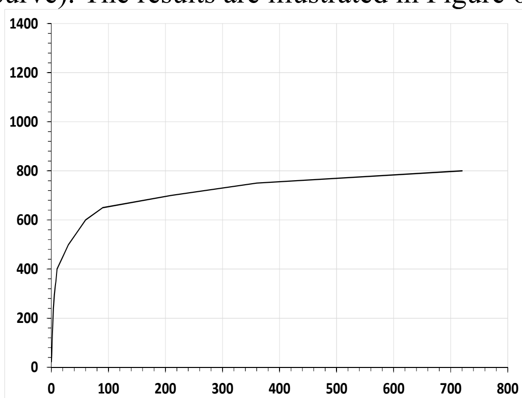


Fig.5 Temperature rise over time.

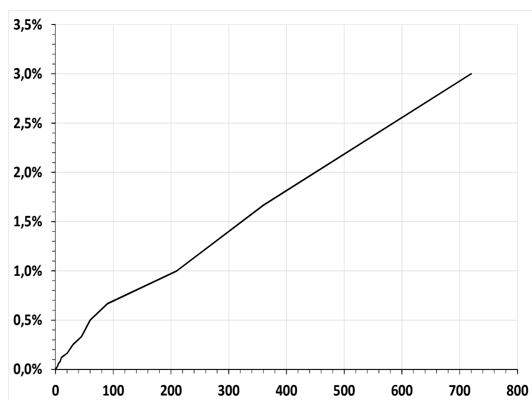


Fig.6. Maximum deflection over time.

The curve obtained in Figure 6. Is indicative of the influence of both considered parameters in the actual response of the building. In the assessed scenario, the maximum vertical drift seems to have a linear correlation with the time lapsed. This is because of the considered relationship between structural temperature and building response and the temperature rise as a function of time. The rate with which temperature increases over time is inversely proportional of the rate of building performance deterioration over the applied temperature. However, the relationship between time and displacement is not random, but depends mainly on the temperature as a function of time. Even though the correlation of structural performance and fire scenario can be anticipated, the fire pattern depends on various parameters, such as the initial causes of the fire, the potential fuel or other flammable material that will sustain or increase the temperature levels and even cause abrupt jumps in the pattern, the building characteristics (easy or poor air flow, active or passive fire protection measures, etc.).

5. CONCLUDING REMARKS

- Evaluation of the building performance under appropriate fire scenarios can allow for the determination of the required design or retrofit strategy in order to increase its collapse resistance when structural fire occurs.
- Structural performance under real fire scenarios depends substantially on the characteristics of the fire.
- Real time monitoring and analysis with suitable software can improve the decision making process, based on the calculation of the collapse risk at each point of time.
- Further research is required in order to associate the cause of fire with the structural response and improve the modelling of time-dependent analyses under structural fire (transient thermal analyses).

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ΕΚΤΙΜΗΣΗ ΤΟΥ ΚΙΝΔΥΝΟΥ ΓΙΑ ΠΡΟΟΔΕΥΤΙΚΗ ΚΑΤΑΡΡΕΥΣΗ ΜΕΤΑΛΛΙΚΩΝ ΚΑΤΑΣΚΕΥΩΝ ΥΠΟ ΣΥΝΘΗΚΕΣ ΦΩΤΙΑΣ

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ΠΕΡΙΛΗΨΗ

Η παρούσα εργασία παρουσιάζει μια μεθοδολογία εκτίμησης της κατάστασης της κατασκευής και της απομένουσας ικανότητάς της με τη χρήση των θερμικών δεδομένων. Η προτεινόμενη μέθοδος αποτελεί μια προσπάθεια συσχέτισης των χαρακτηριστικών του κυρίως αιτίου (δηλαδή της αυξημένης θερμοκρασίας λόγω φωτιάς) και της απόκρισης της κατασκευής. Τονίζεται η επιρροή της θερμικής χρονοϊστορίας που χρησιμοποιείται όταν πραγματοποιείται ανάλυση της συμπεριφοράς του κτιρίου με βάση το χρόνο. Με τη μέθοδο που παρουσιάζεται είναι δυνατή η επιλογή στρατηγικής επέμβασης χωρίς την απαίτηση επιτόπιου ελέγχου που συνήθως είναι αδύνατος, καθώς και της έγκαιρης εκκένωσης όταν η κατάρρευση της κατασκευής είναι αναπόφευκτη.

Λέξεις κλειδιά: χάλυβας, φωτιά, προοδευτική κατάρρευση, απομένουσα ικανότητα