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Investigation of fire-induced collapse scenarios for a steel high-rise building

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ABSTRACT

The paper deals with the problem of understanding and evaluating the structural response of steel buildings to fire and outlines a general framework for the structural fire safety design of high-rise building. Among all building typology, the fire design of high-rise buildings is particularly challenging with respect to both non-structural and structural design aspects: the enhanced design difficulties in providing i) a safe and prompt vertical evacuation of the building and ii) an effective vertical compartmentalization for avoiding vertical fire spread, refer both to non-structural aspects (architectural design choices and active measures) and won't be investigated in detail in this paper; the paper focus instead on structural design aspects, with specific reference to the iii) enhanced susceptibility of high-rise buildings to disproportionate collapse, due to the significant vertical elevation of the structural system and to the complex and often untraditional design.

With reference to fire-induced collapses, a particular dangerous situation is represented by the spread of failures to elements not directly involved in the fire, i.e. element that due to their location or because of greater insulation have still a relatively low temperature at the time of failure: in this respect, the example of a high rise building is presented in the paper, where, depending on the fire scenarios considered and as different beam-column stiffness ratio, fire damages can remain localized to the heated zone or involve other elements in the failure.

Disproportionate damages induced by fire can therefore be avoided with a robust design of the structural system. This requires however a study of the response to fire of the structure as a whole, which is a quite difficult task, especially for complex structures such as high-rise buildings. Aim of the paper is thus to exemplify a method for performing this kind of investigations and to outline problematic aspects in the modeling and in the interpretation of the results.

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1. INTRODUCTION

Among all building typology, the fire design of high-rise buildings is particularly challenging (Craighead, 2003) with respect to both non-structural and structural design aspects (Fig. 1): the enhanced difficulty of i) evacuating the building safely, which is mainly due to the vertical elevation and of the crowding of the premises, as well as of ii) limiting the fire development, which is a consequence of the higher risk of vertical fire spread, refer both to non-structural aspects (architectural design choices and active measures), and won't be investigated in detail in this paper; the paper focus instead on structural design aspects, with specific reference to the iii) enhanced susceptibility of high-rise buildings to disproportionate collapse, due to the significant vertical elevation of the structural system and to the complex and often untraditional design.

1.1. Structural measures for fire safety

The importance of including passive measures in fire design, in addition to (and not in alternative of) the active ones is due to the necessity of limiting the consequence and not only the occurrence of a structural fire. In this respect, several structural fires occurred in buildings where fire protection systems were present but not properly operative, e.g. due to incomplete installation (First Interstate Bank, Los Angeles, 1998) or faulty or maintenance (Parque Central tower, Caracas 2004), or where a sprinkler system was foreseen but not yet installed (Mandarin Hotel, Beijing 2009). As witnessed by few other cases of fire in multi-storey buildings (WTC, U.S. 2001; Windsor Tower, Madrid, 2005; Delft University, Delft 2009), structural fires may have very severe consequences and lead to major collapses of the structural systems, with much higher societal costs due to the loss of the property and often an higher number of injuries.

With reference to fire-induced collapses (Bontempi et al., 2010), a particular dangerous situation is represented by the spread of failures to elements not directly involved in the fire, i.e. element that due to their location or because of greater insulation have still a relatively low temperature at the time of failure, such in some of the examples mentioned above, where the initial failure of few heated members led to a progressive collapse of the floors, which fell one over the other, in a pancake type collapse modality (Starossek, 2009).

1.2. Structural integrity in case of fire

The ability of the structural system to undergo limited damages under accidental actions (such as structural fires) is nowadays demanded by many regulations (ASCE 7-02, 2002; ISO/FDIS 2394, 1998; EN 1991-1-7, 2006) and referred to as structural integrity requirements in the paper (Giuliani, 2009). Most codes and guidelines however do not provide detailed methods for assessing and improving the response of structural system to fire, which, particularly in the case of complex structures such as high-rise building, is a quite difficult task (MSB-6, 2008). This is due to the fact that simplified method for fire safety design can hardly be applied to this kind of structures, since the response of single elements to fire can be very different from the response of the building as a whole (Usmani et al, 2000). On the other side, advanced design methods for structural fire safety generally imply the use of multi-physics finite element codes and require much more effort in term of time modeling experience, due to a strongly nonlinear response induced by material degradation and large displacement, and to possible triggering of buckling or other local mechanisms.

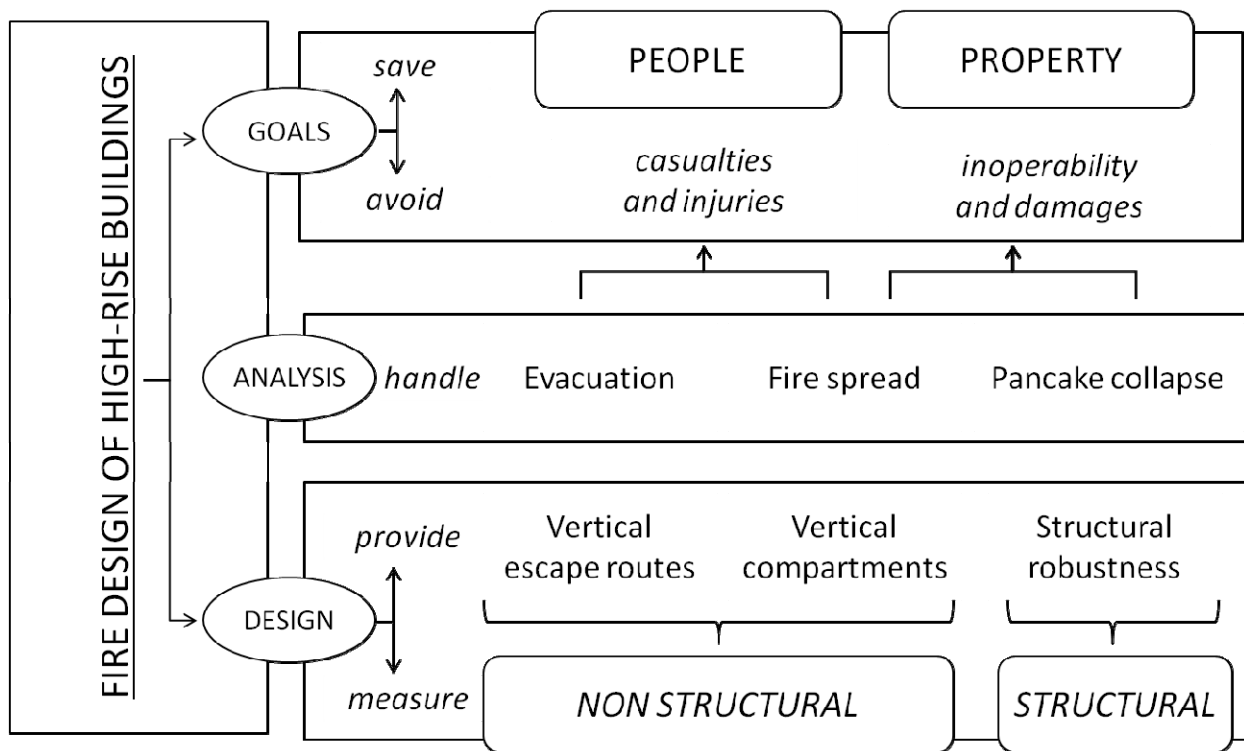


Fig. 1. Goals, challenges and safety measures for the design of high-rise buildings against fire.

2. INVESTIGATION OF THE STRUCTURAL RESPONSE TO FIRE

Aim of the paper is thus to exemplify a method for performing this kind of investigations in order to evaluate and improve in case of need the structural response of steel buildings to fire. In particular, problematic aspects in the modeling and in the interpretation of the results have been outlined and discussed in the presentation of the results.

2.1. Case study

The building considered as case study is a steel high-rise building, whose premises are devoted to offices and residential use. The building has been designed on the basis of the geometry and characteristics of a building recently built up in Latina, Italy (Fig. 2). The building is composed of 40 stories and has a framed structural system. A vertical bracing system provides stiffness against horizontal actions, while no horizontal bracing system is foreseen within the floor planes, since a bidirectional concrete floor slabs provide the necessary in-plane stiffness. The inclusion of hollow spheres in the concrete, together with the biaxial symmetry of the slabs, allows for long beam spans and small beam profiles that can be contained within the height of the slabs, so that relatively slender beams are used, in comparison with the dimension of the columns, whose profile need to be quite big, particularly at the bottom of the buildings.

The beam slenderness can represent a problem for fire safety during the construction of the building, when fire protection systems are not yet installed and the insulation and horizontal restraint provided by the concrete slab is not present. This situation is of interest from the point of view of structural fire safety provisions and has been thus considered for the investigations.

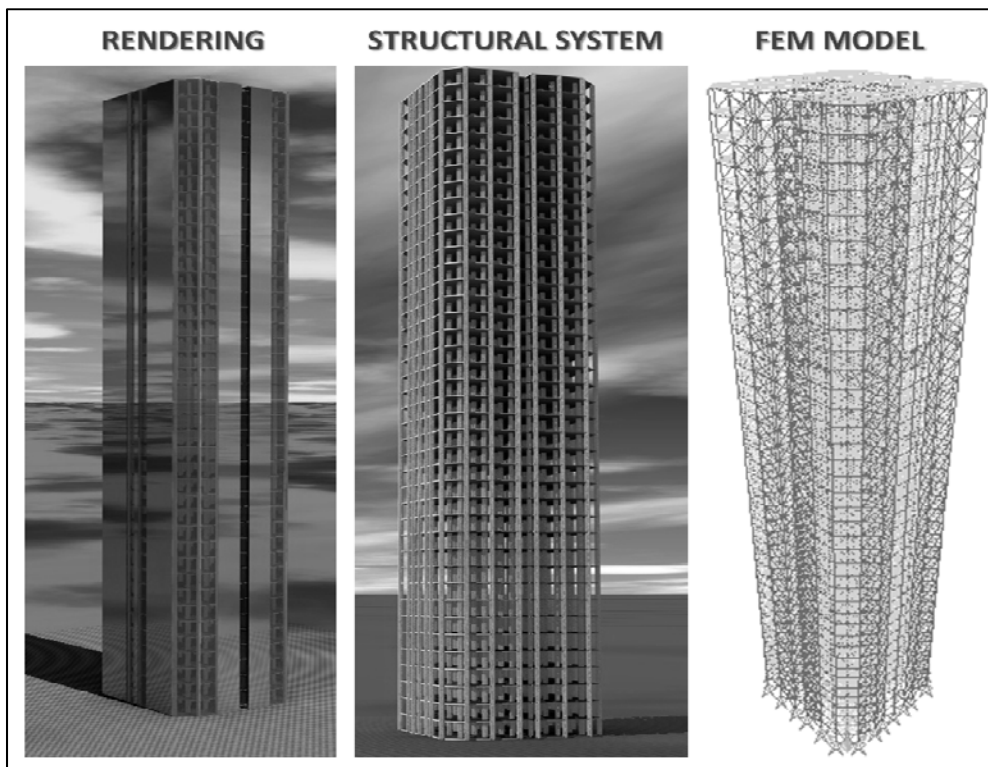


Fig. 2. Rendering and FEM of the investigated high-rise building.

2.2. Methodology

When the complexity of the structure is such that prescriptive rules and simplified design procedures cannot be followed, a Performance-Based Fire Design (Pbfd) approach can be used with respect to structural fire safety aspects. The procedure is broken down in few subsequent steps exemplified by the flowchart of Fig. 3.

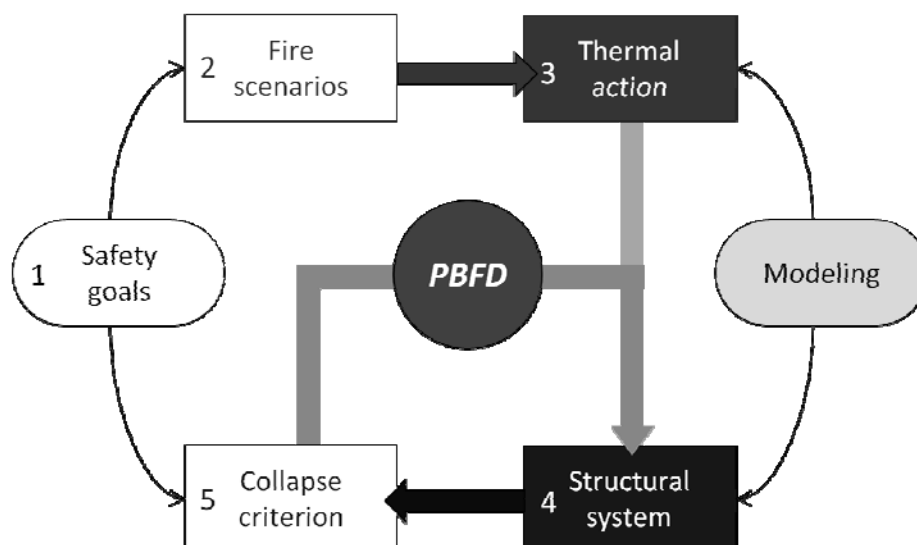


Fig. 3. Analysis steps for Performance Based Fire Design (Pbfd)

Safety goals

The starting point is the definition of safety goals, which in the presented case are limited to the prevention a major collapse of the building due to fire, which could occur in a construction stage. Limited local damages are therefore accepted, even if they imply the total loss of some elements. From the point of view of a general fire safety design however, it can be also of interest to avoid high costs due inoperability of the premises and significant repair of the structural system. However, a fire during a construction stage these costs are limited and the number of people in the building is limited to few people, so this safety objective can be considered acceptable. This is even more true considering that fire which occurs during the building construction is a very unlucky event, which represents though a quite severe design condition, since the structure is most likely not yet insulated nor protected by a sprinkler system.

Fire scenarios

The second step concerns the identification of relevant fire scenarios. In literature, the identification of design scenarios is often obtained by means of a risk analysis (Faber et al, 2003; Nii et al, 2010), which is however a relatively onerous procedure and in case of Low Probability - High-Consequence (LP-HC) events is complicated by the fact that most probable scenarios are not necessarily the most severe ones in term of consequences and costs. Therefore in the practice, few fire scenarios are often pragmatically identified on the structure (Gkoumas et al, 2008) on the basis of engineering experience and preliminary simplified structural investigations.

For this study the fire is considered to be localized to some area within a single floor at the time. Since floors have different elements and loads however, different fire scenarios along the building height have to be considered, in order to highlight possible differences in the propagation of the collapse due to different structural sub-systems. In this paper, only the results related to the 1st and 2nd fire scenarios will be presented, with reference to the 5th and 35th floors.

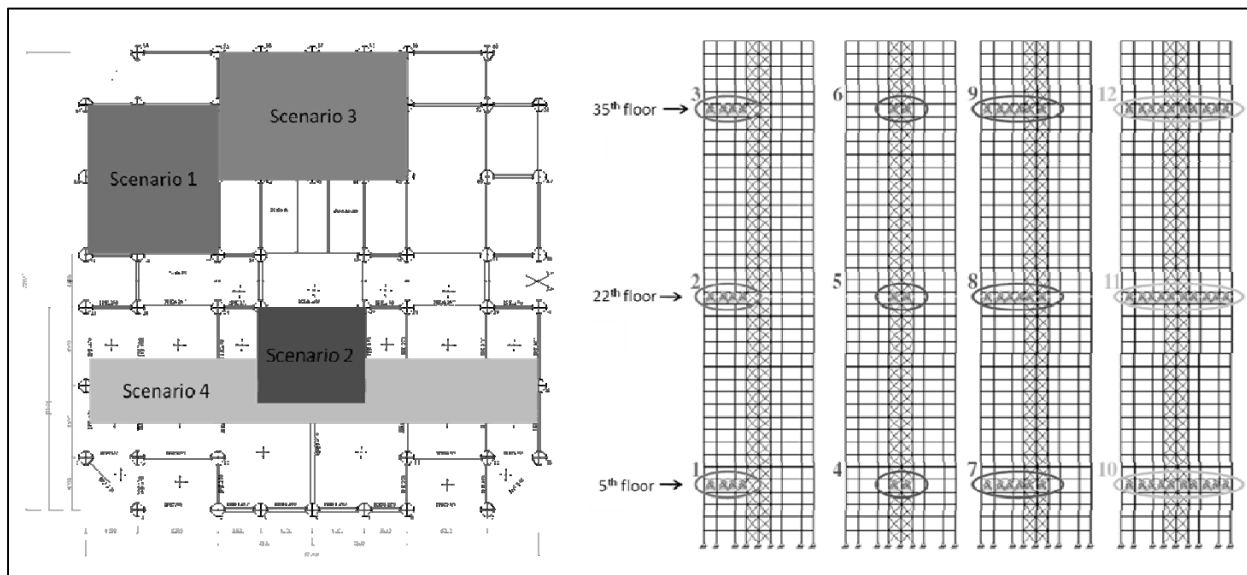


Fig. 4. Fire scenarios within the floor and along the elevation of the building.

Thermal action

Once the fire scenarios have been chosen, the fire action has to be modeled. More or less

realistic temperature-time curve can be considered for the fire (natural, parametric or nominal fire). In this study a nominal fire model has been used for the sake of simplicity, in the form of the standard ISO 834 fire curve. In the results presented in the following, only the beams included in the area highlighted in Fig. 4 are considered to be heated by the fire.

The heating curves of those beams have been calculated under the assumption of uniform temperature in the element, according to the Eurocodes formula for unprotected steel (EN1993-1-2, 2005), using a convective coefficient $\alpha = 25 \text{ W/(m}^2\text{K)}$ and a total emissivity $\varepsilon = 0.5$ (the shadow effect of the profiles is neglected on the safe side).

Modeling aspects

Modeling in detail such a big and complex structure can be quite onerous in term of analysis time, but also in term of difficulties in the interpretation of results, which is the main goal of each analysis: in order to understand properly the structural behavior and also be able to check the validity of the outcomes, it's important to simplify the model as much as possible and then refine it at further stages in case of need: in this study, simple substructures have been investigated first and then structural models have been gradually refined, on the basis of the expected or identified mechanisms, which should be better highlighted or represented (Fig. 5).

In this respect, as mentioned above, a central point of fire-induced investigations concerns the identification of a possible spread of the local damages from the heated members to elements not directly involved in the fire. In a 3D building the collapse propagation can occur both within the floor plane where the fire has triggered and along the building elevation.

Two different type of substructure have been therefore considered for this purpose: i) a floor model, where the direct effect of fire on heated beams can be evaluated (vulnerability to fire) and then the consequence of a possible failure of the heated beams on the rest of the floor system can be investigated (structural robustness of the system); ii) a frame model, where a possible overloading and collapse of the columns consequent to beam failure could be investigated. If the possibility of collapse propagation is highlighted both within the floor plane and along the vertical elevation, then the validity of the results obtained by the investigation of the single substructure model could be invalidated, due to rough modeling of 3D effects on the frame model and of column boundary conditions in the floor model. A refined model, such as iii) a multi-storey floor model could be implemented, where the number of the floors considered depends on the expected vertical involvement but is also limited by the number of nodes and element in the model, which could strongly increase the solution time and make the study inefficient.

Concerning the floor substructure, it is important to point out that in order to maintain the model simple and to better highlight the structural characteristic of the beams and columns that play a role in the vulnerability to fire and robustness of the structures, the floor slabs have not been modeled in a first stage. This condition can be representative of a construction stage, where the beams, which are not insulated, are left unprotected by the absence of the floors. Furthermore, in order to outline the effects of different beam column stiffness ratio, several distinct floor levels have been considered in the investigation. In the following, the outcomes related to the investigation of 5th floor and 35th floor will be presented as most relevant ones.

Outcomes related to the frame model, did not evidence a vertical propagation of the collapse and will not be presented here. However, from a methodological point of view, the

importance of a 3D frame model has to be stressed, since, especially if the presence of floor slabs is not considered, possible out-of-plane displacements should be duly represented, in case of buckling failure of the beams.

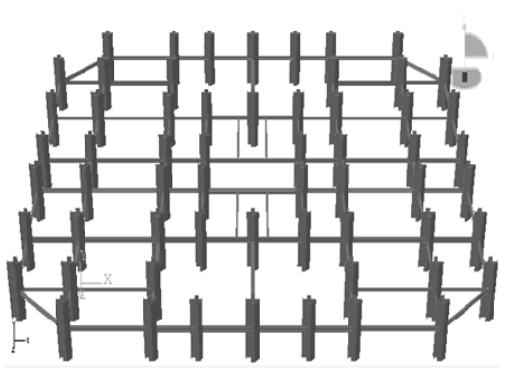
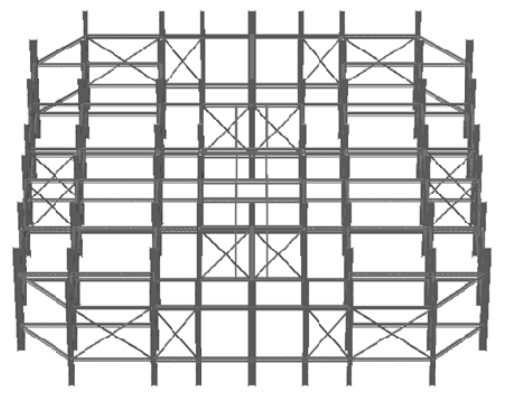

MODEL TYPE	FINITE ELEMENT MODEL	NODES	ELEMENTS	D.O.F.
#1 FLOOR MODEL SINGLE STOREY		2358	954	9556
#2 FLOOR MODEL TWO STOREYS		7610	2392	14988
#3 FRAME MODEL ALL STOREYS		11477	3536	22638

Fig. 5. Different models used for investigating the building response.

Structural response

The analyses take into account thermo-plastic material and geometric nonlinearities. Dead

and live loads pertinent to beams are applied as line forces and considered in a first static analysis step, together with self-weight. In a second load step the temperature-time curves obtained by heating each steel profile with the standard fire are applied to the nodes of the elements directly considered in the fire, depending on the fire scenario considered, while other elements have been assumed to remain cold throughout the investigation. An implicit dynamic solver has been used in order to overcome convergence problems due to local mechanisms and be able to follow the propagation of failures.

Collapse condition

The last step concerns the interpretation of the results and the identification for each fire scenario of collapse conditions, i.e. the conditions that represent the overcoming of the safety objectives identified at the beginning of the design process. In the presented case, the collapse condition is represented by a well-identifiable circumstance; however, the identification of the collapse condition can be more difficult, depending on whether the safety objectives are aimed at i) preventing major damages, or ii) limiting local damages too, or iii) maintaining even the functionality of the premises.

In the first case, a collapse condition which is representative of the failure of one or few elements is the runaway of a significant point of the structure, with this term meaning the accelerating and irreversible downward displacement of the considered point (Usmani et al, 2003). In the second and third case, a nominal collapse can be assumed, which is often defined as the element displacement that overcomes a given limit. Different values for this limit are reported in literature, depending on the safety level considered (Pettersen et al, 1976; Gentili et al, 2011).

2.3. Main results

Among all the performed investigations, the outcomes related to the 1st and 2nd at the 5th and 35th floor are more relevant for the paper purpose and are presented in the following.

Fire scenario #1

The 1st fire scenario is characterized by an early failure of the horizontal elements directly involved in the fire, which, due to the absence of the restraint provided by the slab, buckle out of plane after few minutes of fire. The failure of the heated beams occurs at very low temperatures (around 100 °C), when no degradation of the steel mechanical properties, which typically plays a determinant role in fire-induced collapses, has occurred yet. The beam failure therefore can be entirely ascribed to the eigenstresses induced by the hindered thermal expansion consequent to the strong column - slender beam frame type: specifically, the first three collapsing beams have IPE270 profiles, while the columns adjacent to them have HEM1000* profiles.

It seems relevant to highlight the fact that the very early failure of the beams prevents the redistribution of high stresses on the columns, as visible in the bottom part of the figure. Therefore the high vulnerability of horizontal elements to fire is accompanied by a robust behavior of the vertical load carrying system. A possible buckling of the columns, whose buckling length suddenly increases when the horizontal restraint provided by the beam is lost (Usmani et al, 2003), doesn't seem a concern in this specific case, due to the low loading condition and the very high stiffness of the column at the 5th floor.

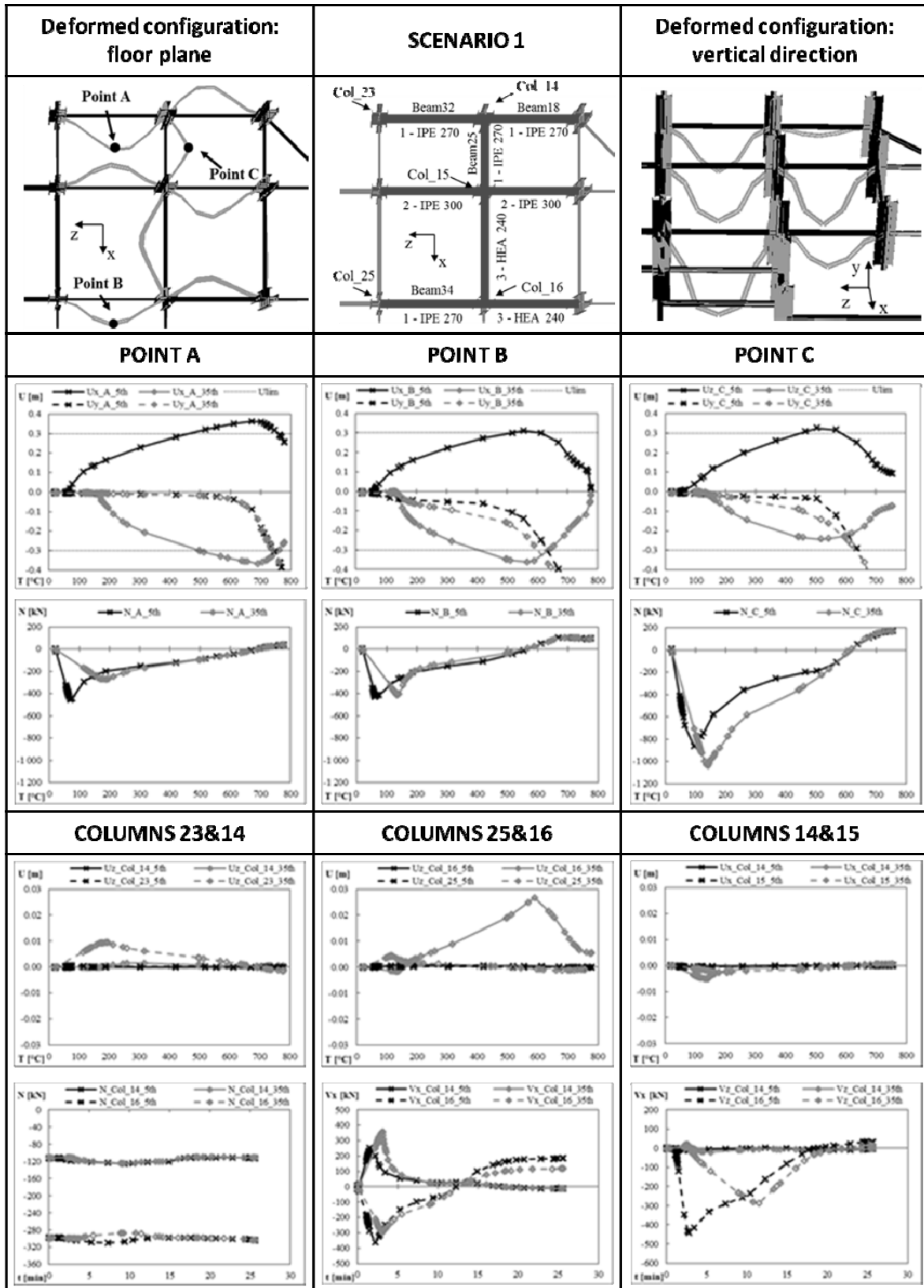


Fig. 6. Displacement and stress of beams and columns at 5th and 35th floor for fire scenario #1.

In the top part of Fig. 6, the deformed configuration at the 5th floor is shown with reference to the displacements in the floor plane (left) and in the vertical direction (right). In the central part of the figure, the trend of horizontal and vertical displacement with the temperature is reported for the mid-span point of three of the heated beams (indicated as points A, B, and C in the top part of the figure). Underneath, also the internal axial force of the three beams is reported. In the bottom part of the figure, the horizontal displacement of the top node of the two columns connected to each one of the three beams (columns 25 and 15, 23 and 14, 14 and 15) are shown. The increment of axial stress in the columns is also reported underneath.

In Fig. 6 the differences between the element displacements and forces at the 5th (black lines) and 35th floor (colored lines) are reported. As visible in the central part of the figure, the out-of-plane buckling occurs later for all the beams at the 35th floor as a consequence of the lower restraint provided to the beams by the columns, which have a much smaller profile at the 35th floor (HEM400) than at the 5th floor (HEM1000*). Therefore, while the columns at the 5th floor completely hindered the thermal expansion of the beams, the columns at the 35th floor are slightly displaced outwards under the thrust of the beams, which therefore resist slightly longer to the fire.

Fire scenario #2

The outcomes for the 2nd fire scenario at the 5th and 35th floor are shown in term of deformed configuration and forces and displacement of heated members in Fig. 7.

Both in case of the 5th and 35th floor, a buckling mechanism occurs, which involves three of the heated beams (whose mid-span is indicated with point D and E, since two beams have a symmetrical behavior). It is interesting to notice that the outer horizontal beams (indicated as beam 65 and 68 in the top part of the figure) buckle, while the inner horizontal beams (indicated as beam 66 and 68) don't, since they are not connected to a column on one side but lay over a beam (indicated as beam 58) which provide a lower horizontal restraint and allow for greater thermal expansion, preventing an increment of the axial stresses.

As in the 1st fire scenario, the collapse mechanism at the 35th floor is delayed with respect to the 5th floor. However a significant difference with respect to the 1st fire scenario is represented by the fact that the buckling mechanism is different for the two floors: the beams at the 5th floor buckle out of the plane, while the same beams at the 35th floor have time for developing a significant vertical displacement before failing and the buckle occurs therefore along the vertical direction.

Another particular aspect is that at 35th floor a propagation of the failures to two cold beams (whose mid-span is indicated with points F and G in the top part of the figure) can be evidenced. The higher buckling resistance of single beam members, which is a consequence of the lower horizontal restraint provided by the columns, leads to the occurrence of a different and less local buckling mechanism, which involve 8 beams together and 2 beams that fall outside the area directly heated by the fire

It is also interesting to highlight the following consideration: a high vulnerability of the system has been ascribed to the very stiff columns (as at the 5th floor), which by preventing the thermal expansion of beams could lead to an early buckling mechanism (as in the 1st fire scenario); on the other side, more slender columns (such as at the 35th floor) may lead to a delayed buckling and have therefore a positive effect on the overall resistance. However, this

situation can also be detrimental, as shown in the 2nd scenario, where the higher resistance of beams to a local buckling determines the triggering of a bigger buckling mechanism, which also involves elements not directly affected by the fire.

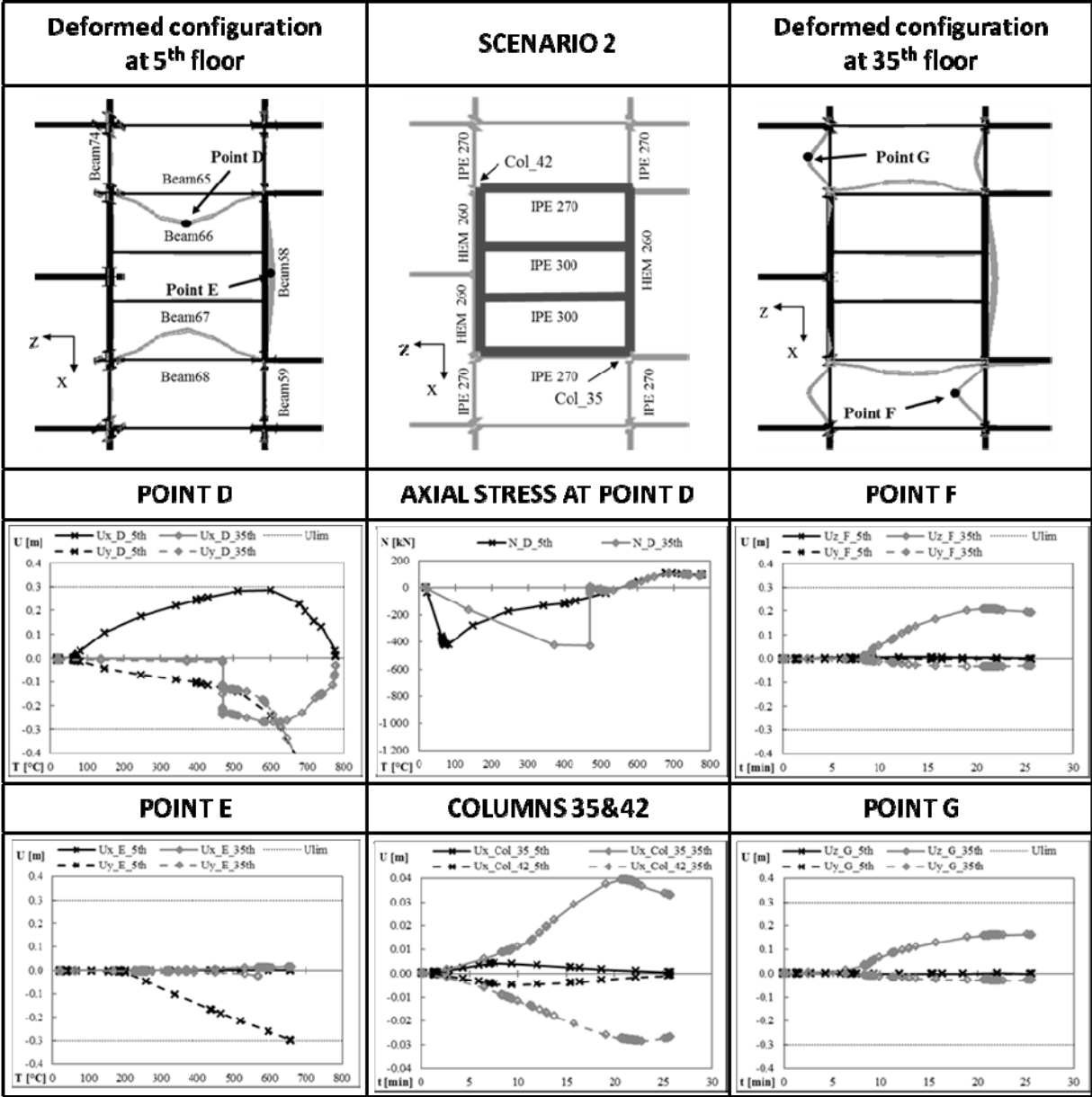


Fig. 7. Displacement and beam and columns and axial stress of one beam at 5th and 35th floor for fire scenario #2.

3. CONCLUSION

3.1. Potentiality and limitations of advanced design methods

With reference to fire-induced collapses (Bontempi et al., 2010), a particular dangerous situation is represented by the spread of failures to elements not directly involved in the fire, i.e.

element that due to their location or because of greater insulation have still a relatively low temperature at the time of failure. This situation can lead to a disproportionate structural damage, such as shown by some recent case of structural collapse mentioned in the first section, where the initial failure of few heated members led to a progressive collapse of the floors, which fell one over the other, in a pancake type collapse modality (Starossek, 2009).

In this respect, the example of a high rise building is presented in the paper, where, depending on the fire scenarios considered and as different beam-column stiffness ratio, fire damages can remain localized to the heated zone or involve other elements in the failure. It is important to stress out that the spread of the collapse is very dependent on the structural characteristic of the elements and their organization within the system.

Disproportionate damages induced by fire can therefore be avoided with a proper robust design of the structural system, which, at least for complex structures seem to be an important requirement even when the probability of occurrence of a structural fire and consequent local damages is very low, as if fire protection systems are foreseen in the building. This requires however a study of the response to fire of the structure as a whole, which is a quite difficult task, especially for complex structures such as high-rise buildings. Furthermore, even if the demand of structural integrity is stated in several national and European regulations, as mentioned early, advanced method for assessing and improving this structural requirement are not clearly formulated by codes and guidelines and are in general very dependent on the type of structural typology and investigation to be performed.

3.2. Summary of the investigation outcomes

In this paper, the investigations of an high-rise building for fire-induced collapse are described in detail, with respect to the modeling procedure and the analyses performed: the reason of simplified assumptions in the modeling and consequent limitations in the results are presented and discussed and some important aspects concerning the definition of collapse condition and interpretation of the results are highlighted.

Among all performed investigation, few significant outcomes have been highlighted in particular, which are summarized below.

- 1 When slabs are not present (e.g. construction stage), the stiff column – slender beam structural system (5th floor) seems very vulnerable to fire, due to the very early failure of beams that buckle at quite low temperature, as a consequence of the hindered thermal expansion.
 - a. In this respect, it is important to point out that these failures could possibly not be detected by following prescriptive design rules, since according to European standards (EN1991-1-2, 2004; EN1993-1-2, 2005) consideration of thermal expansion could be neglected, provided that standard fire is used.
 - b. Specific provision in the design of connections details should therefore been foreseen for allowing thermal expansion of slender beams, at least when simplified design methods are used.
- 2 A relatively robust behavior counterpoises to the high vulnerability of the system, since:
 - c. a vertical propagation of the collapse seem unlikely, due to the very modest redistribution of stresses on the column

- d. within the floor, the effects of fire remains mostly localized to the area directly involved to the fire
- 3 Where the stiffness difference between beam and column is less pronounced (35th floor), the possibility of a propagation of the failures within the floor area is evidenced (2nd fire scenario). Here, the higher buckling resistance of single beam members, which is a consequence of the lower horizontal restraint provided by the columns, leads to the occurrence of a different and less local buckling mechanism, which involve several beam spans, also outside the area directly heated by the considered fire. This mechanism seems particularly dangerous, since, when simplified fire design methods are used, verifications are carried on single elements and global buckling mechanisms could be neglected.

The outcomes summarized above present several limitations, due to the simplifications (substructures and boundary conditions) and assumptions (actual loading and resistance of the materials, limited number of fire scenarios) used in the investigations and further studies are currently being undertaken on more refined models of the building, which include the consideration of floor slabs and the effect of fire on two adjacent floors. However, the presented results are of interest in the view of describing a general methodology for carrying on fire investigation on complex structure, more than in the view of foreseeing the exact behavior of the specific structure considered.

3.3. Fire induced collapse mechanisms

From the point of view of robust design provisions, it seems more relevant to try and highlight some common characteristics of structural systems that play a role in the type and severity of collapse mechanisms, as summarized in Fig. 8, where a generalization of the results of investigations undertaken by the authors (Budny et al, 2010; Gentili et al, 2011) and of several other outcomes reported in literature (Gillie et al, 2002; Usmani et al, 2003; EUR 24222, 2007; Song et al, 2009) is attempted and described in the following.

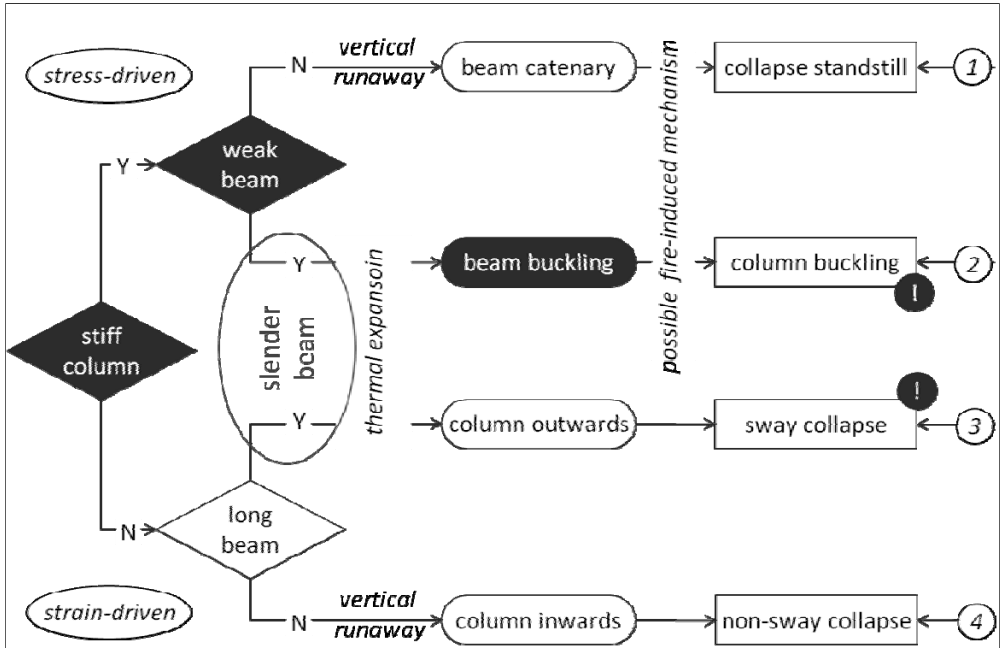


Fig. 8. Failure mechanisms of steel framed buildings.

In the figure, the colored boxes indicate the collapse mechanism highlighted for the investigated structure, where the hindered thermal expansion of the beam leads to a buckling collapse mechanism within the floor. Even if not relevant for this specific building, the loss of horizontal restraint given to the column by the failed beams could in principle induce a buckling failure of the column and lead to a progression of the collapse downwards (Usmani et al, 2003).

Another particularly severe circumstance (exclamation marks are used in Fig. 8 to highlight dangerous collapse mechanisms) is the opposite one, i.e. the case where thermal expansion of horizontal members is very significant and capable of significantly displacing the columns outwards, such as in case of very long or pitched beams. In the case of single story buildings (EUR 24222), a sway collapse (Song et al, 2009) can trigger, if the thermal expansion is prevailing on the material degradation (i.e. the horizontal displacement on the vertical runaway) and a collapse condition can be identified before the beam-column connection point starts moving inwards due to occurrence of large vertical displacements and nonlinear geometric effects (Gentili et al, 2011).

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