Resilience of tall steel moment resisting frame buildings with multi-hazard post-event fire

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\textbf{ABSTRACT}

Infrastructure resilience is the ability of an infrastructure asset to limit the effect and duration of damaging extreme events. The four main components of the concept of resilience are robustness, resourcefulness, recovery and redundancy most of which are very difficult to be quantified. In addition to this difficulty, a careful study of extreme event cases can demonstrate that it is very common for an extreme event scenario to include cascading multi-hazard events such as blast, floods, earthquake, and fire. This paper studies the resilience of a multi-story steel frame with multi-hazard considerations which include a post-event fire scenario. The initiating extreme event is simulated through the threat-independent alternate load path method of analysis and a post-event fire is considered following the extreme event. The work in the paper combines previous work by the authors on stability-induced collapse of damaged steel structures and closed-form solutions for temperature predictions of wide-flange components during a fire scenario. For the purposes of the fire scenarios, new fire time-temperature curves are developed based on experimental data from the well-known Cardington fire tests. The results show that even when a structure can withstand an extreme event scenario, a post-event fire consideration is highly critical to evaluate the remaining time of survival of the structure before collapse. It is shown that the sequential method of multi-hazard analysis can lead to very short available time periods before the post-event fire leads to the complete collapse.

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1. Introduction

The concept of infrastructure resilience is relatively new for civil engineering, but at the same time it is fundamentally important. During the last decade, several researchers (Alampalli and Ettouney [1], Bruneau and Reinhorn [2], Cimellaro et al. [3] and Ettouney and Alampalli [4]) have started approaching the problem of quantifying the concept of resilience in order to fully understand how can a system be more resilient than another and what techniques should be used to improve the resilience of existing systems.

The resistance of a system to maintain critical operations during the crisis is called robustness (or otherwise the capacity/resistance to progressive collapse). This property is considered as one of the major components of resilience and involves the structural system itself and its design, but also its interdependency with other structural systems and networks. Structural robustness regards the resistance of a structural system to an unexpected appearance of damage within it, due to an extreme event scenario. Robustness is the only component of resilience which has been somewhat quantified, since the vast majority of the infrastructure resilience research has been focused on this concept. In this direction, two major regulatory documents have already dominated the field of structural robustness (DoD [5] and GSA [6]). These documents include methods of analysis for performing progressive collapse analysis for structures (buildings) that are undergoing an extreme event. Since the range of hazardous extreme events is extensive, the two documents define the threat-independent requirements and the alternate load path method of analysis which introduces damage into the system through the notion of key component removal. For the case of building structures, the key components are columns.

All the above constitute the efforts of the research community to analyze structures for extreme (mainly man-made) hazardous events. Among the many incidents of progressive collapse of buildings, some of the most widely known events related to blast or impact are the World Trade Center Collapse in New York in 2001 as a result of the impact of two airliners on the Twin Towers, the 1995
Oklahoma City bombing, the 1968 Ronan Point collapse in the UK, the recent 2014 collapse of a building at New York's Harlem due to a gas explosion and the recent 2015 collapse of a building at New York's East Village also due to a gas explosion. The characteristics of these building collapses are that the triggering event is an explosion (gas or explosive related) or the collision of the airliners (for the case of the WTC collapse) which are considered as extreme event scenarios. However, the main common pattern in most of these cases is a developing post-event fire that can be the critical catastrophic parameter for the integrity of the structure. This is the case for the WTC collapse according to the National Institute of Standards and Technology investigation and also the case for the two recent building collapses in New York City. For these cases the structural system survived the initial effect of the extreme event but finally collapsed due to the effect of the post-event developing fire. Therefore, there are cases for which a structure can be severely damaged by the triggering hazardous extreme event and then undergo a developing fire while in the damaged state.

The phenomenon of fire induced collapse of structures has been analyzed by many researchers during the last decades. Wald et al. [7] presents the results of an experimental program to investigate the global structural behavior of a 8-story steel-concrete composite frame at the Cardington laboratory. This particular experiment is considered by many as extremely important for the examination of the temperature development within the various structural elements and the atmosphere in buildings. Flint et al. [8] investigated the effects of fire on long span truss floor systems in a tall building environment, as a representative model of the type of construction used in the World-Trade Center Towers 1 and 2. The results of this paper indicated that the interaction of the highly deflected floors with the exterior or perimeter columns can lead to structural collapse. Sun et al. [9] presents collapse mechanisms of steel-framed structures with different bracing systems under a static-dynamic procedure which is developed. Fang et al. [10] investigated the influence of different structural design schemes on the collapse resistance of a fire damaged eight-story reference steel-composite car park via parametric studies. Agarwal and Varma [11] presented the assessment of the importance of gravity columns on the stability behavior of a typical mid-rise (10 story) steel building subjected to corner compartment fires, while Nigro et al. [12] proposed a probabilistic approach integrating the Monte Carlo simulation with plastic limit analysis in order to assess the probability of failure of a structure subjected to fire. Finally, Quiel and Garlock [13] evaluate some modeling parameters that affect the use of FE models to predict the behavior and capacity of a high-rise steel moment resisting frame under fire, focusing on the perimeter columns and the girders that frame into them perpendicular to the building’s exterior. The first approaches to the problem of fire considerations following an extreme event have appeared in the literature. First of all, Quiel and Marjanishvili [14] present a study of the effects of fire following an extreme event (i.e., blast or impact) that causes the failure of one column on the perimeter of a common steel building frame. The analysis involves a structure that is designed to satisfy the DoD guidelines and the goal is to evaluate the effectiveness of the design implemented for progressive collapse resistance when the structure is subjected to fire following the emergence of localized damage. The analyzed structure is a 5-story office building in which a ductile progressive collapse mode is the governing progressive collapse analysis when fire is not considered. In addition, Chen and Liew [15] presents a mixed element approach for analyzing steel frame structures subjected to a localized explosion and followed by fire. This analysis is applied to study the ultimate behaviors of a steel column and a three-story steel frame under explosion and fire. Similar concepts have been presented by Della Corte et al. [16] and Elhami Khorasani et al. [17]. In this case, the work is focused on the problem of fires following earthquakes.

The current paper is firstly based on previous work of the authors in the field of progressive collapse. This previous work is included in four papers (Pantidis and Gerasimidis [18], Gerasimidis [19], Gerasimidis et al. [20] and Gerasimidis and Sideri [21]) and presents the strong connection of the phenomenon of progressive collapse with instability failures. The main contribution of that work is the identification of buckling-induced collapse mechanisms in steel frames involving removal of columns, a new analytical method of progressive collapse analysis, and finally a new partial distributed method of progressive collapse analysis.

Secondly, the current paper is based on the previous work of the authors in the field of the mechanics of thermal response of columns in a fire. This previous work is included in the following papers: Garlock and Quiel [22,23] and Quiel and Garlock [24]. The main contribution of this work is a simplified closed-form methodology used to predict the thermal and structural response of steel perimeter columns in high-rise building frames exposed to fire. In addition, that work examines the behavior of wide-flanged steel sections with axial load and a thermal gradient through the section depth due to uneven exposure to fire.

The work presented in the current paper integrates this scholarship on progressive collapse and thermal gradients. It focuses on fire considerations on tall 2D steel frames after an extreme event. The threat-independent requirements along with the alternate load path method of analysis are applied for the removal of a column and the structure is then undergoing several fire scenarios. The steel frame used for all the analysis is a 15-story perimeter steel moment resisting frame, for which the stability failure of some of the remaining columns (after the column removal) is the governing progressive collapse mode and not the typical yielding-type catenary action collapse mode which is mostly found in shorter structures.

The objective of this paper is to evaluate the effects of different fire scenarios on the progressive collapse resistance of a tall steel frame. To this end, we performed a parametric study representing eight fire scenarios that varied the following: (1) component based versus atmospheric based temperature fields, (2) three time-temperature curves based on atmospheric temperature data, and (3) steady state vs. transient temperature analyses.

2. Post-event fire progressive collapse analysis

For the purposes of the post-event fire progressive collapse analysis, the current work analyzes a structure undergoing an extreme event by using the alternate load path method of analysis. The original contribution is the consideration of a post-event fire scenario. In that sense, as shown in Fig. 1, the structure firstly experiences an extreme event which is simulated through the alternate load path method by a key component removal (in this case, a column). Then, different fire scenarios are applied at the damaged structure around the area of the column removal, in order to evaluate the time needed for the collapse of the system and to evaluate the collapse mode of the structure. For the purposes of this work, there are four main concepts which are combined and are described in detail in the following sections (Fig. 1).

- First of all, the current work is focusing on structures which are stability governed after the column removal. For the frame used in this analysis, the collapse of the structure after a column removal involves the buckling failure of one of the columns adjacent to the removal.

- Secondly, the current work is using the experimental data from the experiment at Cardington, UK (British Steel [25]). This data is used to develop new temperature-fire curves which are describing real fire scenarios.

- The 3-jumped mass model and the closed-form solutions presented in previous papers by the authors (Garlock and
Quiel \cite{22,23} and Quiel and Garlock \cite{24} are used to make accurate predictions for the temperature of the different wide-flange components of the structure as the fire scenario is developing.

Lastly, for the steel material behavior, the Eurocode 3 material laws are used (CEN \cite{26}).

3. Benchmark steel frame studied

The response of a structure to the appearance of damage depends mainly on the extent of damage and the properties of the structure. The alternate load path method of analysis has simplified greatly the engineering process for deciding the extent of damage and the notion of a key component removal has been defined as the paradigm to model damage. The response of a structure to a key component removal is the main focus of progressive collapse research. In this field, there are different collapse modes which have been identified after a column removal in a steel frame. Previous work by the authors (Pantidis and Gerasimidis \cite{18}, Gerasimidis \cite{19}) has shown that usually for low-rise buildings the governing collapse mode involves ductile flexural failures of the horizontal members above the column removal, while for mid- to high-rise steel frames the governing mode involves the buckling failure of one of the remaining columns of the frame. For very tall structures previous work has shown that a long-wave instability collapse mode can also appear \cite{27}.

The current work is focusing on the behavior of a steel frame which is 15-stories high and it is a slightly modified version of frame #15 from Gerasimidis \cite{19} and Gerasimidis et al. \cite{28}; it is an orthogonic frame with a floor height of 3 m and a bay width of 5 m. The frame has 4 bays and the cross sections for the columns are HEB650 (for floors 1–3), HEB450 (for floors 4–6), HEB340 (for floors 7–9), HEB280 (for floors 10–12) and HEB200 (for floors 13–15). The beams’ sections are IPE550 (for floors 1, 2) and IPE500 (for floors 3–15). Fig. 2 includes the geometry of the steel frame analyzed in the paper.

Fig. 2 also illustrates the typical progressive collapse analysis without any post-event fire considerations. The column removal accounted in the present paper occurs at the ground floor, in the middle of the frame and the collapse mode for this case is governed by the buckling failure of one of the adjacent to the removal columns. It must be mentioned here that the progressive collapse analysis of the steel frame without post-event fire considerations showed that the load needed to buckle the adjacent column was higher than the sum of dead and live loads, which means that the frame can be considered to be safe for progressive collapse requirements. The selection of this frame and this specific case was aiming to observe the effect of fire on the capacity of the column adjacent to the removal. Since this component is the critical component for the structural integrity of the structure, a potential post-event fire in the area of the removal could be extremely damaging.

4. Component-based and atmospheric-based temperatures

Part of the work of this paper involves the development of new fire temperature-time curves based on real experimental data, which can be later used in the post-event progressive collapse analysis. The experimental data used, as mentioned previously, comes from full-scale fire tests at Cardington Laboratory (British Steel \cite{25}) which is
The full-scale experiments were conducted on a 8-story composite steel and concrete braced frame with three stiff cores. The program of the experiments included four major fire tests which were applied to investigate different aspects of structural behavior. These tests were:

1. 1D - restrained beam
2. 2D - plane frame
3. 3D - corner fire
4. Office fire (demonstration)

The temperature measurements from each one of the above tests included several locations on the components of the structure (which will be referred from this point on as component-based data) and also several locations in the atmosphere of the frame (which will be referred from this point on as atmosphere-based data).

For the purposes of the current paper, both component-based and atmosphere-data were used for the post-event progressive collapse analysis. The data from three of the four tests were used: the 2D-plane frame, the 3D-corner fire and the office fire tests.
4.1. Processing component-based temperature measurements

4.1.1. 2D-plane frame test

The component-based measurements were used only from the 2D-plane frame test since the post-event progressive collapse analysis involves a 2D steel frame. Fig. 3 shows the comparison between the horizontal dimensions of the 15-story steel frame used for the post-event fire analysis of this paper and the dimensions of the frame from the test at Cardington. The sketch at the bottom part of Fig. 3 is taken from British Steel [25]. Regarding the elevation dimensions, Fig. 4 shows at the left a sketch from British Steel [25] and at the right the dimensions of the 15-story steel frame of the current work. The height of the first floor of the frame at Cardington was 3.8 m while the height of the first floor of the 15-story frame is 3 m, so a scale difference of 1:1.267 is introduced in the model. It must be noted here that the sensors at the Cardington experiment were measuring temperatures on the columns at four different locations on the column (see Fig. 4, sections A–D). Following this setting, the current work has defined four zones on the column at which different temperatures are applied, as shown at the right part of Fig. 4. In the same figure, a couple of sketches have been added in order to show the number and the location of the sensors within the cross-section of the column. The sensors from tests at Cardington always provide at least one measurement on each flange and two measurements on the web of the column. For the current work, it was assumed that each flange has a separate temperature while the web has a third separate temperature and this is shown in Fig. 4 as a three lumped-mass model (T1, T2, T3). Fig. 4 includes the information about the exterior beams' temperature zones.

Fig. 4. Geometrical adjustments and temperature zones for the exterior column of the frame: a. Cardington geometry, b. 15-story frame geometry.

Fig. 5. Beams’ temperature zones.
column of the frame while similar adjustments have been made for the interior columns of the frame. For the beams of the frame, the zones are shown in Fig. 5.

The left part of Fig. 4 which comes from British Steel [25] also shows the fire protection set-up for the frame at the test. It must be noted here that the experimental set-up had a part of the column at the top which is unprotected and this is reflected on the temperature measurements from the sensors. Based on all the above, it was considered useful by the authors to use the component-based measurements from the plane-frame test at Cardington as reliable data to be used for the 15-story steel frame and the post-event fire analysis.

4.2. Processing atmosphere-based temperature measurements

The atmosphere-based measurements were used from the 2D-plane frame test, the 3D corner fire and the Office fire. These measurements were processed to develop three new fire curves of temperature versus time.

4.2.1. 2D-plane frame test

Firstly, the experimental data from the 2D-plane frame test was used. For this experiment, the location of the sensors were along the length of the beam (PB1, PB2, PB3, PB4, PB5, PB6 and PB7 in British Steel [25]). The top part of Fig. 6 includes the maximum,
minimum and average values as functions of time. It can be seen from this graph that the variance in any given time step is not very big and the red line in this graph indicates the average temperature value measured for every time step. For the purposes of this work, a new function has been developed to describe as accurately as possible the average temperature experimental values. This function is used to generate the blue line at the top graph of Fig. 7 which defines the new temperature-time curve based on this test. This graph also presents the good correlation between the new curve and the average experimental data from the test.

The new curve from the atmosphere-based temperature measurements of the 2D-plane frame test can be defined from the following expressions (for all these equations temperature is in °C and time is in seconds):

\[
T_A(t) = \begin{cases} 
20 & \text{for } t \in (0,210) \\
\frac{799}{120} t - 124.667 & \text{for } t \in (210,720) \\
\frac{1297}{33600} t + 412 & \text{for } t \in (720,9120) \\
4.086 \times 10^{-5} t^2 - 2.051 t + 39340 & \text{for } t \in (9120,34680)
\end{cases}
\]
4.2.2. 3D-corner fire test

The measurement locations for the 3D-corner fire test are in total 16 (more details can be found in British Steel [25]). The measurements for the corner fire test also include data from different heights such as 500 mm below decking for points 1–16, 1000 mm below decking for points 4, 7, 10 and 13 and 2000 mm below decking for points 4 and 10. The experimental data can be found in Fig. 6, while the new temperature-time curve can be found in Fig. 7. For this case, the new curve can be defined from the following expressions:

$$T_b(t) = \begin{cases} 
23 \degree C & \text{for } t \in (0, 660) \\
0.441t - 268.136 & \text{for } t \in (660, 1680) \\
473.156 & \text{for } t \in (1680, 2160) \\
0.237t - 39.794 & \text{for } t \in (2160, 4200) \\
962.716 & \text{for } t \in (4200, 4950) \\
(1.794 \times 10^{-4}t^3 + 2.422t^2 + 618t + 29610) & \text{for } t \in (4950, 12660) 
\end{cases}$$

The three new time-temperature curves are included in Fig. 8.

4.2.3. Office fire (demonstration) test

For the office fire test, the measurements here also include different heights within the room, while the measurements of the facade were not taken into account. Similarly to before, the experimental data can be found in Fig. 6, while the new temperature-time curve can be found in Fig. 7. For this case, the new curve can be defined from the following expressions:

$$T_c(t) = \begin{cases} 
21.6 \degree C & \text{for } t \in (0, 450) \\
0.852t - 360.207 & \text{for } t \in (450, 690) \\
3.54t - 2214.806 & \text{for } t \in (690, 810) \\
0.337t + 379.77 & \text{for } t \in (810, 1260) \\
0.165t + 596.442 & \text{for } t \in (1260, 1770) \\
-0.146t + 1145.737 & \text{for } t \in (1770, 2010) \\
0.186t + 479.009 & \text{for } t \in (2010, 2370) \\
-0.086t + 1108.877 & \text{for } t \in (2370, 2760) \\
(3.695 \times 10^{-5}t^3 + 2.803t^2 + 474.5t + 32450) & \text{for } t \in (2760, 30000) 
\end{cases}$$

The new time-temperature curves are included in Fig. 8.

5. Mechanics of wide-flange steel sections for thermal gradients due to fire

The heat transfer analysis is performed to obtain steel temperatures for the beam and column sections. The heat transfer analysis
is based on the closed-form solution developed by Quiel and Garlock [24], representing the steel cross-section using multiple lump masses (shown in Fig. 1). The procedure takes into account the thermal gradient in the cross section for non-uniform fire exposure. Based on the work of Quiel and Garlock [24], the beam-column cross-section is discretized into three coarse fibers (one for each flange and one for the web, located at their respective center of gravity) when the gradient is along the strong axis. Heat transfer between the three lumped masses is calculated as a time-series integration.

The methodology calculates heat transfer from hot gases to each lumped mass at exposed surfaces, from each lumped mass to the ambient environment at unexposed surfaces, and between lumped masses.

The above procedure is used in this study to find temperature distribution in the cross section of columns. In the case of beams, the approach is modified to include the effect of concrete slab on the top flanges.
flange temperature (which acts as a heat-sink). The model is adjusted to include an additional heat flux between the upper flange of the beam and the concrete slab. An empirical equation developed by Ghojel and Wong [29] is used to calculate the heat transfer between the top flange and the slab.

The heat transfer analysis reads fire gas temperature from curves developed in the previous sections. In addition, the procedure requires thermal properties of steel as an input. The equations from CEN 3 [26] have been used in this paper to model specific heat and thermal conductivity of steel that change with temperature. After thermal analysis is completed, the obtained temperatures in the cross section are input to the finite element program to perform structural analysis at elevated temperature. Similar as before, EC3 models are used to define mechanical properties of steel at elevated temperatures, those are yield strength and modulus of elasticity.

For the benchmark steel frame studied in this paper, the temperatures at all the beams' and columns' webs and flanges based on the three new fire temperature-time curves are shown in Fig. 9.

### 6. Application of post-event fire scenarios for the benchmark steel frame

#### 6.1. Description of the computational model

The mechanical model is simulated with the finite element software ABAQUS [30] using the beam elements B32OS for all the beams and columns of the frame apart from the ones which are undergoing temperature change. For these last ones, shell S4R elements are used in order to more accurately analyze their behavior and identify potential local instability failures. The method of analysis for the frame includes all the necessary computational aspects for the correct identification of stability failures such as in-section integration for the beam B32OS elements and all material and geometric non-linearities.

#### 6.2. Transient versus steady state analyses

Based on the data and the new fire temperature-time curves, 8 different fire scenario analyses were defined and are presented in Table 1. The table includes the post-event fire scenarios, the fire area, the type of analysis, the new temperature-time curve, the collapse time, and the mode of collapse. The post-event fire scenarios are described in Fig. 10.

**Table 1: Post-event Fire Scenarios**

<table>
<thead>
<tr>
<th>Post-event Fire Scenario #</th>
<th>Fire Area</th>
<th>Type of Analysis</th>
<th>Temperature Data</th>
<th>Collapse</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td></td>
<td>Steady-State</td>
<td>Component-Based</td>
<td>NA</td>
</tr>
<tr>
<td>2</td>
<td></td>
<td>Steady-State</td>
<td>Atmosphere-Based</td>
<td>A</td>
</tr>
<tr>
<td>3</td>
<td></td>
<td>Steady-State</td>
<td>Atmosphere-Based</td>
<td>B</td>
</tr>
<tr>
<td>4</td>
<td></td>
<td>Steady-State</td>
<td>Atmosphere-Based</td>
<td>C</td>
</tr>
<tr>
<td>5</td>
<td></td>
<td>Transient</td>
<td>Component-Based</td>
<td>NA</td>
</tr>
<tr>
<td>6</td>
<td></td>
<td>Transient</td>
<td>Atmosphere-Based</td>
<td>A</td>
</tr>
<tr>
<td>7</td>
<td></td>
<td>Transient</td>
<td>Atmosphere-Based</td>
<td>B</td>
</tr>
<tr>
<td>8</td>
<td></td>
<td>Transient</td>
<td>Atmosphere-Based</td>
<td>C</td>
</tr>
</tbody>
</table>
These analyses are categorized in steady-state and transient analysis and use either component-based or atmosphere-based temperatures. For the steady-state analysis several different time increments are picked at which the temperatures are either taken directly from the measurements (when component-based measurements are used) or calculated (when atmosphere-based measurements are used), the temperatures are applied in the model and then a separate push-down analysis (collapse analysis with incrementally increased loading) is performed until collapse is reached. This way, a separate collapse load is calculated for each of the time increments selected for analysis. When the load required to collapse the structure is lower than the dead plus live load, it is considered that the frame will collapse at that time.

The transient analysis is performed by applying a specific load on the structure (in this case the dead and live load is applied) and then the temperatures are increased in the members at different times until collapse is reached. With the transient method of analysis, a final collapse time is produced for the structure. It must be mentioned here, that the evolution of time is equal for the three different fire curves A, B and C.

An overview of the concept of the post-event method of analysis is presented in Fig. 11. Following the extreme event (the removal of a column), the data from the experiments in Cardington is used in two ways. Firstly, the component-based data is used for fire scenarios 1 and 5. The computational analysis leads finally to the calculation of the collapse capacity of the frame in terms of time, i.e. collapse time. Secondly, the atmosphere-based data are used for the development of three new fire temperature-time curves which are used to calculate the temperatures of the components in time through the analytical closed-form expressions. Then this analysis leads eventually to the collapse mode and time.

6.3. Results

The results from all the analysis are presented in Figs. 12–16 and Fig. 10. Fig. 12 shows the results of the analysis from fire scenario 1. The vertical axis of the graph represents the capacity of the frame while the horizontal axis is time. The capacity of the frame is presented in kPa, in terms of equivalent uniform loading on the floors of the building. According to the design (Gerasimidis [19] and Gerasimidis et al. [28]) the dead load is 4 kPa and the live load is 2 kPa, hence \( D + L = 6 \) kPa. There are 14 points shown in the graph which demonstrate 14 separate push-down analysis with different temperatures in the members.

As mentioned previously, the response of the specific steel frame to column removal is governed by the inelastic buckling failure of one of the columns adjacent to the removal. This failure is also found in the beginning of the fire scenario, essentially until \( t = 30 \) min (see collapse mode A in Fig. 12). However, as time passes and the temperatures begin to rise, after \( t = 40 \) min, the collapse mode changes and is governed by the flange buckling of the cross section at the top of the column adjacent to the removal (see collapse mode B in Fig. 12). This finding is somewhat expected since fire analysis 1 is component-based and the temperatures at the top of the columns are very high. The reason for these high temperatures is that the Cardington experiment had a specific setup of an unprotected column part at top of the column. It must be mentioned here that this was the setting at the Cardington experiment and it is not adopted by the authors as good design practice.

As shown in Fig. 12, the capacity of the frame reaches the dead and live loads at \( t_1 = 86 \) min and this time is considered as the expected time until collapse.
Fig. 12. Results from fire scenario 1.

Fig. 13. Results from fire scenario 2.
Fig. 13 includes the results from the analysis of fire scenario 2. As shown in Fig. 10, the difference between scenario 2 and scenario 1 is that scenario 2 is atmosphere based temperature and the columns which undergo high temperatures are only the ones around the removal. In addition, there is no fire protected zone in the columns as it is assumed that the fire protection is removed as a consequence of a possible blast event. This is an advantage of the atmosphere-based new temperature-time curves, as the level of fire protection on removal.
the members can be arbitrarily selected. For the component-based temperatures this is not an option since the measurements already include the effect of fire protection.

In this case, the collapse mode A is developing in the beginning of the fire scenario as expected, but very soon (after $t = 20$ min) the collapse mode changes slightly as both of the adjacent columns buckle simultaneously (collapse mode C). However, at $t = 60$ min the collapse mode changes significantly (see collapse mode D in Fig. 13) as the failure mechanism involves now the buckling of the column flange which is facing the fire. The time needed for collapse is in this case $t_2 = 110$ min.

All the steady-state fire scenarios are included in Fig. 14. This figure includes fire scenario 1 which is the only component-based fire scenario and also fire scenarios 2, 3 and 4 which are all atmosphere-based. In addition, fire scenario 1 applies fire to all the ground floor columns and beams while fire scenarios 2, 3 and 4 apply fire to the members just around the column removal. Fire scenario 2, which has the slowest growing fire (see curve A in Fig. 8) survives the longest (110 min). Fire scenario 4, which has the fastest growing fire (see curve C in Fig. 8) survives the shortest (27 min). In all fire scenarios, collapse occurs during the growth phase of fire (see Fig. 8). These results show that the fire time-temperature curve plays an important role in collapse time.

Fig. 14 also demonstrates that for $30 < t < 50$ for fire scenario 2, for $30 < t < 50$ for fire scenario 3 and for $15 < t < 20$ for fire scenario 4, it seems that the capacity of the frame is slightly increasing compared to the initial value for $t = 0$. The temperatures of the columns at these time periods are generally less than 400 °C, so the respective material law for $T < 400$ °C has been used from CEN [26].

For temperatures higher than 20 °C (and lower than 400 °C) and for stresses below the yielding stress, this material law associates plastic strains with low stress values. This relationship is stronger as the temperatures are higher. For example, for $T = 100$ °C the plastic strains begin at stresses close to the yield stress (235 MPa), but for $T = 400$ °C plastic strains start appearing when the stresses are around 100 MPa. This effect leads to a redistribution of forces to the remaining columns and thus slightly increasing the capacity of the frame.

A summary of results is also presented in Fig. 10. As a whole, the results indicate the following:

**Steady state versus transient.** Fig. 10 shows that the transient analyses have slightly shorter failure times than steady-state, but it is essentially insignificant. The failure modes are the same for both types of analyses. It should be mentioned that the transient analyses are more accurate as it also includes the accumulation of damage and deformations as the fire event is developing, while the steady state analyses always start from the pristine condition. Fig. 15 shows the column axial force and Fig. 16 the column vertical displacement (of the node above the column removal) for the transient analyses (Fire Scenarios 5, 6, 7, and 8). It is seen that the maximum column axial force, the maximum displacement, and the failure times are essentially the same regardless if the analyses are steady-state or transient.

**Component-based versus atmospheric-based.** A comparison between these two analyses can be done by comparing Fire Scenarios 1 and 2 (for steady-state) as well as 5 and 6 (for
Fig. 17. Typical resilience chart accounting for one hazard.

transient). Fig. 10, and a comparison of Figs. 12 and 13 (discussed earlier) show that the component-based temperatures have shorter failure times and different modes of failure.

Time-temperature curves. The atmospheric-based temperature models represent three different time-temperature curves (A, B, C) as represented in Fig. 8. Fig. 10 and Figs. 15 and 16 show significantly different failure times. The collapse mode in all cases is the buckling of the flange, which faces the fire of the adjacent-to-the-removal-column. As explained previously, the slower growing fires will result in larger collapse times, since it will take longer to heat and weaken the steel.

The analysis results show that the time needed for the collapse of the frame is actually very short if a fast growing fire is taken into account through a transient analysis ($t_8 = 21$ min). It is actually shown that although the specific frame can survive the column removal event which simulates an extreme event, the post-extreme event fire will lead to collapse in a very short time which could possibly not allow any measures to be taken to suppress the fire.

7. Post-event fire consideration on resilience

The U.S. Presidential Policy Directive [31] on Critical Infrastructure Security and Resilience defines the term resilience as “the ability to prepare for and adapt to changing conditions and withstand and recover rapidly from disruptions.” A sudden shock to the community, such as a malevolent action, can cause a sudden drop in the operability of the systems in the community, as shown in Fig. 17. Resilience is related to the remaining level of operability after the event and the restoration/recovery time to return to normal operation. At times, during restoration and repairs, the community may return to a functionality level higher than that of prior to the disturbance.

In the case of cascading multi-hazard events, such as fire following blast, performance of the system is affected after the first event
(blast), yet the subsequent event (fire) causes a second shock to the system reducing the level of operability even further (shown in Fig. 17). Fig. 18 shows that the second event may lead to a total failure depending on the available remaining capacity in the structure after the first event and the level of shock introduced to the system from the second event.

The following terminology will be used further for the description of multi-hazard resilience of the frame:

1. The **pristine** case where the columns are intact and there is no fire,
2. the **progressive collapse** case where the columns are removed but no fire,
3. the **multi-hazard** case where the columns are removed and there is a fire.

### 7.1. Multi-hazard resilience for the benchmark steel frame

Based on the results in this paper, the building under study was designed to absorb the blast shock and had residual capacity to take dead and live loads following the extreme event. However, analyses show that, if subsequent fires start, the building collapses while the time to collapse depends on the fire scenario.

Fig. 19 presents the multi-hazard resilience curve for the building. The function/performance operational level for the building is defined from Eq. (4):

\[
F_P(t) = C(t) - D ,
\]

where, \(F_P(t)\) is the function/performance of the building as a function of time, \(C(t)\) is the collapse capacity of the building as a function of...
time and $D$ is the load demand of the pristine structure under the extreme event (the dead plus live load combination).

For $t$ before the appearance of the extreme event, $C(t)$ represents the reserve of the structure against collapse provided through the initial design of the structure. This is the capacity of the pristine case. For the benchmark frame a push-down collapse analysis was performed for the pristine frame and the collapse load was found to be $C(t) = 12.95$ kPa. This is the collapse load which corresponds to the pristine structure (for $t$ before the appearance of the extreme event). As mentioned previously, the Dead plus Live Load is 6 kPa. This means that the reserve of the structure to an extreme event is 6.95 kPa.

For multi-hazard resilience, the first loss of the collapse capacity reserve occurs immediately after the extreme event which is simulated in the specific study with the column removal scenario. This describes the progressive collapse case and this first capacity loss is attributed only to the column removal. As shown in Fig. 14 for the benchmark frame this leads to a collapse capacity of 8.77 kPa, leaving a remaining 2.77 kPa as a collapse reserve. The effect of post-event fire is not accounted yet, since the reduction in collapse capacity comes solely from the column removal. It must be emphasized here, that this procedure describes the progressive collapse scenario as used today by researchers and practitioners. Since there is a remaining 2.77 kPa of collapse capacity, it becomes clear that the structure is not collapsing as a direct result of the column removal.

However, the post-event fire scenario as presented in Fig. 19 (when the 3D office fire steady-state results are used) shows that the structure will collapse only 27 min after the extreme-event occurs. This result was achieved when the post-event fire was applied to the already damaged structure and describes the multi-hazard case. It must be emphasized here, that this procedure describes the progressive collapse scenario as used today by researchers and practitioners. Since there is a remaining 2.77 kPa of collapse capacity, it becomes clear that the structure is not collapsing as a direct result of the column removal.

The immediate concern is if the time to collapse is long enough for the occupants to safely evacuate the building. Meanwhile, the recovery time to full functionality of the community relates back to the resiliency of the system. The key in the above discussion, and for future considerations, is that the recovery time and the level of damage incurred should be in proportion with the intensity of the hazard or shock introduced to the system considering the effects of cascading events that follow.

8. Conclusions and discussion

This paper studies the influence of a fire developing after the appearance of an extreme event on building structures. The extreme event is simulated using the commonly applied alternate load path method and the notion of member removal. The fire scenarios were applied to the structure with the removed member and the collapse modes and collapse times were evaluated. The main goal of the paper is to investigate the potential catastrophic consequences of a post-extreme event fire scenario. The fire scenarios applied in this work are based on experimental data from the Cardington experiment and new fire temperature-time curves which have been generated using the data. There are some important conclusions drawn which are summarized as follows:

- Eight different fire scenarios were imposed on a tall steel building designed to resist progressive collapse. In all fire cases, the structure collapsed. The steady-state versus transient analyses showed no significant difference in results. The component-based and atmospheric-based temperatures showed some differences in results. The largest difference was seen in the different fire time-temperature curves, since the rate of heating a member well affects the time it takes to weaken the steel.
- For the benchmark steel frame the collapse mode has always been governed by a stability failure either by the clear short-wave buckling of a column or by the local buckling of the flanges of the columns which are facing the fire. The generalization of this conclusion however would require an extensive parametric analysis that is beyond the scope of this paper.
- The most important conclusion from all the analysis presented in this paper is that there are structures such as the one analyzed here, which are perfectly capable to withstand the extreme event scenario but have little reserve for a post-extreme event fire development. This is the main goal of the paper, to show that the common appearance of a post-event fire can be catastrophic for the structure in a very short time. The result using the real office new fire temperature-time curve showed that the collapse of the specific frame would happen 21 min after the beginning of the fire. This finding raises reasonable concern on the capability of the structure to withstand the whole sequence of an extreme event (including the
post-event fire considerations). Therefore, it is not adequate to perform an analysis only applying the notion of member removal, but it is very important to perform a post-event fire analysis and investigate whether the structure has adequate remaining capacity to avoid collapse for a time period which would allow measures to be taken.

- The analysis presented in this paper has not included the effect of traveling fire which could have an even more catastrophic influence on the structure leading to even shorter collapse times after the beginning of the fire scenario. Also, the current analysis is limited to a two-dimensional computational model which is not accounting for potential 3D effects.

Overall, the results underline the importance of a post-extreme event fire analysis. It would be un-conservative to perform separate member removal progressive collapse analysis and a separate fire analysis of the pristine structure as they would lead to different results. The new sequential method presented in this paper starting from the member removal and then accounting for a fire scenario on the damaged structure leads to critical collapse modes and collapse times which are necessary to be accounted for in an extreme event analysis.

References


