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Structural analysis of multi-storey steel frames exposed to travelling fires and traditional design fires

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ABSTRACT

Most of the current understanding of building behaviour in fire is based on the adoption of the standard and parametric temperature-time fire curves. However, these design fires are based on small scale tests and idealize the thermal environment as uniform. Thus, they have important limitations on their applicability to large enclosures. Instead, in large open-plan compartments, travelling fires have been observed. To account for such fires, a design tool called Travelling Fires Methodology (TFM) has been developed and used for design. The aim of the present study is to compare computationally the structural response of a multi-storey steel frame subjected to both uniform design fires (available in current standards) and travelling fires. A two-dimensional 10-storey 5-bay steel frame designed according to ASCE 7-02 is modelled in the general finite element program LS-DYNA. Different fire exposures are investigated. They include travelling fires, Eurocode parametric curves, ISO-834 standard fire and the constant compartment temperature curve from the SFPE standard. These fires are applied to different floors, one at a time, to explore the influence on the structural response, resulting in a total of 80 different scenarios. The development of deflections, axial forces and bending moments is analysed. Uniform fires are found to result in approx. 15–55 kN (3–13%) higher compressive axial forces in beams compared to small travelling fires. However, the results show irregular oscillations in member utilization levels in the range of 2–38% for the smallest travelling fire sizes, which are not observed for any of the uniform fires. Peak beam mid-span deflections are similar for both travelling fires and uniform fires and depend mainly on the fire duration, but the locations in the frame and times when these peak displacements occur are different. The results indicate that travelling fires and uniform fires trigger substantially different structural responses which may be important in the structural design and selection of the critical members.
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1. Introduction

Most of the current understanding of building behaviour in fire is based on the adoption of the standard and parametric temperature-time fire curves. However, these design fires are based on small scale tests, assume flashover and therefore an idealized uniform thermal environment in a compartment. Thus they are only applicable to small enclosures (<100 m^2 [1]). In large open-plan compartments, fires have been observed to travel, resulting in a highly non-uniform temperature distribution within the enclosure [2]. These fires are referred to as *travelling fires*. Examples of such accidental events include the World Trade Centre Buildings 1, 2 & 7 (2001), the Windsor Tower fire in Madrid (2006), and the Plasco building fire in Tehran (2017).

The use of post-flashover design fires that assume uniform temperature conditions within a compartment that are used in codes are justified by the assumption that they result in the most severe condition for structural members and therefore represent the worst case scenario irrespective of the structure details and the structural metric examined. However, following the aforementioned travelling fire incidents, the structural fire engineering community expressed concerns about the validity of traditional post-flashover fires for large compartments. More specifically questions were raised about the effect of longer fire durations and the effect of non-uniform temperatures on structural performance. As a result, Stern-Gottfried and Rein developed a novel design methodology called Travelling Fires Methodology (TFM) to represent the travelling nature of fires in large compartments [3].

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Recently, TFM has been improved to account for more realistic fire dynamics and range of fire sizes and is referred to as Improved Travelling Fires Methodology (iTFM) [4]. In this methodology, the travelling fire size is varied to simulate cases where different percentages of the floor area are engulfed in flames at a given point in time. Temperatures in the compartment are described by two regions referred to as near-field and far-field. Near-field represents hot temperatures in the vicinity where fire directly impinges on the ceiling, and far-field represents the cooler smoke temperatures further away from the fire.

In recent studies [3,5,6] on the thermal response it has been found that structural members are likely to reach higher temperatures when subjected to travelling fires in comparison to uniform fires (i.e. Eurocode parametric temperature-time curves [7]). Higher temperatures lead to a higher loss of material strength. This is especially important for steel as it exhibits a rapid reduction in yield strength for temperatures higher than 400 °C and can drop by 25% of its room temperature strength at 500 °C [8]. The peak bay rebar temperature in the study by Stern-Gottfried and Rein [3] was found to be 556 °C under travelling fires, while for the same fuel load density considering Eurocode parametric curves, the rebar only reached temperatures from 252 °C to 363°. Law et al. [5] investigated the structural response of a concrete frame to travelling fires using an earlier version of TFM. The authors observed that the parametric curves lead to less severe conditions than travelling fires in terms of peak rebar temperature as well.

In terms of the structural response, in the studies [9,10] where a localized fire was considered, it was concluded that the assumption of uniform temperature distribution within steel beams may lead to unconservative results. The exposure of a structure to a travelling fire, which is a localized fire that moves, can have an even more adverse effect. In a study on a steel space frame exposed to a localized travelling fire [11] authors identified that travelling fire can result in more extensive fire damage than a stationary localized fire. This is not only due to the likely higher peak temperatures within members, as identified previously, but also due to the simultaneous heating and cooling at different locations within the structure.

The first study to consider structural response under travelling fires and to highlight the need for more realistic thermal field correlations was carried out by Bailey et al. [12]. In the computational study of a two-dimensional steel frame they observed that a travelling fire caused larger residual displacements in the source bays than a uniform fire by up to 92 mm (29%). Similar study has been undertaken by [13] assuming reduced loading on the structure and the same observations were made. Additionally, the work by [12,13] and recent work on the influence of travelling fires on composite construction [14] and post-tensioned concrete floors [15] indicated a cyclic behaviour of stresses and deflection development with time patterns in structural elements which have not been previously noted. However, to represent a travelling fire in these studies parametric temperature curves were used and shifted from one bay to another after a prescribed period. As noted by Stern-Gottfried and Rein [2], such representation ignores the pre-heating and post-heating of structural elements by hot smoke. They state that the latter might be the reason for the observation of a cyclic behaviour in structures subjected to travelling fires. Röben et al. [14] investigated the response of composite steel-concrete structures to vertically travelling fires. In this case, as identified by [2], observations of cyclic stresses seem to be more realistic because significant preheating of the upper floor (non-fire floor) prior to fire occupying it is unlikely.

Behaviour of structures subjected to travelling fires that assume non-uniform temperature distributions have been investigated by a number of researchers [5,11,16–19]. Röben [16] and Law et al. [5,17] applied an early version of TFM [20] to study the response of a composite structure and a generic multi-storey concrete frame, respectively, subject to travelling fires. Results in [16] showed larger displacements (by approx. 80–500 mm, 12–56%) and compressive axial forces (by up to approx. 1.7 MN) for uniform fires. However, for travelling fires, irregular (i.e. not symmetrical) displacement patterns that are not experienced in case of uniform fires, and larger residual tensile forces (by up to approx. 0.7 MN, 18%) and bending moments (by up to approx. 82.5 kNm, 5 times) have been observed. On the other hand, in the work by Kotsovinos [19], on the behaviour of tall composite buildings with a concrete core and perimeter long-span steel beams, travelling fires (using TFM [3]) were found to result in larger floor displacements (by approx. 380–1220 mm, 25–122%) and plastic deformations (by 30–90 mm, 2–7 times) compared to parametric fires but lower compressive (by approx. 155–270 kN, 24–38%) and tensile (by approx. 125–545 kN, 44–83%) axial forces in the steel beams. Work by [5,17] showed that travelling fire scenarios resulted in more onerous stresses within the concrete frame than parametric temperature-time curves. It was also identified in the latter studies [5,17] that fire sizes between 10% and 25% of the floor area resulted in the most severe conditions in the terms of critical deflection, rebar temperature, and strain. Utilization analysis of columns in [17] identified the fires of 5%-10% of the floor area to be the critical scenarios. Rezvani and Ronagh [18] investigated the structural response of a 2D moment resisting frame subject to travelling fire represented by TFM [3]. Results indicated that fire size can have a significant effect on the failure time and temperature as well (by up to 62% and 11%, respectively).

In the previously identified studies the differences between the effects of travelling fires and uniform fires on the structural response have been evaluated for a post-tensioned concrete floor [15], a concrete frame [5,17], a tall structure with a concrete core and perimeter long-span composite beam truss system [19], a composite construction [16], and a steel frame [12,13]. The differences between the effects of the travelling and uniform fires on the structural response appear to be mainly affected by the extended total duration of travelling fires, changing location of higher localized near-field or bay temperatures with time (i.e. travelling nature of fire), and structural system examined. These studies concluded that consideration of more realistic fire exposure such as travelling fires is important for the structural response, because such fires may induce higher stresses and deflections than uniform fires, and that a uniform fire assumption is not always the most conservative.

However, some of these studies [5,16,17] used earlier TFM versions where far-field temperatures were represented by a single temperature. The studies on a more generic steel frame [12,13] assumed uniform parametric fire temperatures for smaller bays and shifted them at arbitrary spread rates from one to another ignoring the effects of pre-heating as discussed previously. Parametric fires are based on different fire dynamics and therefore their use for non-uniform fires is not representative. In other studies on steel frames analysis was limited to the consideration of only one travelling fire scenario [11] and the behaviour of the same structure under uniform fire exposure was not considered [11,18]. Therefore, more research on steel frames is still needed to investigate the differences between the effects of uniform fires and more realistic travelling fire exposures and whether the same differences in structural responses are observed as in [12,13].

This study has two aims. The first aim of this study is to extend the work on the structural response of a generic multi-storey steel frame subjected to a more realistic travelling fire exposure [4], which unlike previous studies on steel frames [12,13] accounts for spatially varying temperatures in the compartment, physically possible fire spread rates, and temperatures. The second aim of this work is to investigate how the structural response of the frame

changes with fire occurring on different levels of the same building, which to the best knowledge of the authors has previously only been considered on a 3-storey steel frame [21]. The building examined in this study is a very generic steel frame based on the case study presented by NIST on typical US construction. As a result, although the conclusions of this paper are specific to this steel structure, they are likely to have a much wider applicability compared to previous studies on concrete buildings or other more unique structural forms.

2. Finite element model

2.1. Multi-storey frame

The multi-storey steel frame considered in this analysis is based on the moment resistant frame published by NIST [22]. 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 American Society of Civil Engineers (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) [22]. In this paper, the unfactored design loads where used as the combination for the fire limit state. The beam sections are W14 × 22 on all floors. The column sections on Floors 0–3, Floors 4–6, and Floors 7–8 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 is not taken into account, which has been shown to have a beneficial effect on the structural response during fire as a result of tensile membrane action [23]. However, the effect of cooling due to the presence of the concrete slab on the steel beams is considered in the heat transfer analysis as discussed in Section 2.3. In previous numerical studies on composite structures [16], a high-rise moment-resisting steel frame [24], and tall composite buildings with a concrete core and perimeter long-span steel beams and trusses [19,25], it was found that a 2D model using beam elements, in general, gives a good representation of the structural response to fire when compared to the 3D model using beam and shell elements. In a previous study on composite structures [16], the effects of the concrete slab on the structural response were determined to be more significant during cooling than heating, leading to reduced axial forces and higher residual moments as the slab cools down more slowly than steel beams. Also, in the same study, the 2D model was observed to underestimate deflections, but the load due to the concrete slab was not taken into account, which could have resulted in lower values compared to the 3D model. However, the results showed a close agreement in the trends in global structural behaviour. In a study on tall composite buildings with a concrete core and perimeter long-span steel beams and trusses [25], it was also observed that the 2D model is less redundant and, therefore, results in a more onerous response. This is because, unlike the 3D model, in the 2D model redistribution of forces is to a lesser extent of the structure. For the cases investigated in this study, if composite action between the steel beam and the concrete slab was considered in the model, it would likely result in lower beam deflections horizontal displacements due to a stiffer response of the concrete slab. It could also lead to lower tensile axial forces during cooling and thus affect the observations of this study in particular for the smaller travelling fires, where structural members in different areas of the floorplate experience heating and cooling simultaneously for long durations during the fire exposure.

In general, the 2D models in previously identified studies [16,19,24,25] were found to be conservative and show a good agreement qualitatively with 3D models. Therefore, the 2D representation is considered to be acceptable for this study as the

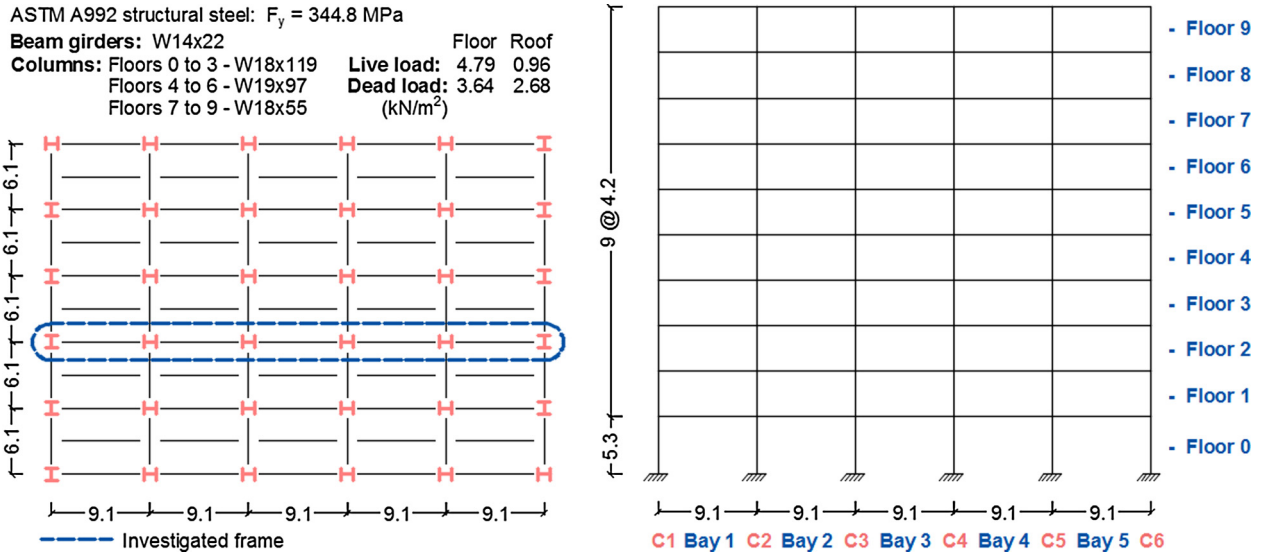


Fig. 1. Plan layout, elevation and structural member details of the investigated frame [22]. Frame dimensions are given in meters.

primary objectives are to analyse the general trends and to compare the outcomes of the model for the different fire scenarios considered. In addition, a 2D analysis has been chosen for the reasons of simplicity, computational time to allow comparison of many different fire exposures (i.e. 80) 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. Moreover, a consistent level of crudeness in the assumptions regarding the fire definition and the structural modelling is important. The travelling fire model adopted in this paper is a simplified fire model developed for design purposes and based on simple algebraic relationship (in a similar fashion to the parametric fires). As a result, the level of complexity of the fire models used in this study, and the relatively simplified structural model, are consistent and in line with standard design practice where simplifications are necessary.

2.2. Fire scenarios

The structural response of the frame subjected to travelling fires (TFM) [4] and standard design fires such as Eurocode (EC) parametric temperature-time curves [7], the standard fire (ISO) [7,26], and the SFPE constant compartment temperature design fire [27] is investigated.

To represent a travelling fire exposure, iTFM [4] is used. It is the most recent version of the Travelling Fires Methodology (TFM), which was developed by [2,3,5]. Key difference with any other scenario for structural design is that TFM considers the non-uniform temperature distribution in the compartment and the long fire durations observed in many large fires. It should be noted that the methodology, although it represents the state of the art of the field, has not been extensively validated yet due to the lack of experiments in large scale compartments. However, the standard fire and Eurocode parametric curves are not validated for large compartments either [2,3]. The travelling fire model presents an idealised exposure for studying the structure under non-uniform fires with the aim of removing complexities and simplifying its use but equally capturing the key phenomena experienced by non-uniform fires. This is similar to other loadings such as earthquake, blast, or wind that are simplified in terms of representing their key responses to the structure for design purposes, rather than representing the event in a more complex form. Despite its relatively recent inception, the methodology has already been used by several consultancies as a complementary design tool for the design of 39 iconic buildings in UK (primarily open-plan offices, e.g. [28,29]) and in previous studies has been found to be more onerous than other scenarios [3,5,12,13,16,17,19]. More experimental research in large compartments is still needed to advance the methodology further and represent more realistic fire dynamics by fitting experimental data when it becomes available in the near future. Although, travelling fires have already been observed in a number of experiments [2].

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 floor area has uniform fuel and, once alight, burns at a constant rate [2,3]. Thus, fire size is governed by the fire spread rate. TFM considers design fires to be composed of two moving regions: the near-field (flames) and the far-field (smoke) (see Fig. 2). The near-field represents the flames directly impinging on the ceiling and assumes the peak flame temperatures measured in real fires [2]. The far-field model represents smoke temperatures, which decrease with distance away from the fire due to mixing with air. 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 [4], 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 on a concrete frame [3,17]. 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) and 500 kW/m^2 (typical value for densely furnished places) [3], respectively.

Two Eurocode [7] parametric temperature-time curves are considered, representing short-hot and long-cool fire exposures based on the study by Lamont et al. [30]. The short-hot fire is characterized by high temperatures and short duration, while long-cool fire is characterized by lower temperatures and longer duration. In [30] it was found that these two parametric fires resulted in different structural behaviour, and for this reason they are included in our study. For the structure examined in the latter study [30] the short-hot EC fire resulted in large initial deflections within a short period of time, while the long-cool EC fire resulted in larger deflections but later in time as a result of different heating rates and durations. The EC parametric curves were generated assuming the same fuel load density as for travelling fires (570 MJ/m^2) and opening factors of 0.176 m$^{0.5}$ (short-hot) and 0.044 m$^{0.5}$ (long-cool). These opening factors correspond to 100% and 25% glass breakage (assuming a weighted average window height of 2.5 m), respectively.

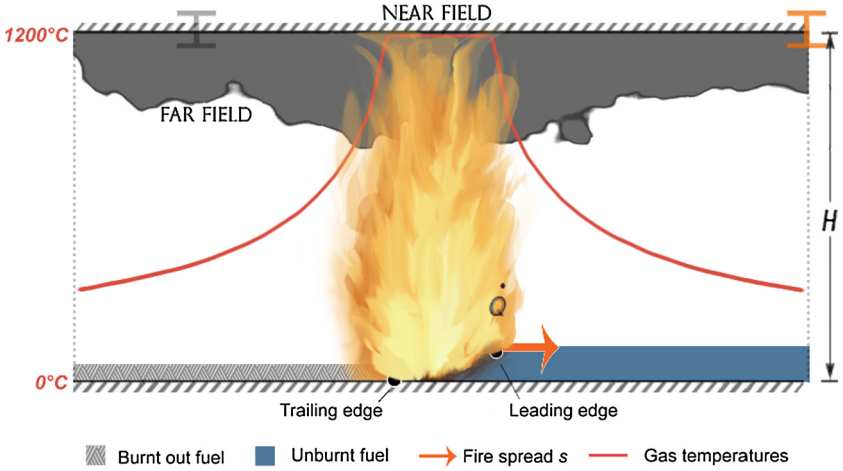


Fig. 2. Illustration of a travelling fire and distribution of gas temperatures in the near-field and far-field [4].

The correlation to represent the standard fire (referred to as the ISO standard fire in this paper) is taken from the Eurocode [7]. Only 60 min of standard fire exposure are considered because of the 60 min fire resistance given to the steel beams (see Section 2.3). In addition, the design fire scenario by the SFPE standard [27] is included in our work. In the SFPE standard, a constant and uniform compartment temperature of 1200 °C is defined until the calculated burnout time. For different opening factors the burnout time can vary, from approx. 15 min to 5 h or longer depending on the assumed glass breakage. For this study, the temperature of 1200 °C is kept constant for 60 min, which corresponds to 100% glass breakage (as in the EC long-cool fire scenario). This fire scenario is referred to as SFPE. SFPE standard fire scenario was chosen to represent the worst case uniform fire in terms of heating exposure, i.e. even more severe than ISO. It should be noted that the method is not explicitly based on physical parameters.

Each fire scenario is considered on every floor of the frame, one at a time. Therefore, in total 80 fire scenarios are investigated. Illustration of the gas temperatures for all fire scenarios at the mid-span of Bay 2 and at the right end of Bay 5 is shown in Fig. 3. By the definition for the parametric fires (EC), the standard fire (ISO) and the SFPE fire, the temperatures are assumed to be uniform across the whole compartment. Thus, for these scenarios, the temperatures shown in Fig. 3 at the two locations are identical. However, the travelling fire gas temperatures for Floor 0 are lower in comparison to the other floors because of the higher column height (Floor 0–5.3 m, Floors 1 to 9–4.2 m). Alpert's correlation [31] used to define gas temperatures in the iTFM is a function of ceiling height.

It should be noted that the authors do not claim that all the design fires assessed in the paper would need to be used as part of a commercial design project. The selection of design fires will inevitable vary between different countries and engineering teams based on local best practice, jurisdiction requirements, the project objectives (life safety, property protection etc.), building and structural layout and other fire safety measures (compartmentation, sprinklers etc.). The parametric study of the paper is extensive for research purposes and to arrive into a wider range of conclusions. There are already a number of probabilistic approaches published in literature (such as those by Arup and Buro Happold [28,29]) where travelling fires are considered effectively in the design.

2.3. Heat transfer

Beams and columns are designed for 60 min and 120 min standard fire resistance respectively using Eq. (1) [32,33].

$$t = 40(T_{lim} - 140)\left[\frac{d_i/k_i}{H_p/A}\right]^{0.77} \quad (1)$$

where t is the time (min), T_{lim} is the limiting temperature of steel (°C), d_i is the thickness of the insulation (m), k_i is the thermal conductivity of the insulation (W/m·K), H_p is the heated perimeter of the section (m), and A is the cross-sectional area (m^2).

A limiting temperature of 550 °C is commonly accepted as the critical temperature for steel in traditional design [34] and, therefore, is used as T_{lim} in Eq. (1). At 550 °C steel maintains only 60% of its ambient temperature strength because of the thermal degradation of its mechanical properties.

The main aim of this work is to analyse and compare the structural behaviour of the frame over a long fire exposure (i.e. travelling fires and uniform fires). Thus, to avoid an early failure of the frame during the fire, a higher fire resistance period is assumed for the columns. In the UK, the typical prescriptively required fire resistance standard for an office building with a height to the last occupied floor of 38.9 m would be 120 min [35].

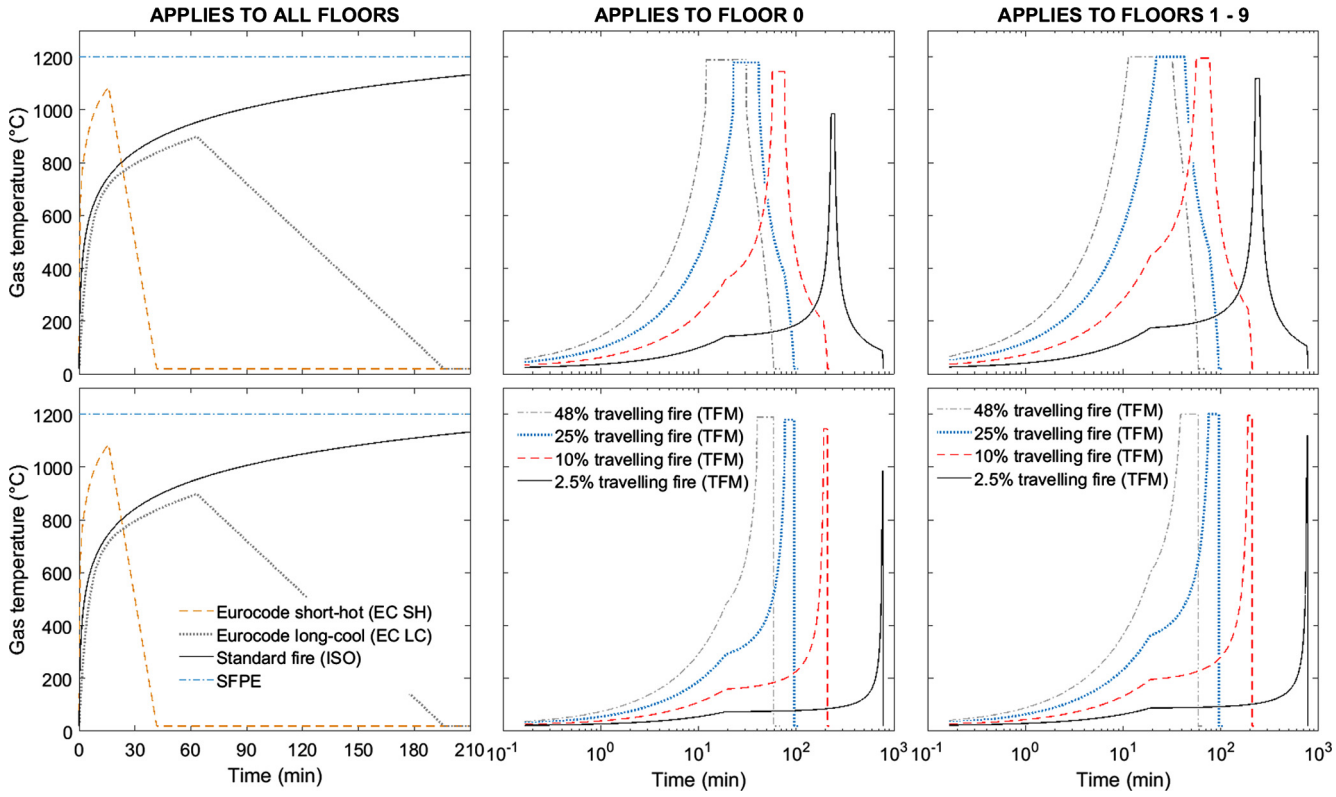


Fig. 3. Gas temperature histories at mid-span of Bay 2 (top) and right end of Bay 5 (bottom) for the eight fire scenarios on each floor of the frame: EC, ISO, and SFPE fires for all floors (left) and TFM fires for Floor 1 (middle) and Floors 1–9 (right).

Steel insulation properties are taken as for high density perlite (thermal conductivity $k_i = 0.12$ W/m·K, density $\rho_i = 550$ kg/m^3, and specific heat $c_i = 1200$ J/kg·K) [32]. Heat transfer to the structural members was carried out, assuming lumped capacitance for separate parts of the cross-section (i.e. web and flanges) according to [3,32], as shown in Eq. (2).

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

where T_s is the steel temperature (K), T_g is the gas temperature (K), ρ_s is the density of steel (kg/m^3), c_s is the temperature dependent specific heat of steel taken from the Eurocode [8] (J/kg·K), and Δt is the time step (s).

For beams, the effect of the concrete slab was considered by excluding the top surface of the upper flange, which is in contact with the slab, in the calculation of the heated perimeter. Thus, adiabatic boundary condition was assumed, though, in reality there would be some heat losses into concrete resulting in an even lower temperatures at the top flange. 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 [32]. 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 those at the same location in the ceiling, and temperatures along the column height are assumed to be uniform to represent the worst case scenario. An illustration of temperature development and distribution in the beam and column sections is shown in Fig. 4.

2.4. LS-DYNA model

The multi-storey steel frame is modelled using the general purpose finite element program LS-DYNA (Release 7.1.1) explicit solver. The program was originally developed specifically for highly nonlinear and transient dynamic analysis. LS-DYNA is capable of simulating the thermal and thermal-structural coupling analysis and has an extensive element and material library, including the temperature dependent material models from the Eurocode for steel and concrete. Prior to this analysis the program was validated and verified by the authors against the available benchmarking and fire test data for structural fire analysis [36]. In previous research, LS-DYNA has been used in structural fire applications for the analysis of tall structures [37] and structural arrangements with bilinear columns [38]. 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 [39] beam element formulation, with a cross-section integration refinement factor of 5. Hughes-Liu beam elements allow for the treatment of finite strains and are simple, computationally efficient, and robust. Beams, Floor 0 columns, and Floor 1–9 columns are divided into 36, 22, and 16 beam elements, respectively. The corresponding beam element length is approximately 0.25 m for all structural members. 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 localized failures in the beams, columns, or in the beam-to-column connections. It should be noted that 3D global effects are not considered in this paper.

A thermally-sensitive steel material type MAT 202 formulation based on Eurocode 3 (EN 1993-1-2:2005 [8]) is used for both steel beams and columns with the default temperature-dependent material properties. In this material model strain-hardening is not taken into account. Steel with initial yield stress of 345 MPa, Young's modulus of 210 GPa [40], and Poisson's ratio of 0.3 [40] is assigned to all members. Both mechanical and gravity loads are considered. Simulations are carried out using the explicit solver of LS-DYNA that uses real-time units to solve the equation of motion. Thus, in order to avoid artificial, high dynamic oscillations, the mechanical and gravity loads are applied in a linear increment over 1 s and then kept constant for the remainder of the analysis. After 2 s, that is once the steady-state solution is attained, thermal loads are applied. Thermal loading to the beams and columns was applied using the formulation which allows the definition of a variable through-thickness temperature distribution as calculated in the previous section. The remainder of the frame is assumed to be at room temperature. In order to reduce the computational

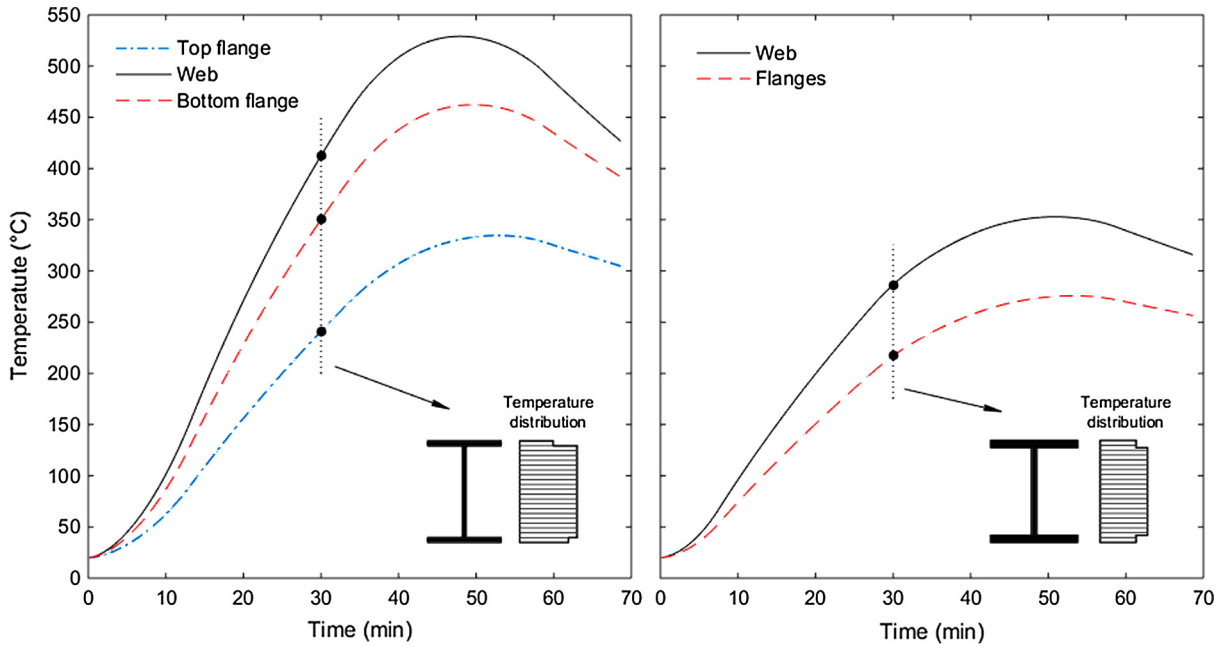


Fig. 4. Temperature development and distribution in beam section at mid-span of Bay 2 (left) and column 2 (right) for Floor 2 exposed to the 48% travelling fire.

time, the temperature development within heated members 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. This means that parametric curve which would last 120 min in physical time would be applied in 1.2 min in the simulation time. Once the simulations were completed, data on the development of axial forces, bending moments and displacements in frame were extracted and are analysed in the following section.

3. Results and discussion

3.1. Thermal response

A comparison of the beam and column web temperatures for all fire scenarios is illustrated in Figs. 5 and 6, respectively. They show that temperature distributions for a frame subjected to travelling fire scenarios are highly non-uniform in comparison to traditional design fires, as could be expected. As the fire travels along the compartment the magnitude of temperature variation decreases for larger fire sizes from 322 °C (25% TFM) and 257 °C (48% TFM) to 218 °C (25% TFM) and 123 °C (48% TFM). For the smallest fire sizes temperature differences in the frame are in the order of 550 °C (2.5% TFM) and 400 °C (10% TFM) at different times. Peak temperatures develop close to the end of fire and towards the end of the fire path, as observed in [4]. Peak temperatures reached in the beam web for different fire scenarios are 692 °C for 10% TFM, 682 °C for 2.5% TFM, 635 °C for 25% TFM, 620 °C for EC long-cool fire, 559 °C for 48% TFM, and 367 °C for EC short-hot fire. The highest peak temperatures and the variation in temperatures for the various TFM scenarios occurs for the smallest travelling fire size of 2.5%. Column temperatures on different floors do not reach

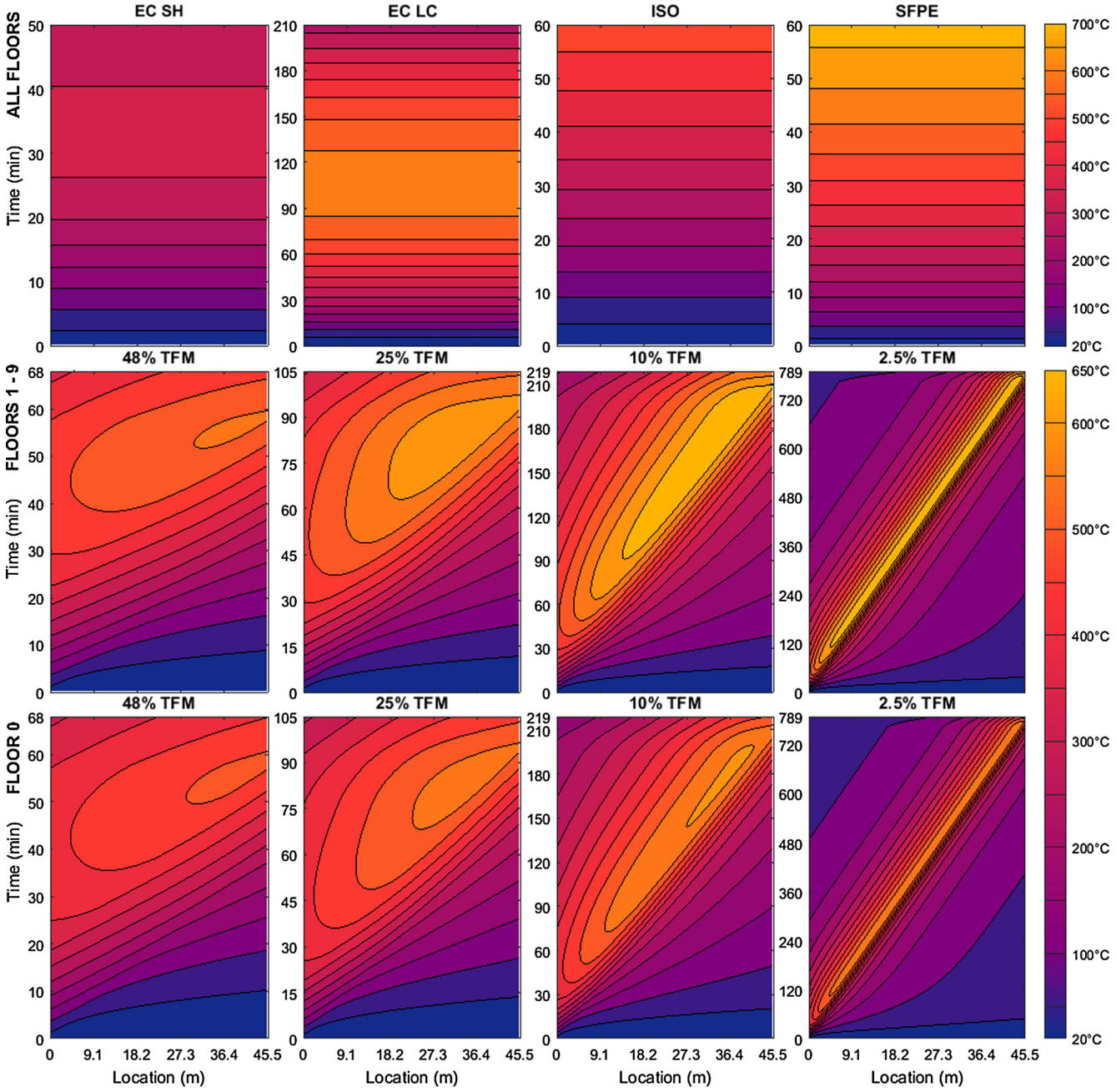


Fig. 5. Beam web temperature variation with time and location along the fire path in the heated floor for the eight fire scenarios: EC, ISO, and SFPE fires in all floors (top) and TFM fires in Floor 1 (middle) and Floors 1–9 (bottom).

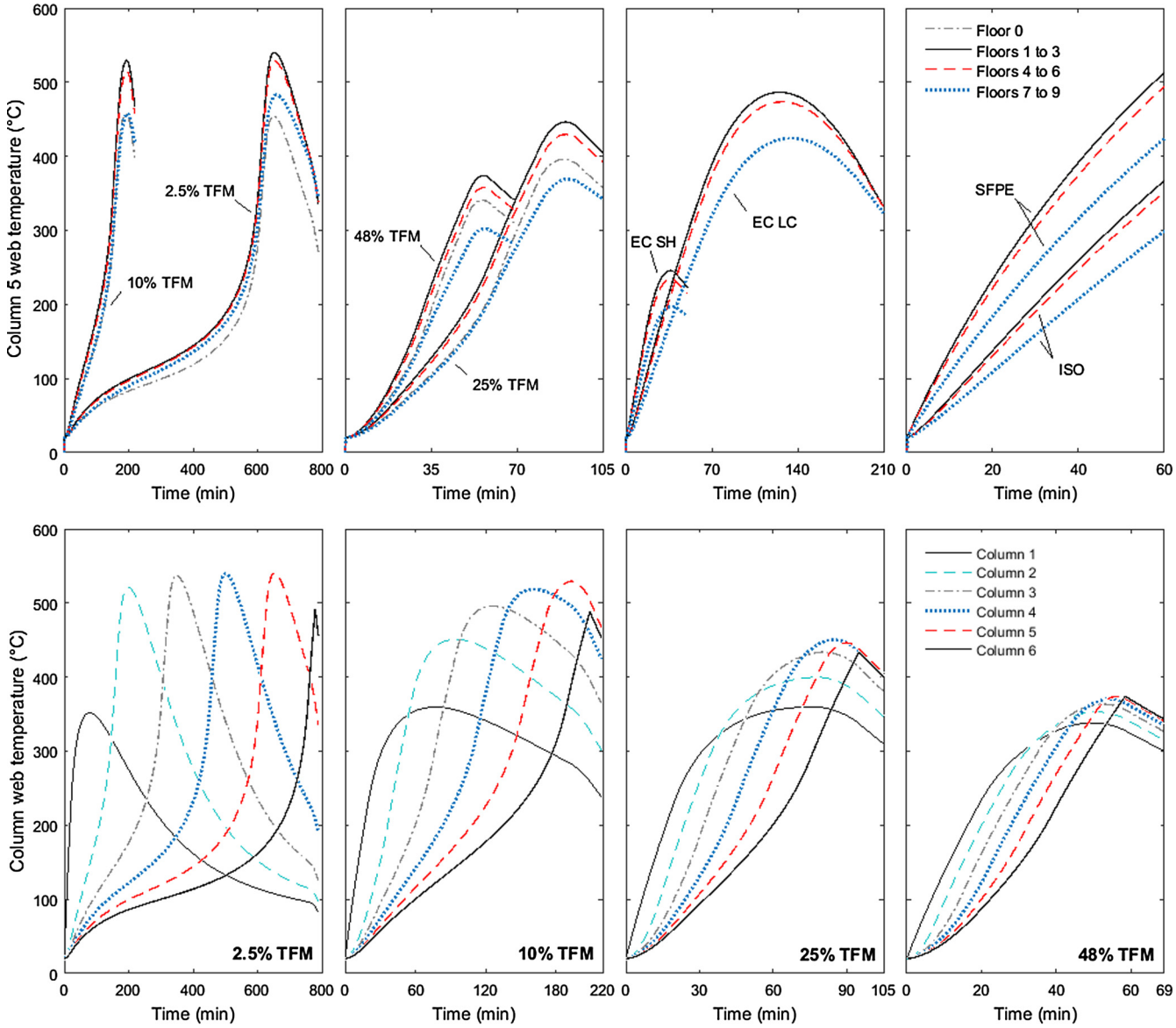


Fig. 6. Column 5 web temperature development in the heated floor for the eight fire scenarios (top); and comparison of temperature development in all heated columns on Floor 2 exposed to travelling fire scenarios (bottom). For EC, ISO and SFPE fire scenarios temperatures of all columns on the heated floor are equal.

similar values because of different column section sizes. For travelling fire scenarios, columns on Floors 1–3 experience up to 90 °C higher temperatures than columns in Floors 0 and 7–9. Temperatures in Floor 0 are low due to high column height as identified in Section 2.2. For uniform fire scenarios (EC, ISO and SFPE), temperatures for all columns on the same floor are equal and the highest temperatures are reached in Floors 0–3.

3.2. Effect of location of fire floor

The development of the beam mid-span displacements, axial forces and bending moments with time for the frame subjected to the 48% travelling fire is shown in Fig. 7. For the illustration of the results for other fire scenarios the reader is referred to Appendix A. Shaded areas represent the range of the displacements and axial forces which develop within the specific beam in relation to the fire location (i.e. floor immediately above or below the fire floor) for different fire floors. 'Floor 0', 'Floor 8' and 'Floor 9' indicate the floors the fire is located on. The development of mid-span displacements, axial forces and moments on these floors is different compared to fire occurring on the intermediate floors due to the reduced number of floors above or below the fire floor and different column sizes, i.e. different level of lateral axial restraint from the surrounding structure. For example, beams on Floor 9 are only connected to the columns on the same floor leading to a low level of axial restraint to thermal expansion, while beams on intermediate floors are connected to and restrained by the columns on the floor above as well. This results in higher axial restraint and consequently higher axial forces by up to 240 kN (13% of the heated section capacity) in intermediate floors as can be seen in Fig. 7. For the 48% TFM at 20 min the difference is up to 300 kN, which is approx. 21% of the maximum axial force capacity of the section at that time.

The results indicate that in general the development of stresses and displacements follows a similar behaviour pattern for all members, even though the fire occurs on different floors. The lowest limiting axial force values correspond to fire occurring on Floor 8. In the bottom floors, the axial force in the heated beams increases by approx. 65 kN (17%) for the 25% TFM, 48% TFM, and ISO fire. This difference goes up to 90 kN (29%) and 124–196 kN

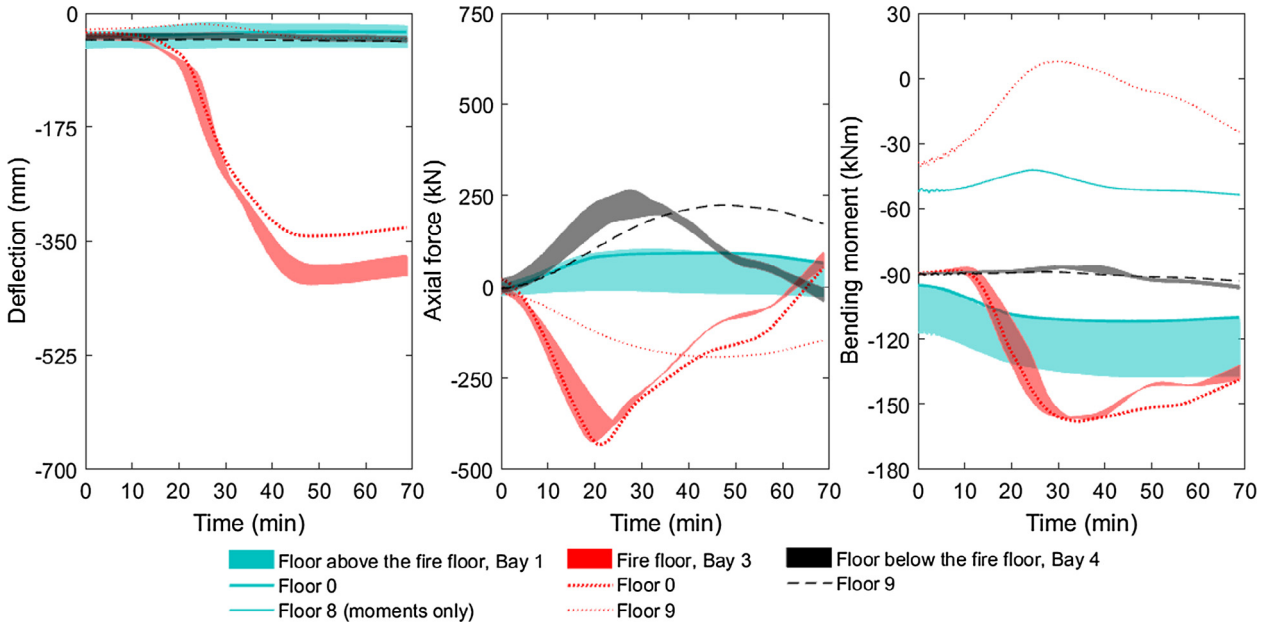


Fig. 7. Variation of displacement, axial force and bending moment development in beams exposed to a 48% travelling fire scenario. Shaded areas represent a range of results when fire occurs on Floors 1–8. Individual lines indicate results when fire occurs on ground Floor (0), Floor 8 (moments only), and top floor (9).

(50%–180%) for the 10% TFM and 2.5% TFM, respectively. These values correspond to 5% (25% TFM, 48% TFM, and ISO), 6% (10% TFM), and 9–14% (2.5% TFM) of the yield axial force section capacity. This is because heated beams on the top floors of the frame are supported by column sections with a lower cross section capacity than the beams on the bottom floors. Thus, even though these columns reach lower temperatures (see Fig. 6) the axial restraint to thermal expansion is smaller in the upper floors. The variation in peak axial forces with different fire floors for the smallest travelling fire sizes (2.5% and 10%) is the highest. This is probably due to the higher variation of temperatures in the frame. Some of the members on the fire floor in the far-field region are relatively cool, thus, providing more axial restraint. Also, the peak axial forces during 2.5% and 10% travelling fires develop much later (at 50–160 min) in comparison to larger travelling fires and uniform fires (20–30 min).

In the cases when the fire occurs in the upper floors, the initiation of yielding within the heated beams occurs up to 15 min later than in the cases when fire occurs on the lower floors. Typically, yielding takes place when the compressive axial force begins to decrease, followed by elasto-plastic response and a sudden increase in deflection [41]. After yielding, the influence of the fire floor location becomes less significant. Higher displacements (by approx. 30–55 mm, or 6.8%–9.4%) develop within the beams when the fire occurs on the top floors of the building rather than the bottom. These observations agree well with the findings from other researchers who investigated the effects of axial restraint stiffness on heated beams [42–44]. They found that, in the cases where beam behaviour is dominated by restrained thermal expansion, higher axial restraint results in the development of higher compressive forces and lower deflections.

The level of axial restraint on the ground floor beams is affected by more factors than in the intermediate floors, which makes it more difficult to judge and make comparisons. On one hand, ground floor columns have fixed supports, which should lead to higher axial restraint but, on the other hand, these columns are more slender (5.3 m high) (i.e. have a lower bending stiffness) than the columns in the upper floors (4.2 m). In addition to that, for travelling fire scenarios members on the ground floor are exposed to lower temperatures in comparison to the upper floors. Thus, slightly higher axial forces and lower deflections occur for TFM scenarios because of the combination of previously identified factors. Results for the ISO fire, which are not affected by different thermal conditions on different floors, indicate that the ground floor beams (i.e. structural beams located at Floor 0) have the highest axial restraint in comparison to intermediate floors. This is because of the higher peak compressive axial forces and lower deflections after yielding of the beam occurs. Analogous results for displacement, axial force, and bending moment in different fire floors were observed for all other scenarios (i.e. EC parametric and SFPE fires).

3.3. Axial force redistribution

Fig. 8 shows the development of axial forces within different beams in the frame for a 25% travelling fire on Floor 2. In each bay the development of axial forces in the heated floor and in the floors above and below the fire follows the same trend. Compressive axial forces develop within the heated beams, while tensile axial forces develop in beams in the adjacent floors. This is because the beams in the adjacent floors have to resist the lateral movement of the columns as the beams on the fire floor try to expand. Therefore, they provide the axial restraint to thermal expansion. The highest axial forces develop in the beams in the floors immediately above and below the fire floor. The axial force within these beams is approximately increasing from 60 to 80% (Floors 0 and 2) of the axial force in the heated beams with time. This drops to from 5 to 30% on the floors further away from the fire by one floor. For a fire occurring on the top floor, axial force development in the floors below is even higher as there is no available axial restraint from any floors above the fire (see Appendix A). In this case the axial force in beams one floor below the fire (Floor 8) is from 90 to 110% for end bays and 140–180% in internal bays of the axial force in the heated beams. It drops to 0–15% (end bays) and 45–80% (internal bays) in Floor 7. It should be noted that conventional guidance does not consider the effect of load redistribution and the increased utilization at other levels during a fire.

No significant axial forces were observed in any other floors. This indicates that the stress distribution is negligible in the frame floors more than 2 floors away from the fire. Later, in Section 3.4.3, it will be shown that the same observation is made for the bending moment distribution in columns. Therefore, modelling only these

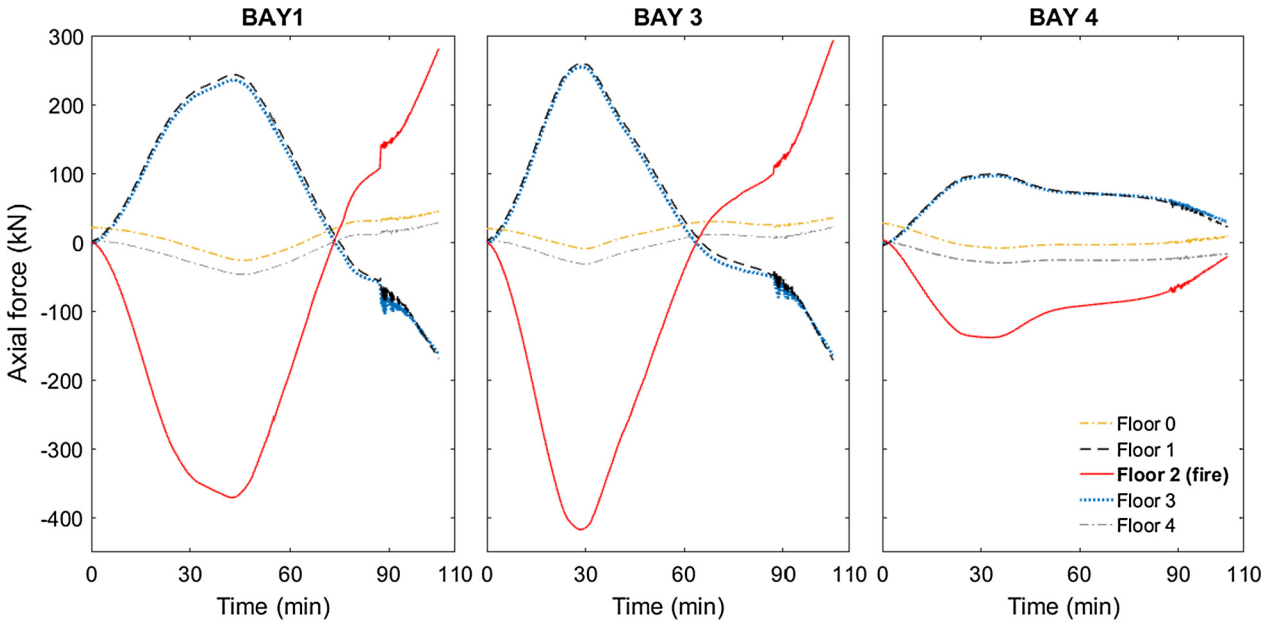


Fig. 8. Variation of axial forces within different bays for a frame subjected to a 25% travelling fire on Floor 2. Bays 1 is the end bay and Bays 3 and 4 are the internal bays of the frame.

floors of the frame is likely to be sufficient to capture the correct restraint conditions and stress development. However, more studies on different geometries should be carried out to confirm this, and the column boundary conditions need to be carefully chosen by designers. In agreement, Law [45] in his study on perimeter column boundary conditions noted that representing 3 floors and 4 column heights resulted in the most significant improvement in accuracy of column bending moment prediction. Similar results as presented in Fig. 7 and discussed in this section were observed for all other fire scenarios occurring in the intermediate floors.

3.4. Effect of fire scenario – travelling fires and uniform fires

3.4.1. Deflections

The comparison of axial force, bending moment, mid-span deflection and lateral displacement development in the heated

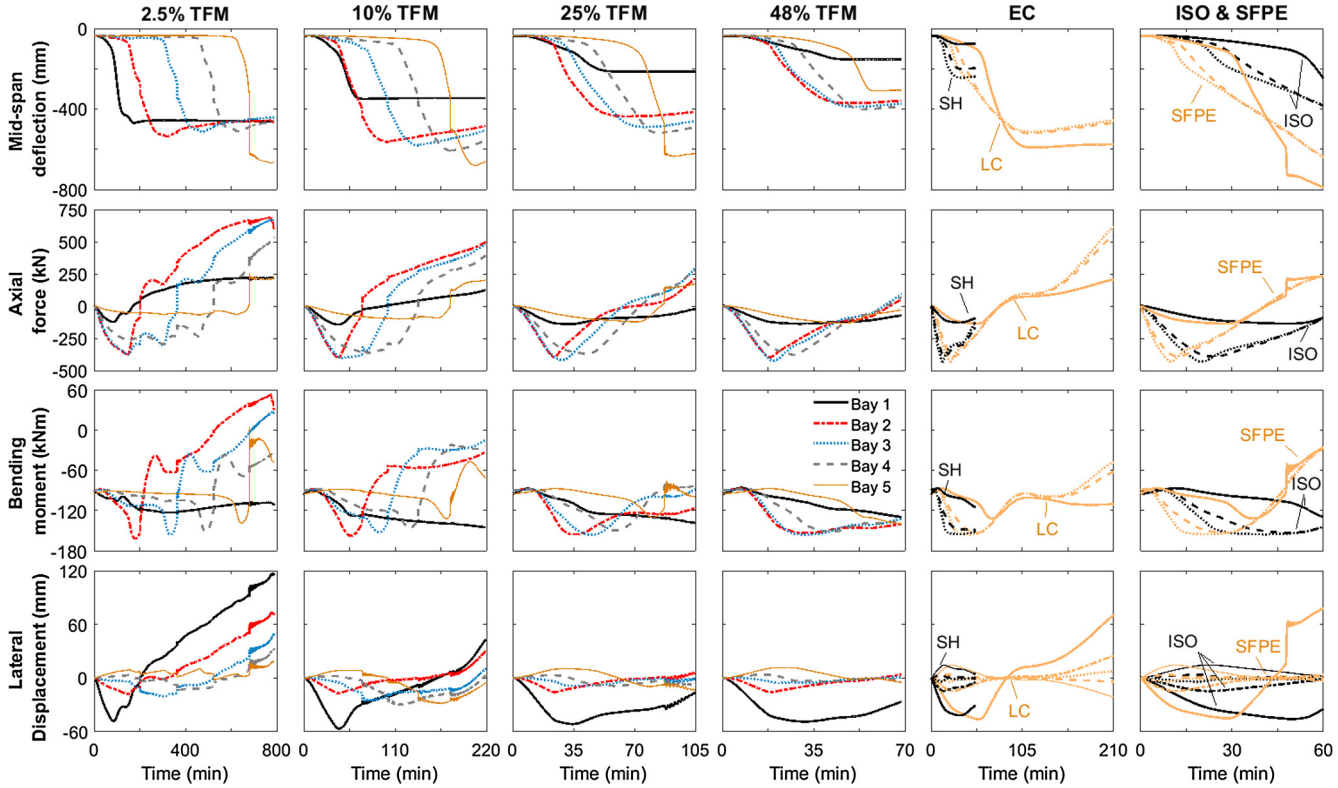


Fig. 9. Development in time of mid-span deflections, axial forces, and bending moments at each of the beams in Floor 2; and development of lateral displacements at the beam to column nodes on the left hand side of the beam. Fire occurs on Floor 2.

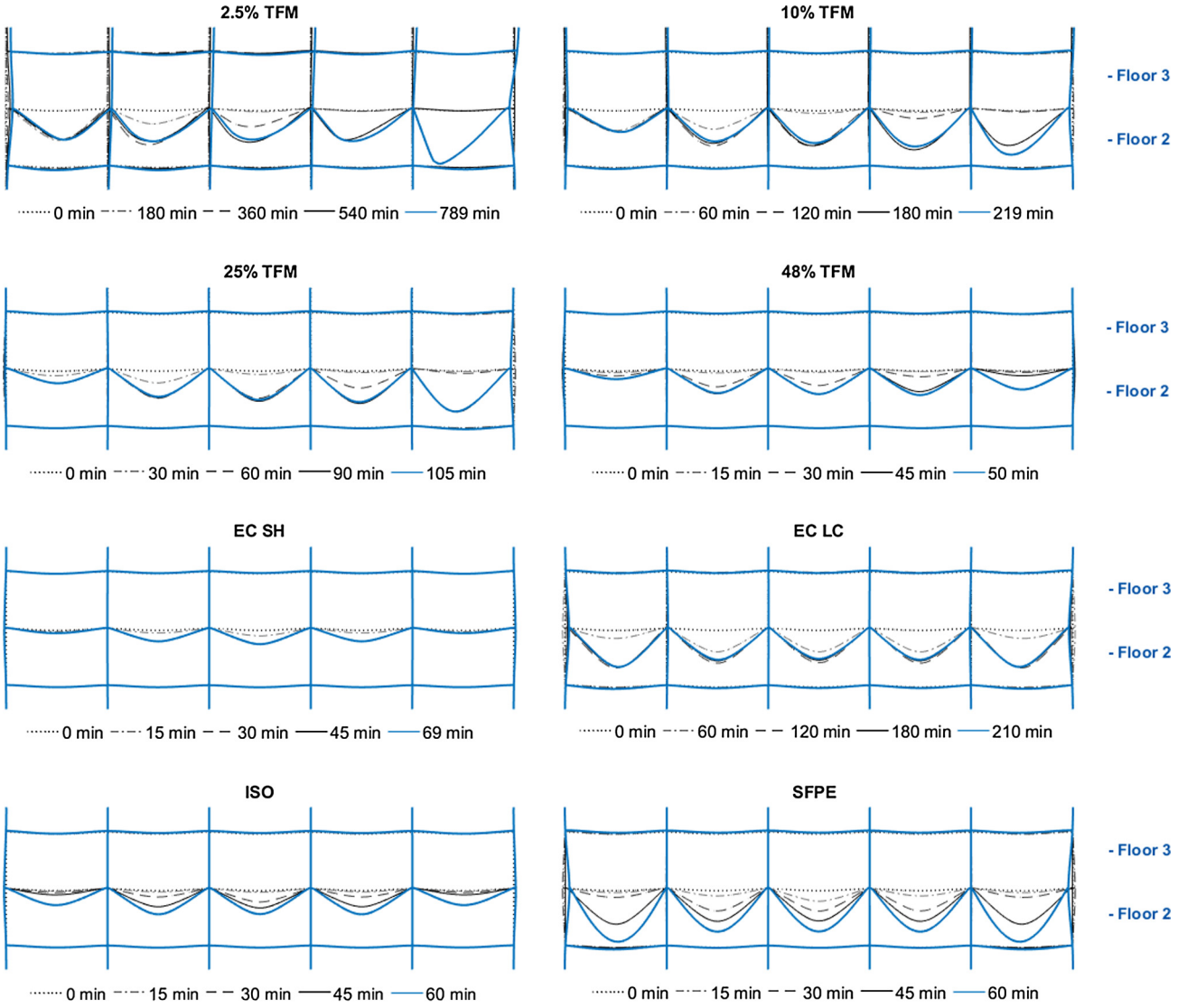


Fig. 10. Deflected shape of the frame at different times of fire exposure for the eight fire scenarios investigated. Fire occurs on Floor 2. Displacement scale factor is 5.

beams on Floor 2 is shown in Fig. 9. The typical deflected shape of the frame for different fire exposures on Floor 2 is shown in Fig. 10. For all travelling fire (TFM) scenarios beam displacements in Bay 1 (end bay) are relatively low in comparison to other bays. Once the cooling begins, the displacements remain constant while in the other bays there is a small recovery. This is because the beams in Bay 1 reach lower temperatures than the beams in other bays, thus resulting in lower thermal expansion and lower compressive axial forces in Bay 1. For the same reason, the peak displacement reached in Bay 1 keeps on increasing with decreasing fire size as the beam is exposed to the near-field for a longer duration. Due to a low axial restraint available, the majority of thermal stresses go into the development of displacements. A similar beam response occurs in Bay 5, but in this case peak displacements are the highest in the frame because beams reach higher temperatures. It can also be observed that for TFM sizes of 10%, 25%, and 48%, the highest axial forces develop in Bay 2 followed by Bay 3 and Bay 4. This is because as the fire progresses more of the structure is heated leading to lower stiffness and thus available restraint.

Higher displacements initially develop in Bay 3 for the standard ISO, SFPE and EC fires and in Bays 1 or 2 for the travelling fires. For the standard fire, SFPE fire, short-hot EC fire, and 48% TFM, displacements develop more rapidly at the early stages of the fire. However, the peak mid-span deflections reached during the latter scenarios (40 cm for the 48% TFM and 24 cm for the EC short-hot fire) are at least 20 cm lower than for other fire cases. No cooling phase for ISO and SFPE fires is considered; therefore, deflections are the highest and would continue increasing until the failure of the frame (i.e. when the loadbearing capacity is reached).

In the work by Bailey et al. [12], Liew et al. [13] and Kotsovinos [19] it was found that travelling fires produced higher displacements than parametric fires for the range of fire sizes and type of structures that the authors examined. However, in the work by Röben [16] the opposite conclusions were made. Ellobody and Bailey [15] reported that maximum displacements can develop either during the travelling fire or uniform fire for the case study examined in the paper, a fully protected MRF. The results shown in Fig. 9 agree with the latter and indicate that it is not the fire type which governs the displacements but the fire duration. In general, in our work, maximum deflections reached are higher for fire scenarios with longer fire durations (2.5%, 10% and 25% travelling fires and EC long-cool fire) than for shorter and hotter fires (45% TFM and EC short-hot fires). This agrees with the findings by Lamont et al. [30] for a protected steel-concrete composite structure

regarding the short-hot and long-cool parametric fires. Short and hot fires were observed to result in faster initial displacements, while long and cool fires resulted in larger maximum displacements but occurring later during the fire. Thus, travelling fires, depending on their fire spread rates and thus duration, could be grouped into similar categories as well (such as "slow" and "fast" travelling fires).

The main difference between travelling fires and uniform fires appears to be that these fires result in different locations where the maximum deflections develop at different times (see Fig. 10). For travelling fires and uniform fires, initially the highest deflections develop in the end bays and central bay, respectively. However, towards the end of the fire, irrespective of fire scenario (TFM or uniform fires), for long-cool fires the peak displacements develop in end bays, while for short-hot fires (EC short-hot fire and 48% TFM) they develop in internal bays. Displacements for travelling fires are also more irregular along the frame as observed in the work by Röben [16].

3.4.2. Axial forces

For all fire scenarios, compressive axial forces in end bays (Bay 1 and Bay 5) are significantly lower than in internal bays. This is because, as identified previously, the axial restraint to thermal expansion is provided by only one column on one of the sides. The highest axial forces develop when the frame is subjected to large fire sizes (e.g. ISO, SFPE, EC, and 48% TFM). Peak axial forces

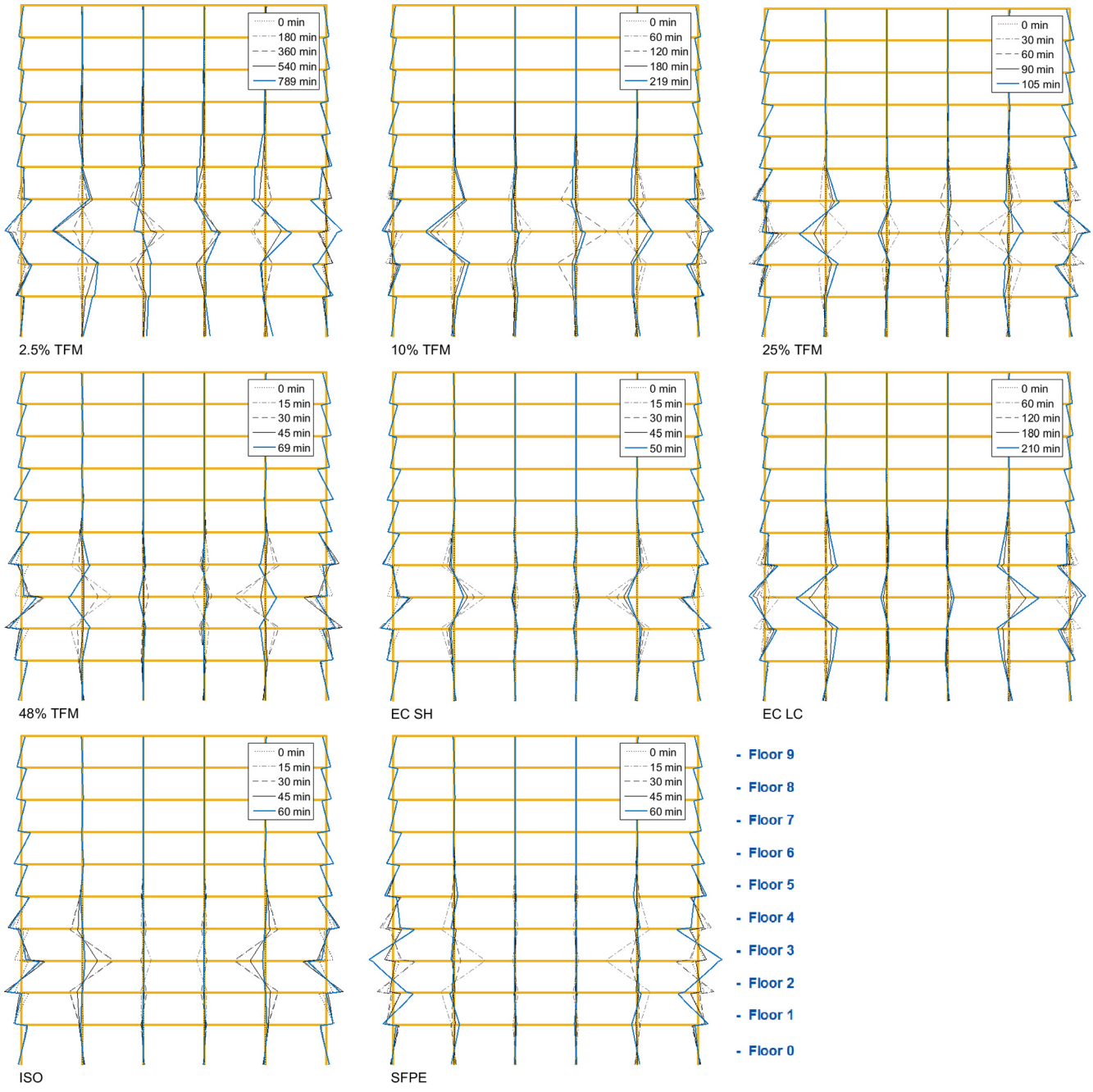


Fig. 11. Bending moment distribution in the heated columns at different times of fire exposure. Fire occurs on Floor 2. Peak bending moments correspond to: 2.5%TFM – 599 kNm, 10%TFM – 560 kNm, 25% TFM – 377 kNm, 48% TF – 360 kNm, EC SH – 410 kNm, EC LC – 535 kNm, ISO – 348 kNm, and SFPE – 367 kNm.

under these fires are 15 kN to 55 kN (2–13%) higher in comparison to 25%, 10%, and 2.5% travelling fires. Under uniform fires and 48% TFM, all beams in the floor are either in compression or tension at the same time, while under smaller travelling fire exposures this is not the case. This is because the total thermal expansion, even if the beams reach much lower peak temperatures exposed to uniform parametric fires, is larger than in beams with very high but localized peak temperatures. For example, beams exposed to the EC short-hot and 2.5% TFM reach the peak temperatures of 367 °C and 578 °C, respectively, but the peak compressive axial force for the EC short-hot fire is 55 kN higher than for the 2.5% TFM.

In addition, yielding of the first beams occurs sooner for more uniform fires, i.e., in the first 10–20 min of fire exposure. For the 25% TFM, 10% TFM, and 2.5% TFM, yielding only takes place at approximately 24 min, 42 min, and 148 min, respectively. After yielding, the compressive axial forces reduce and beams go into tension, initiating the catenary action. Stresses are redistributed to the stiffer surrounding structure, and the rate of increase in deflections reduces. Once the ultimate tensile strength is reached for the 25% TFM and the EC LC, the tensile forces begin to dissipate (see Fig. 9). However, after the average temperature of the heated beam begins to reduce (i.e. cooling begins), the beam regains some of its stiffness, and contraction leads to the increase in tensile forces again until the end of the fire. For other fires, similar behaviour is observed, but the beams do not reach their ultimate tensile strength before cooling because their peak temperatures are lower. As the cooling begins, a small recovery in beam mid-span deflections takes place between 4 and 80 mm. The highest tensile axial forces during the fire develop for the smallest travelling fire sizes.

3.4.3. Bending moments

Peak mid-span bending moments for all fire scenarios are in the same range up to 160 kNm. Larger bending moments tend to develop first in internal Bay 2 for the travelling fire scenarios, and internal Bay 3 for the EC parametric fires. General trends in bending moments follow the behaviour of axial forces. Axial forces increase bending moments and in turn deflections due to the P-Δ effect [43]. The distribution of bending moments in columns at different times of fire exposure is illustrated in Fig. 11. It shows that, as identified previously in Section 3.2, there is little effect on bending moments in columns during the fire higher than two floors above the fire floor, that is, in Floors 5 and higher for a fire occurring on Floor 2. For travelling fire scenarios, as the fire travels through the compartment, a reversal of column bending moments takes place. This is not the case for the uniform fire scenarios. The highest bending moments for both the uniform and travelling fire scenarios develop in the internal columns.

3.4.4. Utilization

The comparison of the evolution in utilization at the mid-point of the heated beams on Floor 2 is shown in Fig. 12. Utilization of the beams was calculated based on the plastic axial load and moment (P-M) interaction curves and axial forces and bending moments at every time step. P-M curves were calculated according to Garlock and Quiel [46]. For further details and more in-depth analyses of the results in terms of frame utilization and failure the reader is referred to [47]. Fig. 12 shows that irrespective of the fire scenario maximum utilization reached at the mid-span of the beams is within a similar range between approx. 94 and 97%. However, the members that reach this level of utilization are

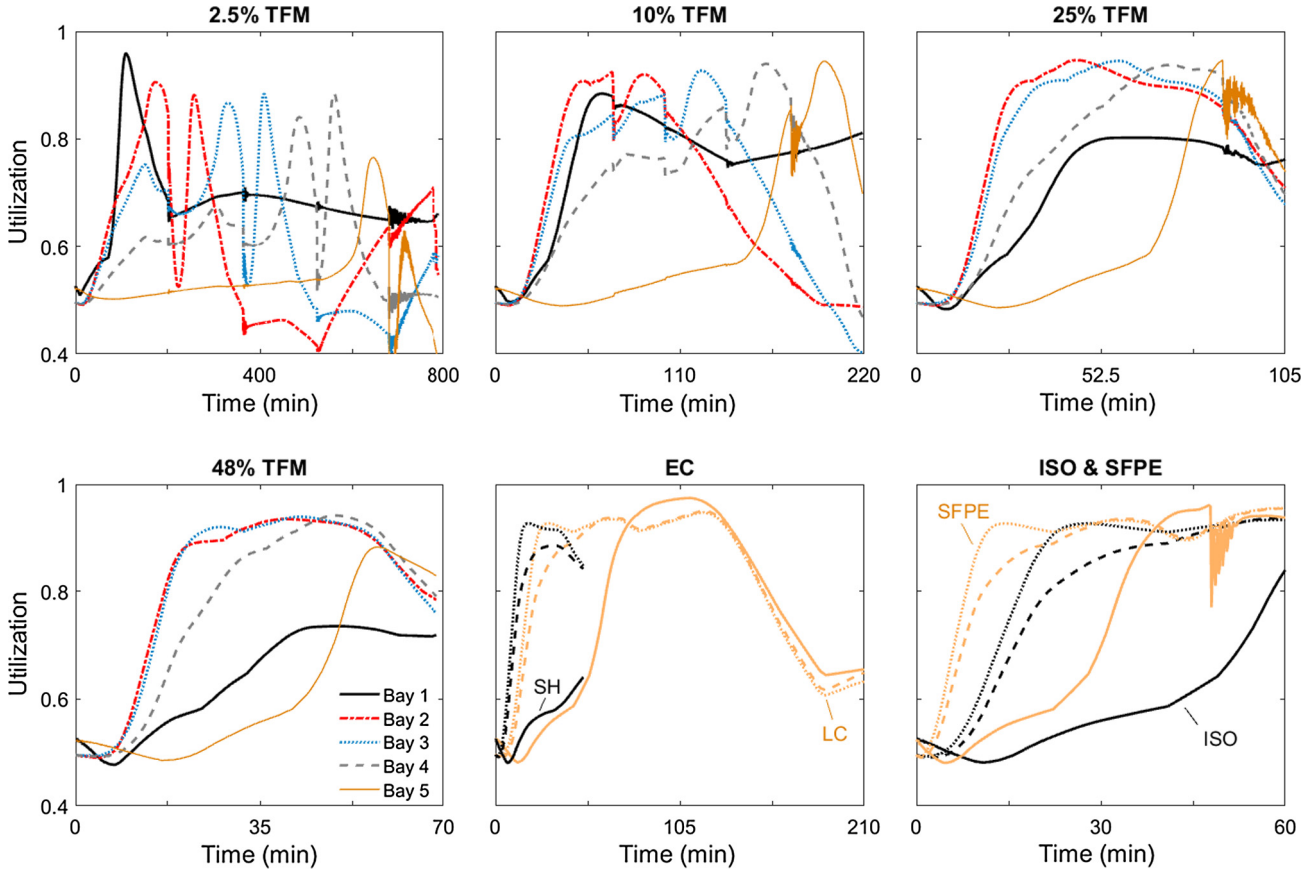


Fig. 12. Evolution in time of utilization at mid-span of the beams in Floor 2; Fire occurs on Floor 2.

different. For uniform fire scenarios initially the internal bays are under higher utilization, but towards the end of the fire the end beams (i.e. Bay 1 and Bay 5) have a higher utilization. For travelling fire scenarios the location of the beam with the highest utilization varies with the fire size. For small TFM scenarios (5% and 10%) end beams reach higher utilization while for large travelling fires (25% and 48%) higher utilization develops in internal beams (Bay 2 and Bay 3). In addition, unlike under uniform and large travelling fires, beams in internal bays (Bays 2–4) subjected to small travelling fires (2.5% and 10%) experience high oscillations in the level of utilization with time. The oscillation amplitudes are approximately in the ranges of 2–38% and 4–16% for 2.5% TFM and 5% TFM, respectively.

Small irregular oscillations of bending moments, axial forces, and lateral displacements for 2.5% and 10% travelling fires can also be seen in Fig. 9. Their approximate amplitudes are 15 kNm, 12–82 kN, and 4 mm, respectively. This is due to the reduction and then increase in tensile force identified previously and due to the spread of fire from one bay to another. The fire length for these cases is shorter than the bay length. Therefore, restrained thermal expansion only takes place at one bay at a time. For 2.5% TFM in Bay 3, the axial forces first increase due to the development of compressive forces in Bay 2. Once the beam in Bay 2 yields, the axial force decreases in Bay 3 as well. Then the near-field travels to Bay 3, and the axial forces begin to increase again due to restrained thermal expansion now occurring in this Bay. After the beam in Bay 3 yields, the compressive axial forces dissipate and the beam goes into tension. In the same manner, the beam in Bay 4 responds to changes in axial forces in the other bays, leading to oscillations in axial force and in turn bending moments. This is because, in

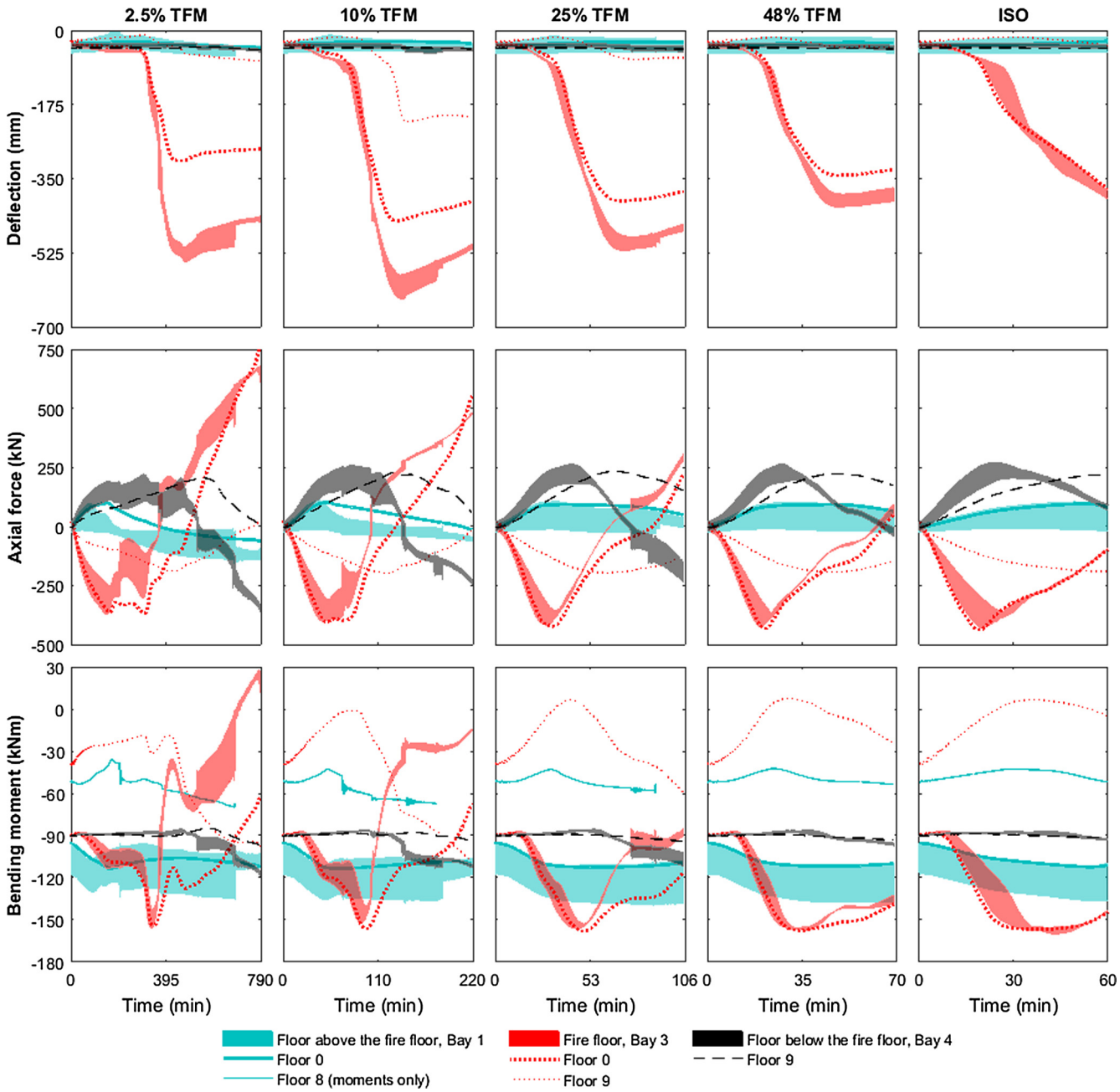


Fig. A.1. Variation of displacement, axial force and bending moment development in beams exposed to different fire scenarios. Numbers for each of the lines indicate results when fire occurs on ground Floor (0), Floor 8 (moments only), and top Floor (9).

these fire scenarios, the beams yield before the fire travels to the other bays. Cyclic axial forces, as already identified in the introduction, can also be observed in the work by Bailey et al. [12]. Such cyclic loading could have an influence on further material degradation. No cyclic forces are present for uniform fires or large fire size TFM exposures.

4. Conclusions

Current design standards assume uniform temperature conditions in the compartment, while in reality, fires in large enclosures travel, resulting in highly non-uniform temperature distributions. In this study, the structural response of a generic protected steel frame exposed to travelling fires and uniform fires defined in the standards (i.e., the Eurocode and SFPE standards) on different floors of the building has been investigated. Travelling fire scenarios, Eurocode parametric curves, the ISO standard fire and the SFPE constant compartment temperature fire scenarios have been considered.

Results indicate that when different floors of the frame (one at a time) are subjected to the same fire exposure for both travelling and uniform fires, the development trends of displacements and stresses in beams are similar, except for fires occurring on the ground floor and the top floor. Higher displacements and lower axial forces develop within beams in the upper floors, where column section reduces in size, due to the reduced level of axial restraint. However, beams subjected to fires on the bottom floors yield only up to 15 min later compared to the fires on the top floors of the frame. The development of axial forces and bending moments in the floors more than 2 floors away from the fire floor is found to be negligible. This indicates that modelling of five floors of the structure (with the fire in the middle floor) could be sufficient to capture the correct structural behaviour and stress redistribution. However, further studies on other types of structures need to be carried out to confirm this.

The rate and magnitude of the highest beam mid-span deflections depend mainly on the fire duration and not the fire type (i.e., TFM or uniform fire). Short and hot fires result in faster development of deflections, while long and cool fires result in larger peak deflections. The same observations were made by Lamont et al. [30]. On the other hand, the locations where these peak deflections occur are different for TFM and uniform fire scenarios. For travelling fires and uniform fires, the highest deflections initially develop in the end bays and central bay, respectively. Also, the displacements under travelling fire exposures are more irregular.

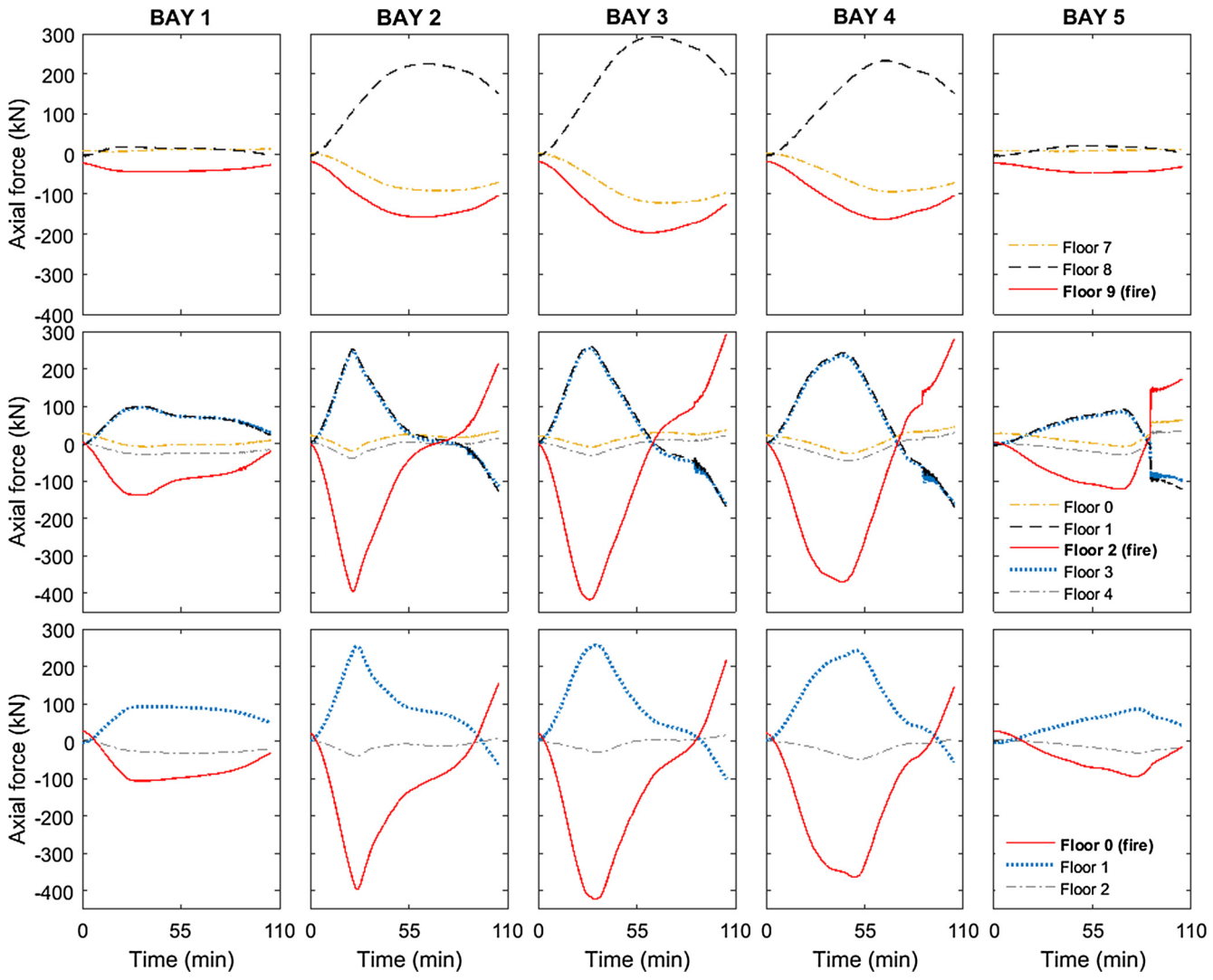


Fig. A.2. Variation of axial forces within different bays for a frame subjected to a 25% travelling fire on Floor 0 (bottom), Floor 2 (middle), and Floor 9 (top). Bays 1 and 5 are the end bays and Bays 2–4 are the internal bays of the frame.

In general, uniform fire scenarios are found to result in 15–55 kN (2–13%) larger compressive axial forces in comparison to travelling fires, while peak bending moments are in the similar range for both travelling fires and uniform fires. When the frame is subjected to travelling fire scenarios smaller than the width of the bay, irregular oscillations of axial forces, bending moments, lateral displacements and thus member utilization (in the range of 2–38%) are observed. This is not the case when the frame is exposed to uniform fires or large size travelling fires. The oscillations in forces and moments could affect material strength.

The assessed case study indicates that travelling fires and uniform fires may trigger substantially different structural responses for the same structure which may be important in the structural design and selection of the critical members. While uniform fires might lead to higher axial forces, travelling fires might result in larger displacements at different times and locations.

Acknowledgements

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Appendix A.

In Sections 3.2 and 3.3, the effect of location of the fire floor in compartment on the structural response of the frame are illustrated for selected fire sizes Figs. 7 and 8. In this appendix, comparison of the results for all fire scenarios investigated in this chapter is presented. Figs. A.1 and A.2 show variation of displacement, axial force, and bending moment in beams exposed to fire, and variation of axial forces within different bays of the frame, respectively. In Fig. A.1, The high variation present in the development of axial forces and bending moments towards the end of the 25% TFM (i.e. when the beam goes into tension) are because of the oscillations from the explicit dynamic simulation in LS-DYNA.

In addition, illustration of the bending moment distribution in the heated beams for all fire scenarios investigated is provided, Fig. A.3. Uneven bending moment distributions at later stages of the fire indicate the formation of plastic hinges and the redistribution of stresses. Plastic hinges during the fire form in the EC, LC, and SFPE fires, and in all travelling fire scenarios except for the 48% TFM.

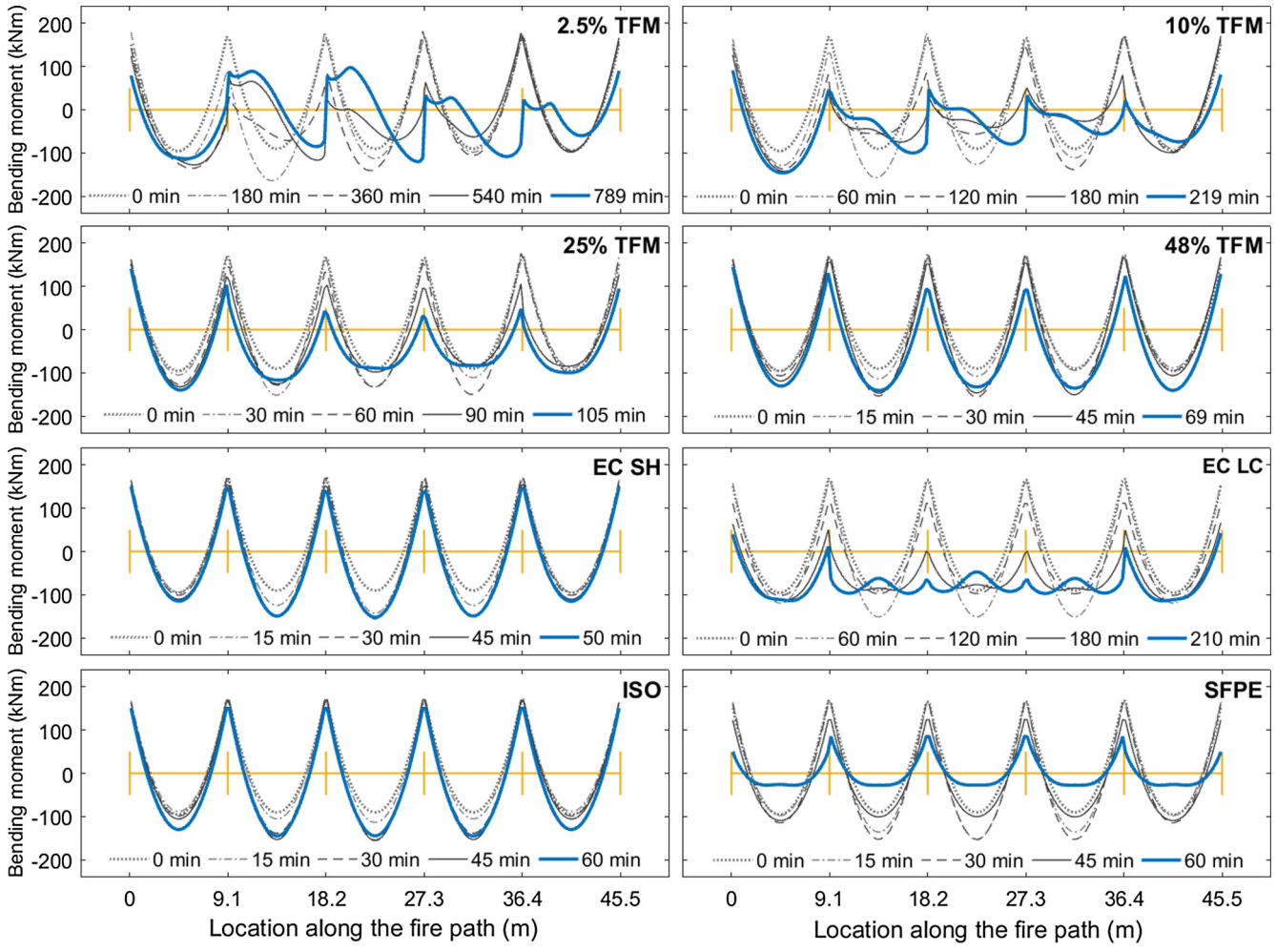


Fig. A.3. Bending moment distribution in the heated beams at different times of fire exposure for all fire scenarios investigated. Fire occurs on Floor 2.

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