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Structural response of a steel-frame building to horizontal and vertical travelling fires in multiple floors

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ABSTRACT

During previous fire events such as the World Trade Centre Towers (WTC) 1, 2 & 7 in New York (2001), the Windsor Tower in Madrid (2005), and the Plasco building in Iran (2017), flames were observed to travel horizontally across the floor plate and vertically to different floors. Such fires are not considered as part of the traditional prescriptive structural design for fire. Recently, the Travelling Fires Methodology (TFM) has been developed to account for such horizontally travelling nature of fires. A dozen of studies have investigated the structural response of steel, concrete, and composite structures to a single-floor travelling fire. 5 out of 6 of the vertically travelling fire studies have been limited to the structures with a long span composite truss system as in the WTC Towers. The aim of this work is to investigate the response of a substantially different structural system, i.e. a generic multi-storey steel frame, subjected to travelling fires in multiple floors, and varying the number of fire floors, including horizontal and vertical fire spread. A two-dimensional 10-storey 5-bay steel frame is modelled in the finite element software LS-DYNA. The number of multiple fire floors is varied between 1 and 10, and for each of these scenarios, 5 different fire types are investigated. They include four travelling fire scenarios and the standard fire. In total, 51 fire simulations are considered. The development of deflections, axial forces, bending moments and frame utilization are analysed. Results show that the largest stresses develop in the fire floors adjacent to cool floors, and their behaviour is independent of the number of fire floors. Results indicate that both the fire type and the number of fire floors have a significant effect on the failure time (i.e. exceeded element load carrying capacity) and the type of collapse mechanism. In the cases with a low number of fire floors (1–3) failure is dominated by the loss of material strength, while in the cases with larger number of fire floors (5–10) failure is dominated by thermal expansion. Collapse is mainly initiated by the pull-in of external columns (1–3-floor fires; 1–9-floor fires for 2.5% TFM) or swaying of the frame to the side of fire origin (5–10-floor fires). This study has assessed a different structural form compared to previous literature under an extensive range of multiple floor travelling fire scenarios. We find that although vertically travelling fires result in larger beam axial forces and initial deflections, simultaneous travelling fires result in shorter failure times and represent a more onerous scenario for the steel frame investigated.

1. Introduction

The understanding of the fundamental mechanics of a whole building behaviour in fire has significantly increased in the last decades, especially following the Broadgate fire in London in 1990 [1,2], which took place in a 14-storey steel framed building under construction. Even though the majority of the steelwork was unprotected and active fire protection methods were not functional, the building showed robust behaviour and did not collapse. Following this accident, full-scale tests of various multi-storey buildings were carried out in Cardington between 1994 and 1999 [3]. The Broadgate fire and Cardington tests showed that steel framed buildings as a whole performed better in fire than indicated by the prescriptive design of individual members. Therefore, prescriptive design approaches were believed to be conservative [2].

However, the prescriptive design was challenged and concerns were raised after the collapse of the World Trade Centre Towers 1, 2 & 7 in New York (2001) [4] and Windsor Tower fire in Madrid (2005). Firstly, the collapse of the buildings during these accidents showed that for buildings with non-conventional structural layout (unlike in the Broadgate fire and Cardington tests) the prescriptive guidance assuming single elements can be non-conservative [5]. Secondly, during these

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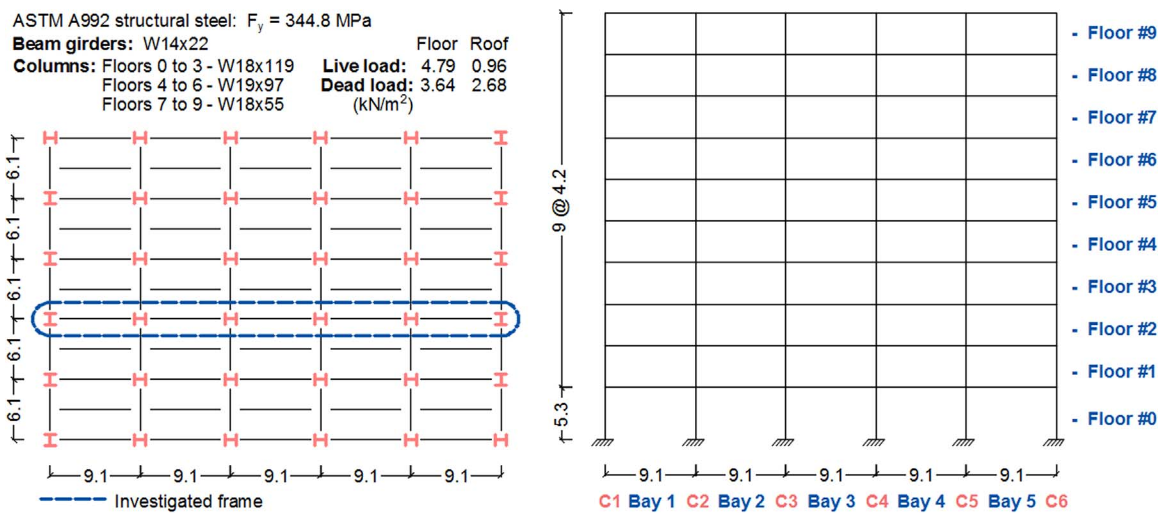

Fig. 1. Plan layout, elevation and structural member details of the investigated frame [18]. Frame dimension units are in meters.

events, fires were observed to travel horizontally across the floor plate and vertically between different floors. Such fires were not considered in the traditional prescriptive design at the time. Design codes and, thus, most of the understanding of the structural behaviour in fire were based on the assumption of uniform fires in a compartment. Recent work [5,6] has shown that, while the uniform fire assumption may be suitable for small enclosures, the large, open-plan compartments typical of modern architecture, do not burn simultaneously throughout the whole enclosure. Instead, these fires, as observed in the accidents, tend to burn over a limited area and move across floor plates as flames spread with time. They are referred to as travelling fires. To account for this travelling nature, the Travelling Fires Methodology (TFM) was developed by Stern-Gottfried et al. [6,7]. Recently, the TFM has been improved to account for more realistic fire dynamics and range of fire sizes and is referred to as iTFM [8]. Unlike traditional design methods, this methodology accounts for non-uniform temperature distributions and long-fire durations observed in the aforementioned travelling fire incidents. The methodology has been applied to investigate the thermal and structural response of steel [7,9,10], concrete [11,12] and composite structures [13,14]. In most of these studies it was concluded that the consideration of more realistic fire exposures such as travelling fires is important for the structural response and that a uniform fire assumption is not the most conservative. However, most of this work has been limited to single-storey travelling fires.

Following the 9/11 events, a lot of research has been carried out on the structural response of structural arrangements similar to WTC Towers (long-span composite truss system) subjected to multiple floor fires [2,5,15–17]. Usmani et al. [2] and Flint et al. [5] carried out computational analysis on the collapse mechanisms of the WTC Towers. The number of simultaneously heated floors and the maximum fire temperature were varied. A generalised exponential curve was used to represent the fire. Collapse was found to primarily be a result of geometric changes (i.e. inward pull-in of the external columns) and occurred at temperatures as low as 400 °C. Based on the latter work Lange et al. [16] identified two main collapse mechanisms (strong floor and weak floor) and proposed a design methodology. These collapse mechanisms were further examined by Kotsovinos and Usmani [17]. The authors performed parametric studies and established the criteria on the occurrence of strong and weak floor collapse mechanisms.

Röben et al. [15] carried out computational analysis on the steel-concrete composite structure exposed to vertically travelling fires with inter-floor time delay. The fires on each floor were represented by exponential curves adopted from the aforementioned studies. The results indicated cyclic deflection patterns of columns which were not observed previously for simultaneous multi-floor fires. The authors concluded that both simultaneous and vertically travelling fires result in different structural responses and either of them can be more onerous. One of the first studies which considered multiple floor horizontally and vertically travelling fires was conducted by Kotsovinos [14]. Fire type (i.e. uniform and travelling), size and inter-floor time delay were varied. To represent the horizontally travelling fire, the TFM [7] was used. In this study, uniform fires were found to result in higher stresses in the floor in comparison to travelling fires. Similarly to the study by Röben et al. [15], cyclic displacement patterns were observed for the cases with vertically travelling fire. In addition, results showed that small inter-floor time delay (300 s) did not have a significant effect on the structural performance.

In all of the previously identified studies significant and extensive work has been carried out to understand the structural response of high-rise structures subjected to simultaneous, horizontally and vertically travelling fire scenarios. However, most of this work on multiple floor fires is limited to structures with a long span composite truss system like in the WTC Towers. Furthermore, the focus of most of the work in [2,5,15,16] was on the collapse of the WTC, and thus the authors did not draw any generic conclusions on the effect on the structural response of the number of storeys subjected to fire. In these studies collapse was mainly associated with the stiffness of the structural members. The effect of the number of fire floors subjected to fire was only considered in the work by Kotsovinos and Usmani [17].

The aim of this work is to investigate the response of a substantially different structural system, i.e. generic multi-storey steel frame, subjected to multiple floor travelling fires and varying the number of simultaneously heated fire floors. Additionally, this work investigates how the structural response of the frame changes with inter-floor time delay, upward and downward fire spread, and opposing fire spread on different floors.

2. Computational model

2.1. The structure

The multi-storey steel frame considered in this analysis is based on the moment resistant frame published by NIST [18]. It is a 10-storey 5-bay frame representative of a generic office building with a floor layout of 45.5 m×30.5 m. It is designed according to the ASCE 7-02 standard. The plan layout and elevation of the building are shown in Fig. 1. In this study the structural fire response of a 2D internal frame with the longest beam span of 9.1 m is investigated. This frame is chosen because it is likely to be more susceptible to instabilities compared to the shorter beams (6.1 m), spanning in the perpendicular direction. All columns in the frame are 4.2 m in height except for the ground floor columns which are 5.3 m high.

The steel beams are designed to support a lightweight concrete floor slab, and composite action is achieved through shear studs. This study utilizes the design dead and live loads and no attempt was made to apply reduction factors to the loads. Design loads on the floor beams are 3.64 kN/m^2 (dead) and 4.79 kN/m^2 (live). For the roof, design loads are 2.68 kN/m^2 (dead) and 0.96 kN/m^2 (live) [18]. The beam sections are W14×22 on all floors. The column sections on floors #0 to #3, floors #4 to #6, and floors #7 to #9 are W18×119, W19×97, and W18×55, respectively. ASTM A992 structural steel with the yield strength (F_y) of 344.8 MPa is considered for all beams and columns. In this paper different bays and columns are referred to as Bay 1 to Bay 5 corresponding to different beam spans and column 1 (C1) to column 6 (C6), respectively, from the left side to the right side of the frame. Different floors of the building are referred to as Floor #0 to Floor #9 going up from the ground floor to the top floor of the frame (see Fig. 1).

Due to the 2D representation of the building, the composite action between the beams and concrete floor slab are not taken into account. However, the effect of cooling due to the presence of the concrete slab on the steel beams is considered in the heat transfer analysis. A 2D analysis has been chosen for the reasons of simplicity, computational time and to allow comparison of many different fire exposures (51) and due to the fact that iTFM defines fires spreading along a linear path. A uniform thermal profile in the side perpendicular to the direction of fire travel is assumed. In the comparative studies of 2D and 3D models [13,14,19,20] it was found that a 2D model using beam elements generally gives a good representation of the structural response to fire when compared to the 3D model using beam and shell elements. In a study on composite structures [13] the effects of the concrete slab were found to be more significant during cooling leading to reduced axial forces and higher residual moments as the slab cools down more slowly. Though, the results showed a close agreement between the 2D and 3D models in the overall trends in behaviour. In a study on tall composite buildings with a concrete core and perimeter long-span steel beams [20] it was also observed that the 2D model is less redundant and, as a result, loses its strength faster. In general, the 2D models in [13,14,19,20] were found to be conservative and show a good qualitative agreement with 3D models. Therefore, the 2D representation is considered to be acceptable for this study as the main aims are to analyse the general trends and to compare the outcomes of the different fire scenarios considered.

2.2. Fire scenarios

The structural response of the frame subjected to TFM [8] and the standard fire (ISO) [21] is investigated. The number of floors subjected to fire simultaneously is varied between 1 and 10 (i.e. a whole frame). In addition, two-floor vertically travelling fire scenarios are considered to analyse the effect of time delay due to upward and downward fire spread between floors and opposing horizontal fire spread on multiple floors. Vertical fire spread delay between floors is assumed to be 10 min and 25 min. Similar values were used in the study by Röben et al. [15] and, as identified in the latter study, are within the estimated vertical fire spread rates of 6 and 30 min based on the Windsor Tower fire in Madrid (2005). In total, 51 different fire scenarios are investigated as shown in Table 1.

To represent a travelling fire exposure, iTFM [8] is used. An illustration of a travelling fire is shown in Fig. 2. This methodology considers a family of fires represented by the percentage of floor area engulfed in flames at any time. It is assumed that the fuel is uniformly distributed over the floor and, once alight, burns at a constant rate. Thus, fire size is governed by the fire spread rate. Each floor of the frame in this study is subjected to four TFM scenarios: fire sizes of 2.5%, 10%, 25%, and 48% of the floor area. TFM sizes of 2.5% and 48% correspond approximately to the limits of likely realistic fire spread rates in compartments as identified in [8], i.e. spread rates of 1 mm/s and 19.2 mm/s, respectively. TFM sizes of 10% and 25% have been found to be the worst case scenarios in previous studies [6,11]. In this frame, travelling fires are assumed to travel from Bay 1 to Bay 5 (see Fig. 1). The fuel load density and heat release rates are assumed to be 570 MJ/m^2 (80th percentile design value for offices based on the Eurocode) and 500 kW/m^2 (typical value for densely furbished places) [7], respectively. The correlation to represent the standard fire (referred to as the ISO standard fire in this paper) is taken from the Eurocode [21]. The standard fire has its origins in the early 20th century and forms the basis of fire resistance rating and standards worldwide. Illustration of the gas temperatures for all fire scenarios at the mid-span of Bay 2 is shown in Fig. 3.

2.3. Heat transfer

Beams and columns are designed for 60 min and 120 min standard fire resistance respectively with a limiting temperature of 550 °C using Eq. (1) [6,21]. At 550 °C steel maintains only 60% of its ambient temperature strength because of the thermal degradation of its mechanical properties and, thus, it is commonly accepted as the critical temperature for steel in traditional design [23].

$$\Delta T_s = \frac{H_p}{A}\frac{k_i}{d_i\rho_s c_s}\frac{\rho_s c_s}{[\rho_s c_s + (H_p/A)d_i\rho_i c_i/2]}(T_g - T_s)\Delta t \tag{1}$$

where H_p is the heated perimeter of the beam or column (m), A is the cross-section area of the beam or column (m^2), T_s is the steel temperature (K), T_g is the gas temperature (K), d_i is the thickness of the insulation (m), ρ_s is the density of steel (kg/m^3), c_s is the temperature dependent specific heat of steel taken from [24] (J/kg K), and Δt is the time step (s). In this study, heat transfer and structural analyses are not coupled. First, heat transfer calculations of

Table 1
Details of the investigated fire scenarios.

Fire scenario	Fire floors (#)	Fire type	Time delay between floors
Horizontal (simultaneous)	5	2.5% TFM, 10% TFM, 25% TFM, 45% TFM and ISO	0 min
	4 & 5		
	4, 5, & 6		
	3, 4, 5, 6, & 7		
	2, 3, 4, 5, 6, 7, & 8		
	1, 2, 3, 4, 5, 6, 7, 8, & 9		
	0, 1, 2, 3, 4, 5, 6, 7, 8, & 9		
Opposing (simultaneous)	4 (→) & 5 (←)[a]	10% TFM and 25% TFM	
	4 (←) & 5 (→)[a]		
Vertically travelling	From 4 to 5 (↑)[a]	10% TFM, 25% TFM, and ISO	10 & 25 min
	From 5 to 4 (↓)[a]		

[a] Arrows indicate horizontal and vertical fire spread directions on and between different floors.

member heating are done in MATLAB and then the resultant transient member temperatures are applied as an input of thermal load in the structural model of LS-DYNA (described in the following Section 'LS-DYNA model'). Heat transfer to the structural members using Eq. (1) is carried out assuming lumped capacitance for separate parts of the cross-section (i.e. web and flanges). This is a common assumption when assessing the thermal response of the protected steelwork as suggested by EN1993-1-2 [24]. Biot number for different sections and parts is in the range between 0.0019 and 0.0154 (< 0.1) indicating that the thermal gradient across the section would be very small [25]. Steel insulation properties are taken as for high density perlite (thermal conductivity k_i =0.12 W/m K, density ρ_i =550 kg/m^3, and specific heat c_i =1200 J/kg K) [22].

For beams, the effect of the slab acting as a heat sink is taken into account. That is, the top surface of the upper flange, which is in contact with the slab, is excluded in the calculation of the heated perimeter. The convective heat transfer coefficient at the free surface, density of steel and radiative emissivity at the free surface are assumed to be 35 W/m^2 K, 7850 kg/m^3 and 0.7, respectively [22]. The time step that satisfies the stability criteria for the heat transfer calculations is 10 s. Vertical temperature distributions in the compartment are not currently taken into account in iTFM. As a result, columns are assumed to be exposed to the same fire conditions as that at the same location in the ceiling. Temperatures along the column height are assumed to be uniform to represent the worst case scenario (i.e. assumed to be conservative). An illustration of temperature development in the beam sections at the mid-span of Bay 2 is shown in Fig. 3. It shows that at the indicated location, beams subjected to small travelling fires (e.g. 5% TF) reach higher temperatures in comparison to large travelling fires (e.g. 48% TFM) by up to approx. 160 °C. Even though large travelling fires may have higher near-field temperatures than small travelling fires, their durations and the time it takes to reach those temperatures are significantly different. 48% TF travels along the compartment in 1 h, while 5% TF takes up to 13 h.

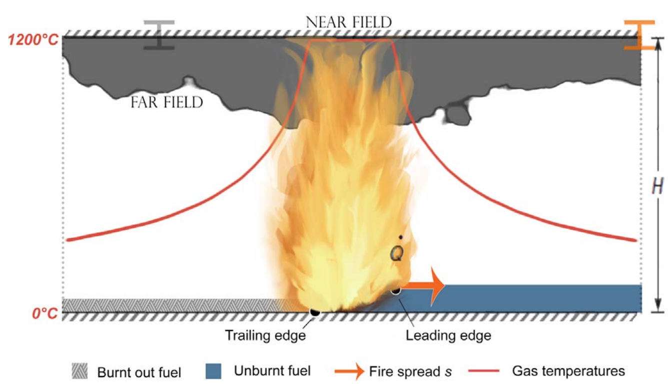

Fig. 2. Illustration of a travelling fire and distribution of gas temperatures in the near-field and far-field [8].

2.4. LS-DYNA model

The multi-storey steel frame is modelled using the general purpose finite element software LS-DYNA (Release 7.1.1) explicit solver. The software was originally developed specifically for highly nonlinear and transient dynamic analysis. LS-DYNA is capable of simulating the thermal and thermal-structural coupling problems and has an extensive element and material library. Prior to this analysis the software was validated by the authors [26] against the available benchmarking and fire test data for structural fire analysis. In the latter study, LS-DYNA has been validated against 4 different benchmark cases which include a uniformly heated beam, composite steel-concrete floor, 2D steel frame and a fire test on loaded steel framework. These benchmarks were chosen to make sure that they capture the key structural response in fire mechanisms. The latter two benchmarks, similarly to this paper, are on the response of 2D steel frames to fire and the same modelling approach is used in this study. All of the parameters for the model presented in this section were chosen based on mesh density and parameter sensitivity convergence studies.

The steel beams and columns are modelled using the Hughes-Liu [27] beam element formulation with a cross-section integration refinement factor of 5. Beams, Floor #0 columns, and Floor #1 to #9 columns are divided into 36, 22, and 16 beam elements, respectively. The corresponding beam element length is approximately 0.25 m. This was chosen as the optimal solution based on the mesh density study. Table 2 shows mesh density convergence results for the frame exposed to the ISO fire on Floor #2. The supports for the ground floor columns are assumed to be fixed, and the beams and columns are assumed to be rigidly connected. The aim of this study is the investigation of the global structural response, and, thus, no attempt was made to capture

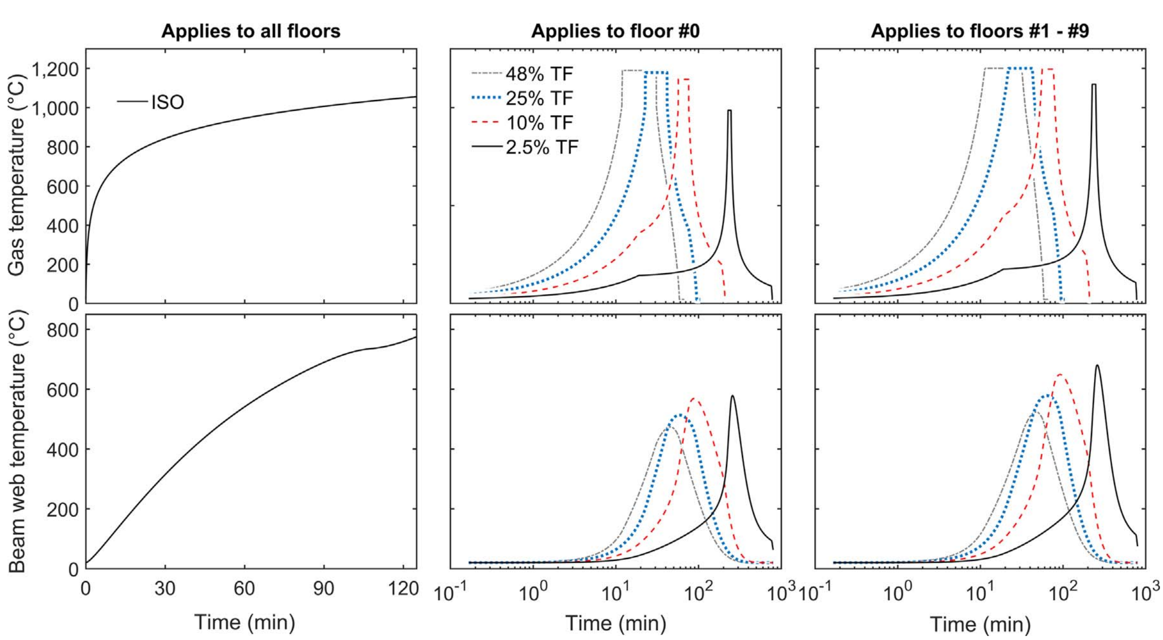

Fig. 3. Gas temperatures (top) and corresponding steel beam web temperatures (bottom) for the considered fire scenarios.

Table 2
Mesh density convergence.

Beam element length (m)	Kinetic energy/ Internal energy (%)	Average relative error (%) - Bay 1 Floor #2			CPU time (s)
		Displacement	Bending moment	Axial force	
0.0625	3×10^{-3}	–	–	–	26,506
0.125	4×10^{-4}	1.72	0.71	5.34	44,190
0.25	5×10^{-4}	5.22	1.65	8.71	5847
0.5	3×10^{-3}	10.98	2.68	19.37	1774
1	7×10^{-3}	17.99	3.76	27.87	556

localised failures in the beams, columns, or in the beam-to-column connections. We use the thermally-sensitive steel material MAT 202 formulation based on EN1993-1–2 [24] for both steel beams and columns with the default temperature-dependent material properties. Steel with initial yield stress of 345 MPa, Young's modulus of 210 GPa [28], and Poisson's ratio of 0.3 [28] is assigned to all members.

Simulations are carried out using the explicit transient dynamic solver of LS-DYNA that uses real-time units to solve the equations of motion. Due to the mainly quasi-static nature of the structural response to fire, when a dynamic analysis is carried out, the most common approach is to scale-down the time in the model [26]. However, it could lead to the introduction of significant artificial inertia forces to the system, which could impact the structural response [26]. In order to avoid artificially high dynamic oscillations, the mechanical and gravity loads are ramped in linear increment over 1 s and then kept constant for the remainder of the simulation. After 2 s, that is once the steady-state solution is attained (i.e. the model is stable), thermal loads are applied. Thermal load is scaled by a factor of 100, which was determined to be an appropriate scaling factor in order to control the inertia effects based on the sensitivity analysis carried out in [26]. This means that a standard fire (in terms of member temperatures) which would last 120 min in real life is applied 100 times faster, i.e. in 1.2 min, in the simulation time (CPU). This does not affect the heat transfer, but affects the computational time. For more details the reader is referred to [26].

3. Results and discussion

3.1. Multiple-floor fires

The development of the beam mid-span displacements, axial forces, bending moments and column lateral displacements for the 5 bays in Floor #5 for a 25% travelling fire scenario is shown in Fig. 4, where comparison is made between a single floor and 5-floor fire scenarios. The results show that under multiple floor fires the structural response is significantly different compared to a single floor fire. In the multiple-floor fire scenario initial (i.e. within the first 50 min) peak beam mid-span displacements and bending moments are lower by up to 270 mm and 63 kN m, respectively, in comparison to the single-floor fire scenario. However, column lateral displacements are larger and indicate the swaying of the frame in the direction of the fire origin until failure occurs at 53 min. In addition, the beams in Floor #5 in the multiple 5-floor fire are under tension rather than compression as in the single floor fire. The reason for that is that during the multiple floor fire scenario the beams in Floor #5, which is the central fire floor, have lower restraint to thermal expansion from the adjacent floors in comparison to a single floor fire. This is because the adjacent floors in the former case are expanding as well. Thus, thermal bowing and moment redistribution dominate the behaviour and the heated beams are in tension.

In general, if there is sufficient restraint from the surrounding structure, at the beginning of fire exposure beams are under compression due to the restrained thermal expansion until yielding or buckling occurs. Yielding takes place when compressive axial force begins to decrease followed by elasto-plastic response and a sudden increase in deflection [29]. This can be clearly seen for a single floor fire scenario. Although not shown in Fig. 4, axial force development patterns are similar in fire floors adjacent to cool floors irrespective of the number of fire floors because of the stiff surrounding structure. Thus, the highest axial forces develop within these floors. This is illustrated in Fig. 5, which shows the peak compressive or tensile axial forces which develop during the fire exposure for different fire scenarios (travelling fires and ISO fire) and different number of multiple floors on fire. Positive and negative values indicate tension and compression, respectively. The results show that, as identified previously, the highest compressive axial forces develop in the fire floors adjacent to the cool floors. As a result, due to stress redistribution the highest tensile stresses develop within the latter (i.e. cool floors). Due to thermal expansion of multiple floors and, thus, lack of restraint to thermal expansion, axial forces that develop in interior fire floors are significantly smaller. They are between 0.1 and 102 kN. That is 0.03–53% of the compressive axial forces (181–335 kN) in the fire floors adjacent to cool floors.

The variation of the peak compressive axial forces which develop in beams and the time to reach it with the number of floors exposed to fire is shown in Fig. 6. The highest axial forces develop earliest for the 25% and 48% TFM and ISO fires irrespective of the number of floors exposed to simultaneous fire. This is because under uniform (ISO) and large TFM fire scenarios thermal expansion of beams in the heated floors is larger in comparison to localised smaller (2.5% and 10%) TFM scenarios. A similar observation was made in the previous work on the analysis of the effect of travelling fires and uniform fires on the structural response of the same frame [10]. In terms of the number of fire floors, the highest axial forces develop for the single floor fire scenario. In this case, the unheated structure provides a higher axial restraint in comparison to intermediate cases where the number of cool floors is reduced. The lowest axial forces develop for the 2-floor and 3-floor fire scenarios. This is likely due to the fact that column section sizes in the fire floors adjacent to the cooler structure are smaller than in the other cases and, therefore, result in a lower level of restraint. However, the structural response in fire can be influenced by many different factors. They include but are not limited to, for example, material non-linearity, geometric non-linearity, restraint conditions, stress redistribution, thermal bowing, restrained thermal expansion, etc. Different combinations of various factors in some cases may lead to very similar results. Therefore, further studies varying the location of the fires and column sizes should be conducted to confirm this. Time to reach the peak axial force in general follows similar trends as the axial forces and is lowest for the 48% TFM.

The failure time and corresponding web temperatures of the element that failed for different fire scenarios are shown in Fig. 7. Failure is defined as the exceeded element load carrying capacity (i.e. rupture) and indicates the local collapse. The results indicate that with the increasing number of simultaneously heated fire floors failure time decreases as expected. This is primarily because of the larger loss of stiffness of the frame as the larger number of the floors is heated. For the cases where the whole frame is exposed to the TFM scenarios there is a slight increase in time in comparison to the 9-floor fire scenarios and for 2.5% TFM failure does not occur. This is likely due to the slightly reduced rigidity of the frame and consequently axial forces and bending moments as the whole frame is expanding simultaneously. Also, high axial forces only develop in ground floor beams (see Fig. 5). In the previous study by the authors, it was observed that depending on the structural metric examined, either travelling fires or uniform fires can lead to failure and be the worst case scenario [10]. Similarly, this study shows that for 1-floor to 3-floor fires ISO results in the earliest failure time, while for the larger number multiple fire floors 25% and 48% TFM indicate an earlier failure time. In addition, no failure is

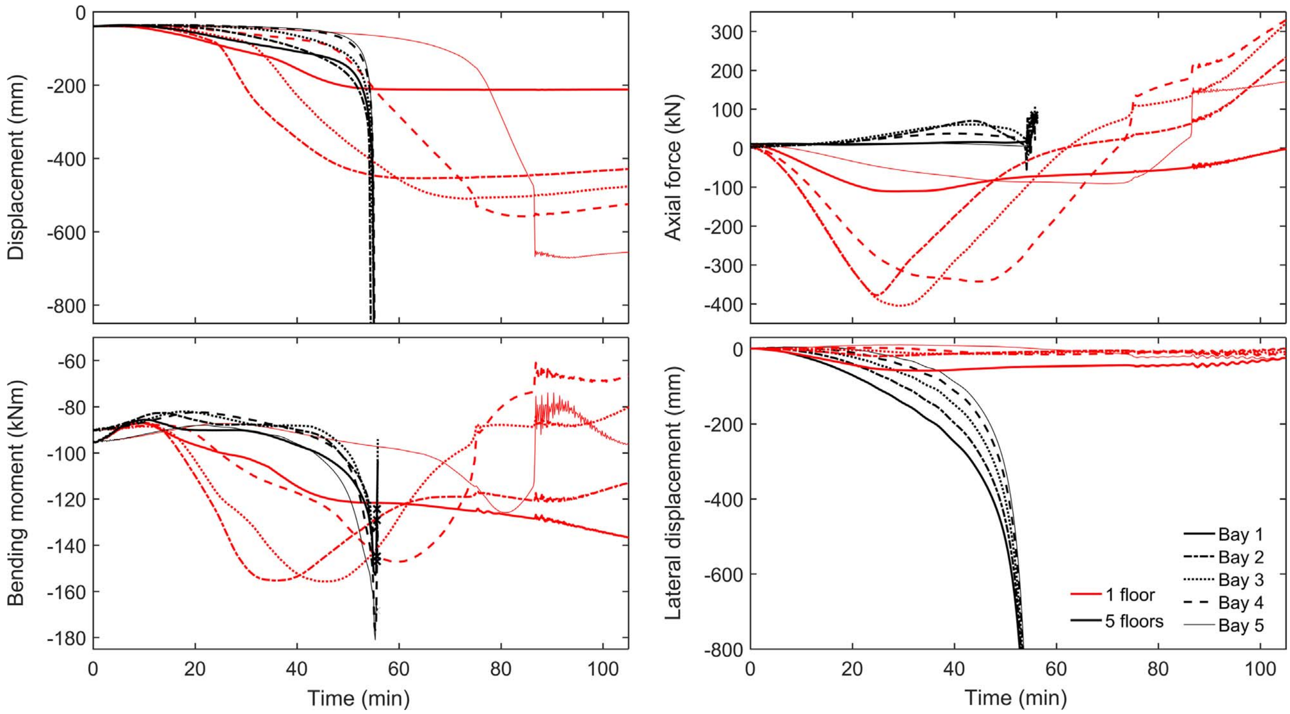

Fig. 4. Comparison of Floor #5 beam mid-span displacement, axial force, bending moment and column lateral displacement development for a frame exposed to the 25% TFM on 1 floor and 5 floors.

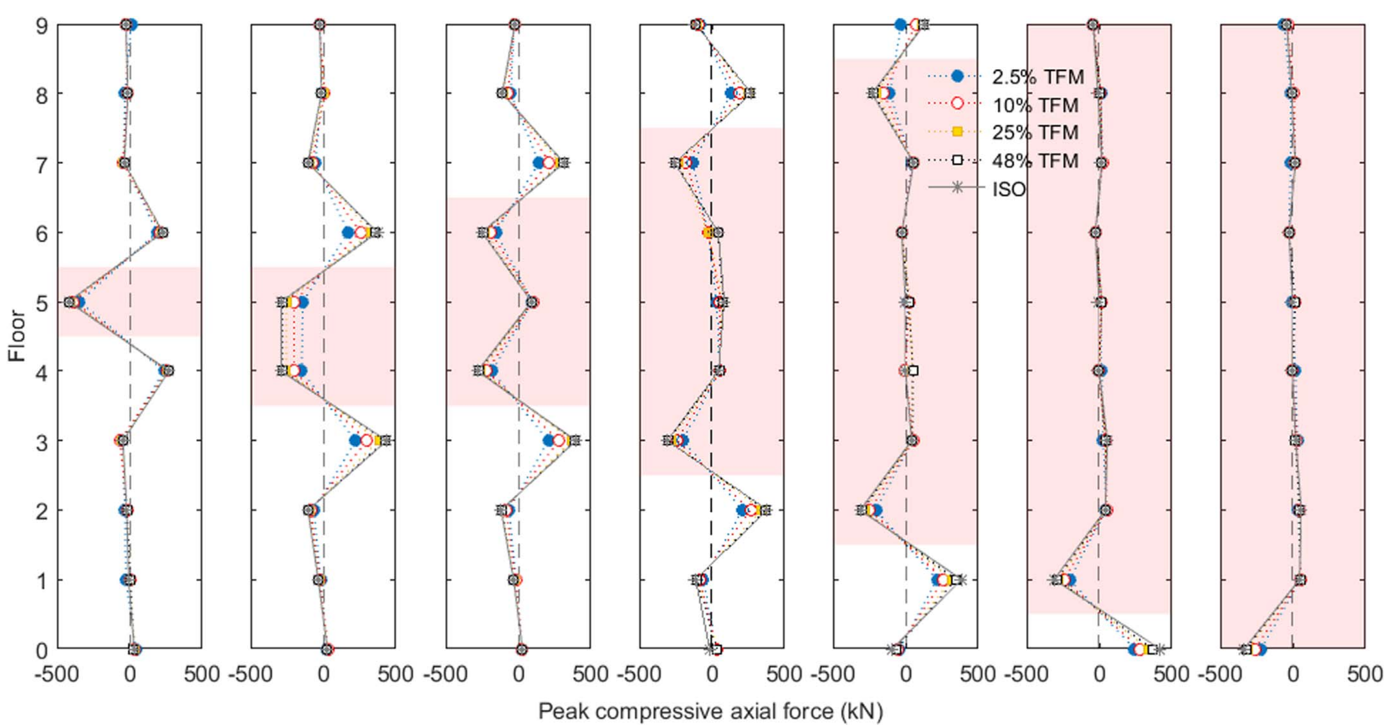

Fig. 5. Peak beam axial forces which develop during the fire exposure on different levels of the building. Shaded area represents the fire floors.

reported for the single floor fire for the latter fire scenarios. It shows that it is important to consider multiple floor fires in the structural design as the single floor fire scenario might not always capture the probable failure.

The web temperatures of the failed element indicate that for smaller and longer in duration TFM scenarios (2.5% and 10%) failure occurs when elements reach temperatures between 500 and 700 °C. These values are within the expected range and are around the temperature of 550 °C, which indicates a loss of material strength of 40%. However, for large TFM scenarios (25% and 48%) and ISO fire with the number of multiple fire floors larger than 5 failure occurs within members with temperatures as low as 125 °C. Due to a relatively uniform temperature distribution and large overall thermal expansion in these cases the frame sways towards the fire origin initiating the failure. A similar failure mechanism occurs for all fire scenarios with 5 or more floors exposed to simultaneous fires except for the 2.5% TFM. An illustration of the typical deflected shapes close to failure is shown in Fig. 8. For the cases with up to 3 heated fire floors and all 2.5% scenarios failure is a

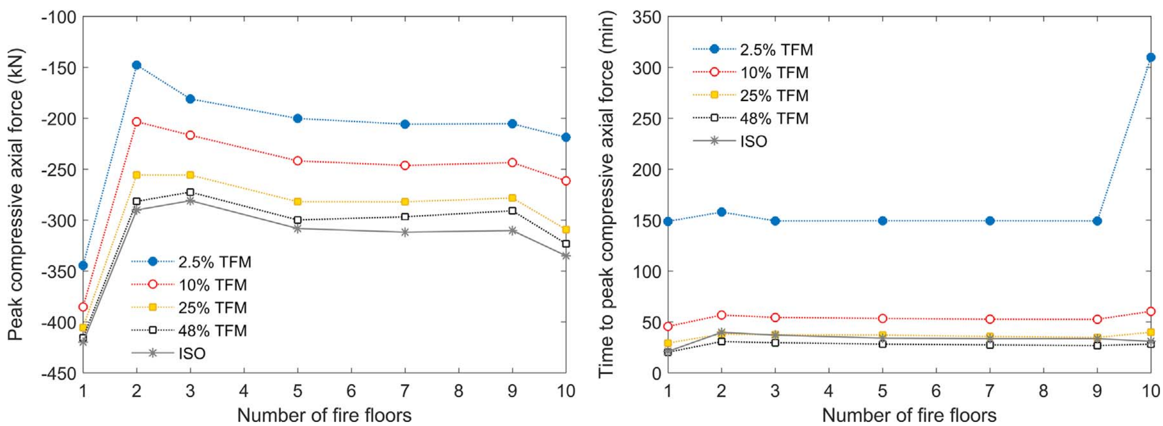

Fig. 6. Peak heated beam compressive axial force variation (left) and time to reach it (right) with fire scenario and number of floors simultaneously exposed to fire.

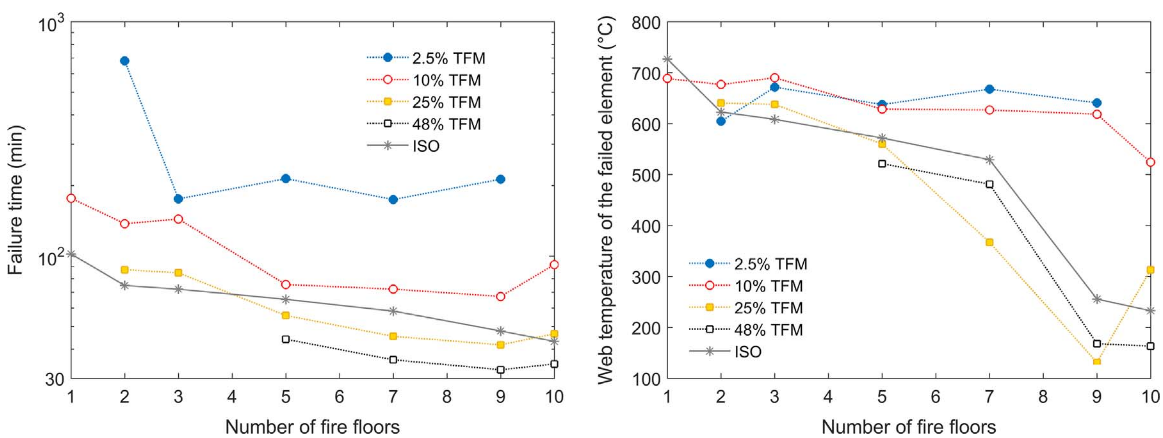

Fig. 7. Variation of the failure time (left) and corresponding web temperature of the element that failed (right) with different fire scenarios and number of s simultaneously exposed to fire.

result of pull-in of external columns at the fire origin or towards the end of the fire path. It is similar to the weak-floor and strong-floor collapse mechanisms reported in [15,16]. For the travelling fire scenarios, cooler beams subjected to the far-field heating or cooling and adjacent cool floors provide a large restraint and path for stress redistribution similarly to the concrete core in the latter studies.

The weak-floor collapse mechanism is identified as the initiation of collapse due to the buckling of the floor below the fire floors [15,16]. For the strong-floor collapse mechanism failure is a result of the plastic collapse and formation of three hinges [15,16]. For the travelling fire scenarios occurring in the multi-bay frame as in this study both mechanisms or a mix of them are observed. For the 3-floor 10% TFM and 5-floor 2.5% TFM, results in Bay 2 indicate deflected shape patterns similar to the weak-floor collapse. In the other TFM and ISO scenarios deflected patters and pull-in of the columns is similar to the strong-floor or a mixed collapse mechanism. For the multi-storey and multi-bay frame analysed in this study failure type seems to be a result of not only the relative stiffness of the member at room temperature and number of fire floors as in [17], but also the travelling fire scenario considered.

Analysis of the axial force and bending moment separately within different members may give an indication of stresses which develop within these members but not of the actual member utilization. Utilization is the factor of the load carried by the member and its load carrying capacity. Two members may have similar peak axial forces developing within them and have significantly different temperature distributions at that time. With increasing temperature, the cross-section load carrying capacity decreases and thus utilization of these members and how close to failure they are might be very different. Therefore, to account for this the results are also analysed in terms of the utilization of the members. Plastic axial load and moment (P-M) interaction curves are calculated according to [30]. Temperature dependent yield strength reduction factors are taken from the Eurocode [24]. Illustration of P-M curves at ambient temperature and during the 25% 2-floor and 9-floor travelling fire is shown in Fig. 9. P-M curves are shown for the elements that failed first, i.e. beam for 25% TFM 2-floor fire and column for 25% 9-floor fire. One curve is shown for the column, because it reaches a temperature of 130 °C only. At this temperature steel still maintains all of its strength. The utilization of the member is found based on these curves, axial forces, and bending moments in different members at every time step. Cumulative density functions (CDF) of frame utilization at failure or the end of fire exposure for all fire scenarios are shown in Fig. 10. CDF shows what percentage of the members in the frame is under the utilization equal to or lower than the specified value. The lower and upper bounds in Fig. 10 show the average and peak utilization, respectively, of different members (i.e. beams and columns). It should be noted that even at ambient temperature utilization of the frame is relatively high.

The results indicate that the frame experiences the highest average and peak utilization in the fire scenarios with 5 or more simultaneously heated fire floors. In general, frame utilization is highest in fire scenarios with the highest failure times. Considering the scenarios for which failure did not occur, results indicate the highest post-fire utilization for the 2.5% TFM 1-floor multiple fire scenario. Average member utilization is between 18% and 80%. For other scenarios (2.5% 10-floor, 25% TFM 1-floor, and 48% 1–3-floor fires) it is lower and between 17% and 67%. For all scenarios, more members are under

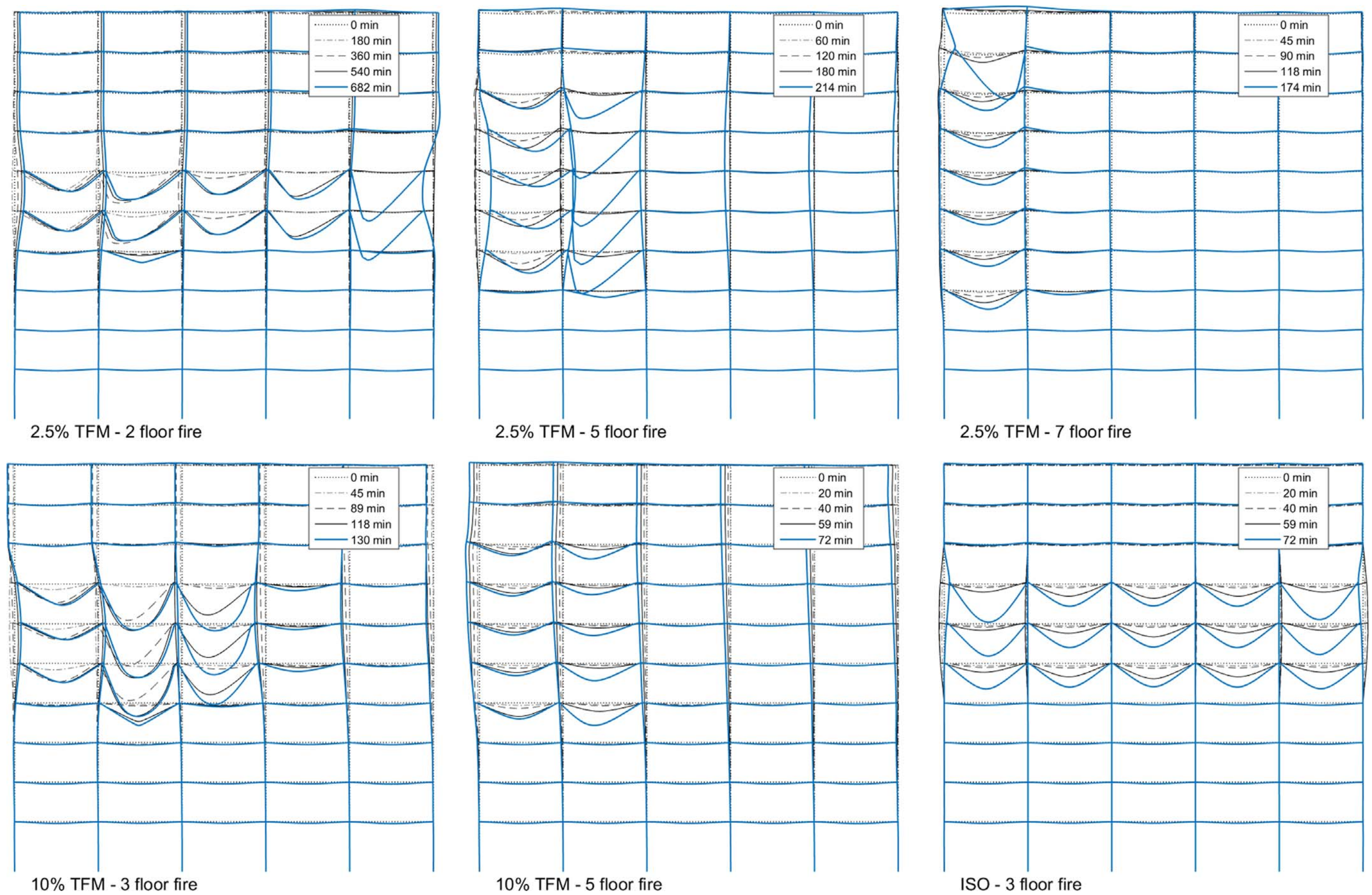

Fig. 8. Illustration of typical deflected shapes of the frame close to failure for 2.5% TFM, 10% TFM and ISO fires and varying number of fire floors.

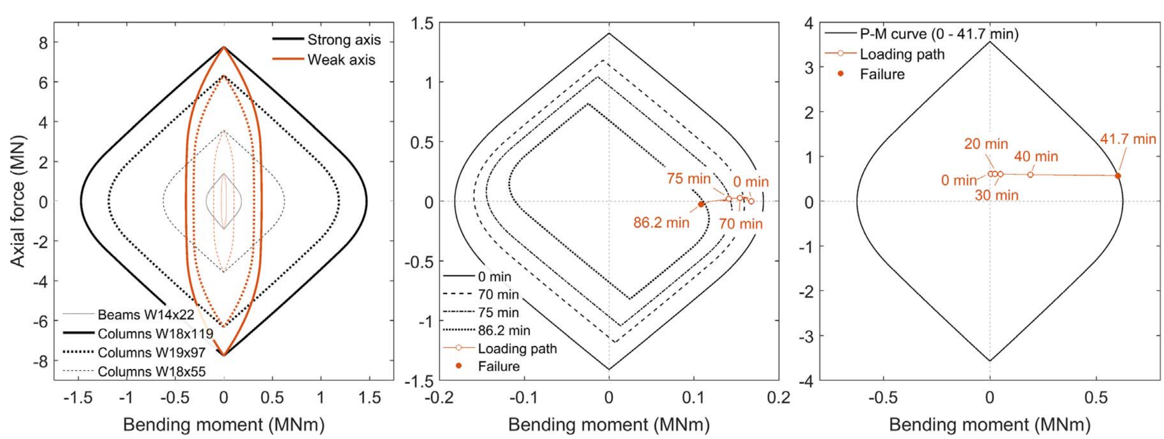

Fig. 9. Illustration of P-M interaction curves for frame elements at room temperature (left); during the 25% TFM 2-floor (middle) and 9-floor (right) fire scenarios for the elements that failed and their utilization.

higher utilization with increasing number of floors on fire. However, the maximum average utilization within the frame is highest for the single floor ISO fire scenario.

3.2. Opposing and vertically travelling fires

Comparisons of heated beam mid-span displacement, axial forces, bending moments and column lateral displacement for simultaneous, opposite and vertically travelling 2-floor 25% TFM are shown in Fig. 11. The development of beam mid-span displacement for all fire scenarios follows a similar trend. Peak displacements develop in Bay 3 and are the lowest in the bay of fire origin. The only difference is that for the vertically travelling fire scenario development in the upper fire floor is delayed. The magnitude of the deflections in both heated fire floors are almost identical. Even in the opposing fire spread scenario development of beam deflections in respect to the fire origin location is similar. That is, deflections on Floor #4 Bay 1 are the same as on the upper floor Bay 5, where the fire is spreading in the opposite direction. Though, the peak beam displacement values are slightly higher (up to 90 mm) in simultaneous and opposite fire scenarios than in the vertically travelling fire scenario. This could be due to a higher restraint to thermal expansion (beams in the upper floor remain cool for 25 min) and larger amount of thermal stresses contributing to the compressive axial force development instead. However, the initial deflections larger

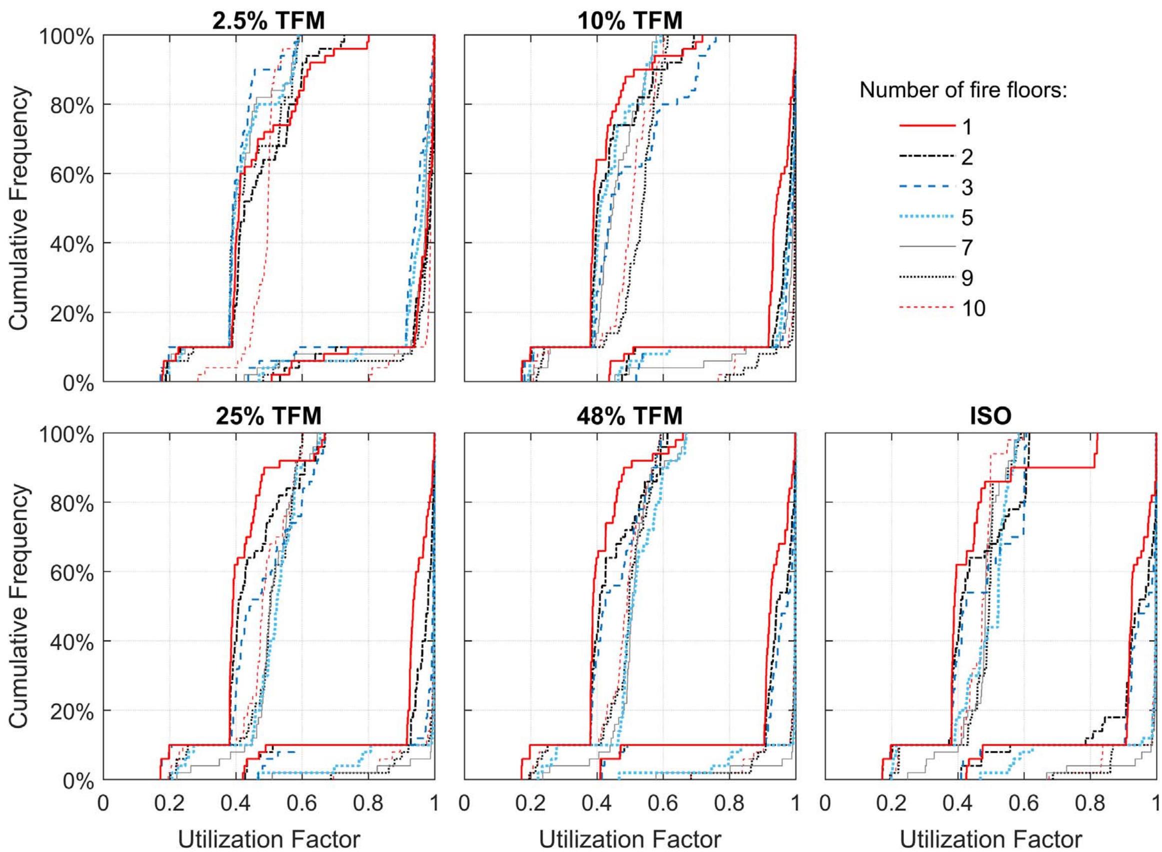

Fig. 10. Cumulative frequency function of frame utilization at failure of the end of fire exposure for all fire scenarios.

than 200 mm develop earlier in the vertically travelling fire scenario.

Unlike beam mid-span deflections, the effect of different fire spread scenarios is more significant on the development of beam axial forces, bending moments, and column lateral displacements. Due to higher axial restraint, as identified previously, the highest axial forces in heated beams develop for the vertically travelling fire scenario. They are higher by up to 120 kN and 150 kN than in simultaneous and opposite fire scenarios, respectively. Similarly, the highest bending moments develop in the vertically travelling fire scenario as well. In addition, for the simultaneous fire, all beams on the heated floors during the heating are in compression while for the opposite fire scenario beams in the bay of origin are in tension.

Column lateral displacements attain the largest values for the simultaneous fire scenario (by up to 20 mm). This is because of the higher combined thermal expansion across the frame and lower thermal restraint. In other scenarios different members either experience near-field temperatures in the opposite sides of the bay or on the same side but after a thermal delay. The largest displacements develop in the bay of the fire origin. Lateral column displacement results in increased eccentricity of the load acting on the columns and, therefore, larger bending moments. Once the heated beams go into tension, column displacement reverses and they are pulled in inwards. This leads to the initiation of failure for the simultaneous multiple-floor fire scenario. No failure is reported for the vertically travelling and opposing fire scenarios. Unlike in previous studies on vertical travelling fires [13,14], no significant oscillations in the lateral column displacement due to the vertical fire spread are observed in this study. This could be due to the different number of fire floors, different beam span length (9.1 m vs. approx. 18 m), stiffness of the beams and the surrounding structure or fire scenario used and needs further investigation to draw any definite conclusions.

The comparison of the CDF of the average and peak utilization of different floor members at 0, 30, 60 and 85 min for different fire spread scenarios is shown in Fig. 12. The results show that the level of frame utilization for different fire scenarios is very similar, particularly for the first 30 min. Even for the simultaneous fire scenario close to failure (at 85 min) the average utilization of the frame is higher by only 2.1%. Similar overall stress and displacement development trends are observed for the other investigated fire scenarios (10% TFM and ISO fire, and vertically travelling fire with a delay of 10 min). In general, similarly to the work by Röben et al. [15] the results indicate that depending on the structural metric of interest (displacements, utilization, or structural failure) either of the fire scenarios can represent the worst case scenario.

4. Conclusions

In this study, the structural response of a generic steel frame exposed to multiple-floor travelling fires and standard fire has been investigated. The results show that the highest stresses develop in the fire floors adjacent to the cool floors while in the intermediate fire floors the stresses are significantly smaller (i.e. axial forces by up to 47–99.97%). Peak compressive and tensile axial forces develop in the fire floors adjacent to the cool floors and in the cool floors, respectively. With increasing number of fire floors (from 2 to 10) and travelling fire size, the peak beam compressive axial forces rise by 41–71 kN (15–48%) and 70–134 kN (20–90%), respectively. However, the highest axial forces develop for fire scenarios with only one fire floor.

The results indicate that for the investigated frame the failure time and collapse mechanism are affected by both the fire type and the number of fire floors. For the cases with 1–3 fire floors failure is mostly dominated by the loss of material strength with temperature and occurs at temperatures between 600 and 730 °C. Failure is initiated by the pull-in of external columns. On the other hand, for the cases with 5 or more fire floors, failure is dominated by thermal expansion and geometric effects and occurs at temperatures as low as 130 °C. The

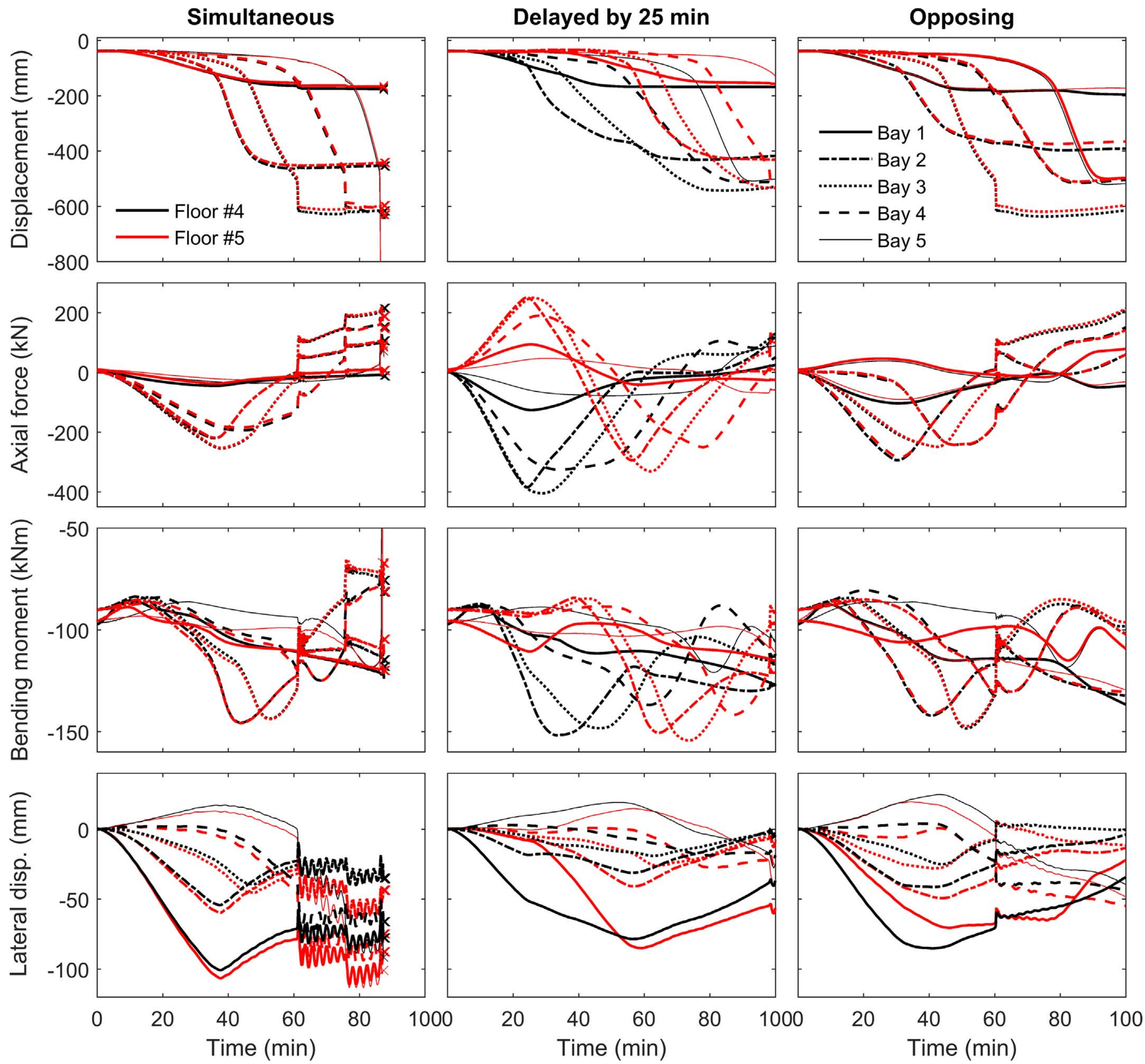

Fig. 11. Comparison of beam mid-span displacement, axial force, bending moment and column lateral displacement development for a frame exposed to the 25% TFM on Floors #4 and #5 and varying fire spread (simultaneous, vertically travelling and opposing).

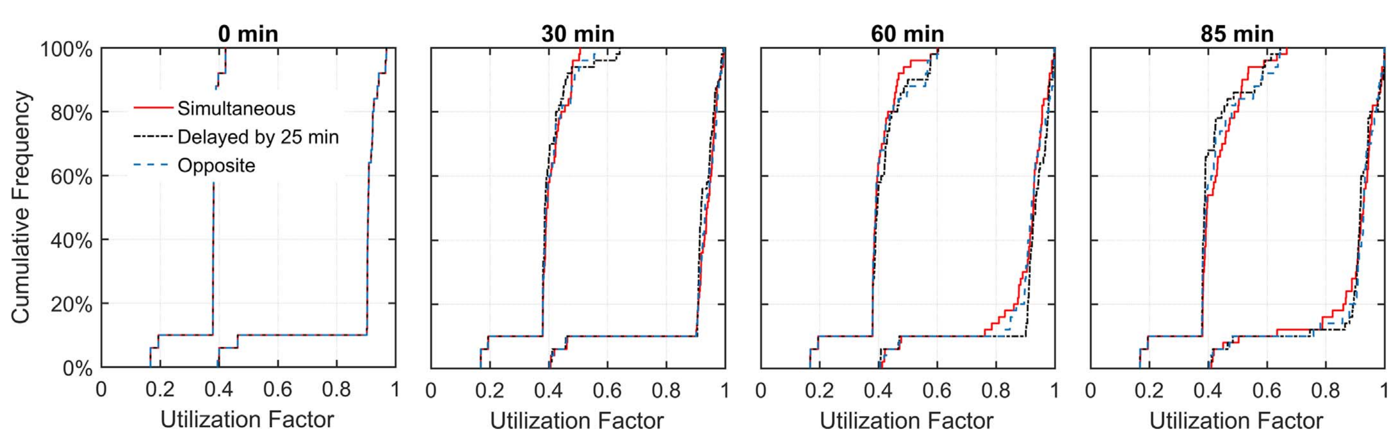

Fig. 12. Cumulative frequency function of frame utilization at 0, 30, 60, and 85 min for simultaneous, vertically travelling and opposing fire spread scenarios.

frame fails by swaying to the side of the fire origin. Failure time decreases with increasing travelling fire size. ISO fire is more onerous for the cases with 1–3 fire floors. For the cases with 5 and more fire floors, 25% and 48% travelling fires indicate earlier failure even though no failure occurs for the 48% TFM with lower number of fire floors.

In addition to varying the fire type and number of simultaneously heated fire floors, the effect of opposing and vertically travelling two-floor fires has been investigated. In general, the patterns of stress and deflection development are similar irrespective of fire spread direction. The results show that vertically travelling fires result in higher beam axial forces and deflections early during the fire exposure. However, simultaneous fires lead to shorter times to failure and could be used to represent the worst case scenario. Also, unlike in the published literature on the structures with long span composite truss systems, results show no significant cyclic movement of columns for vertically travelling fires. This could be due to a different structural system or a small number of fire floors and further studies varying the number of fire floors for vertically travelling fires need to be conducted.

Results show that fire type (travelling or a standard fire) and number of fire floors have a significant effect on the failure time and type of collapse mechanism. One single worst case fire scenario cannot be readily identified, especially considering the uncertainty in the number of fire floors likely to occur in a real fire.

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