Computational analysis of thermal and structural failure criteria of a multi-storey steel frame exposed to fire

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A B S T R A C T

Structural fire design, until recently, has only assumed uniform fires inside the compartment, and the assessment of structural failure has been often based on a critical temperature criterion. While this criterion, to some extent, may be able to indicate the temperature at which the structural element is near to failure, it is based on standard fire tests and, therefore, its validity is limited to individual members exposed to uniform temperatures. It is unclear how representative a critical temperature criterion is of structural failure in the case of multi-story structures, particularly in the case of non-uniform fires such as travelling fires. Therefore, the aim of this study is to assess the validity of the critical temperature criterion for structures exposed to non-uniform fires and compare it to uniform fires. A generic 10-storey steel framed building is modelled using the finite element software LS-DYNA. In total, 117 different scenarios are investigated to cover a wide range of conditions of interest for design of modern steel buildings, varying the fire exposure (travelling fires, Eurocode parametric fires, ISO-834 standard fire, and SFPE standard), floor where the fire is burning, beam section size, and applied fire protection to the beams. For the different fire exposures considered, the analysis predicts structural failure at different times, in different locations and floors, and different failure mechanisms. Moreover, it is shown that there is no single worst case fire scenario: different fires can lead to failure in different structural ways. The comparison of the various structural and thermal failure criteria (ultimate strain, utilization, mid-span deflection, and critical temperature) show that there is no consistency between them, revealing a far more complex problem than reported in the literature. Lastly, this work has illustrated that the critical temperature criterion does not predict accurately the structural failure in time, space or failure mode of steel structures subjected to both uniform and non-uniform fires. Structural failure can only be predicted by advanced structural analysis, and, therefore, heat transfer analysis alone is not sufficient for design. Nevertheless, it was shown that the use of the critical temperature leads to conservative results for simple steel structures. For the sake of comprehensive design, a range of different fire scenarios, including both uniform and non-uniform, should be part of the analysis such that all likely structural responses and failure modes can be considered.

1. Introduction

Structural fire design, until recently, has been based on the assumption of uniform fires in the compartment (i.e. standard fire, the Eurocode parametric time-temperature curves or another post-flashover fire methodology). While this assumption may be suitable for small enclosures, fires in large, open-plan compartments, representative of modern architecture, have been observed to travel, resulting in highly non-uniform, transient temperature distributions within the enclosure \cite{1}. Such fires have been observed most recently during the recent fire at the multi-storey, composite Plasco building in Tehran (Jan 2017), which ultimately collapsed. In order to account for travelling fires, a non-predictive design concept called the Travelling Fires Methodology (TFM) \cite{1–3} has been developed.

The majority of modern buildings have layouts which fall outside the limitations of current prescriptive standards and are more likely to experience travelling fires. Therefore, there has been a significant increase in industry in the performance based design of such buildings, by employing both travelling fires and uniform fires in a probabilistic approach \cite{4–6}. Due to the stochastic nature of fire, a probabilistic approach allows a better understanding of the overall building performance, e.g. to determine the fire resistance period, or the design fires for a building that correspond to its individual target reliability. When carrying out probabilistic assessment, there is often the need to use a single structural failure criterion (intensity measure) which allows multiple simulations as part of the same analysis. The outcome of the assessment would therefore be sensitive to the assumptions regarding the structural failure criterion. Currently, there is no widely accepted, single structural failure criterion. Traditionally, structural response and failure have been assessed in the terms of critical member temperature, maximum displacement or rate of deflection, and exceeded member load-bearing capacity (i.e. collapse). The latter two criteria would require a mechanical response assessment using advanced computational methods (i.e. FEM) and therefore are not feasible for probabilistic
assessments in a design context, due to the computational time and the high number of simulations required. Future improvements in the efficiency of probabilistic methodologies that reduced the number of samples needed, and increased computing power, could make such an approach feasible. However, most of the probabilistic methods used in design for assessing structural response to fire are currently based on the critical member temperature failure criterion [4–7].

It is commonly believed that for regular steel framed buildings without any unusual characteristics (such as those noted in [8,9]) structural fire design based on a critical member temperature failure criterion is conservative. However, even in the early stages of the development of the standard fire tests (e.g. ASTM E119 in the USA) there has been a lot of disagreement on the critical temperature for steel members, because it is significantly affected by the restraint conditions and load redistribution [10]. Since then, Eurocode [11] and other proposals [12,13] have been introduced in the literature to predict critical member temperatures, based on the utilization of the members at ambient temperature. While these may be able, to some extent, to indicate the temperature at which the structure is near to collapse, they are limited to individual members with longitudinally uniform temperatures. In addition, the link between the critical member temperature, based on load-bearing capacity, and other failure criteria, and which of these criteria is more conservative, is unclear. It should be noted that the recently developed ASCE/SEI 7-16 Appendix E does not accept the use of the critical temperature as a performance indicator of structural failure when undertaking a performance based assessment, and obligates the designer to analyse the structural response due to the thermal demand without exception.

Skowronski [14], in his work on heated steel beams, has found that critical member temperature, based on load bearing capacity and deformation limit state, can be up to 30% different, and that depending on the beam size, load and heating rate either of them can be more critical. This difference could likely be even higher for structures subjected to non-uniform fire exposures, such as travelling fires, due to non-uniform heating and longer fire durations. In the work on the thermal response of structures subjected to travelling fires [2,15,16], it has been found that, in general, travelling fires are likely to result in higher peak member temperatures in comparison to uniform fire scenarios (e.g. Eurocode parametric temperature-time curves [17]).

In terms of structural response, recent work has shown that both travelling fires and uniform fires can represent a worst case scenario depending on the structural metric examined (e.g. deflections, plastic strains, axial forces, etc.) since they introduce a different range of behaviours for the same building [15,18–22]. However, only in a few of the latter studies [15,20] the thermal response and critical member temperatures were analysed and/or compared to other failure criteria.

Law et al. [15] investigated the structural response of a concrete frame subjected to travelling fires and uniform fires. Peak temperatures, rebar strains, and deflections were assessed as the failure criteria, and were compared for the different fire exposures. The results indicated that the temperature and deflection criteria were more critical, i.e. indicated that the structure was closer to failure [15] compared to the strain criterion. Structural distress trends with varying travelling fire sizes were similar for all the failure criteria. Rezvani and Ronagh [20] investigated the structural response of an unprotected, moment resisting steel frame, subjected to travelling fires, and reported the critical times and the corresponding temperatures for column buckling and beam deflection limit states. Larger travelling fire sizes resulted in shorter times to failure and larger corresponding member temperatures.

In general, column buckling was more critical in terms of potential failure than the beam mid-span deflection limit state. However, it should be noted that the structure used as part of the aforementioned study was unprotected, and therefore the outcomes cannot be generalized to protected steel frames. In a study on a tall steel moment resisting frame subjected to post-earthquake uniform fires in small localised compartments (i.e. one bay wide only), Khorasani et al. [23] considered four different limit states. The limit states include formation of plastic hinges, tension force, rate of deflection and deflection. However, the authors [23] did not draw any conclusions on the most critical failure criteria. Different limit states were reached depending on the scenario, that is, location of the fire compartment and maximum fire temperature.

Critical member temperature failure criterion is used widely in the prescriptive design and probabilistic analyses to evaluate the structural performance of a building during a fire. Previously identified studies [15,20] are the most recent and limited available research where critical member temperatures were compared to other structural failure criteria. However, these studies are limited to concrete [15] and unprotected steel framed [20] structures. In addition, these papers did not aim to assess the critical temperature criterion and investigated a relatively small number of different fire scenarios. Therefore, it is still unclear how representative such a criterion is of the actual structural response. This is particularly the case in the structures, where non-uniform fires such as travelling fires are likely to occur.

Therefore, the aim of this paper is to fill these gaps in knowledge and study the structural response of a multi-storey steel frame subjected to both travelling fires and uniform fires and to assess the critical member temperature failure criterion in respect to other structural metrics. We investigate the effects of fire exposures, location of the fire floor in the frame, level of applied fire protection, and beam section size. Load-bearing capacity, beam mid-span deflection, critical member temperature, and member utilization limit states are considered as possible failure criteria. To the best of the authors’ knowledge, a similar study assessing the failure criteria of steel frames exposed to non-uniform fires has not been reported in literature.

2. Finite element model

Full-scale testing of real structures is complex, expensive, and time consuming. This is especially the case for structures with large

| Table 1 |
|---|---|---|---|
| **Details of the scenarios considered in the study.** |
| Reference | Equivalent standard fire resistance | Beam section size | Fire floors | Fire exposures |
| --- | --- | --- | --- | --- |
| B60 C120 | 60 min | 120 min | W14 × 22 | 0, 1, 2, 3, 4, 5, 6, 7, 8, & 9 | 2.5%, 5%, 10%, 25%, & 48% TFM, EC SH, EC LC, ISO & SFPE |
| B60 C60 | 60 min | 60 min | W14 × 22 | 7 |
| B120 C120 | 120 min | 120 min | W14 × 22 | 7 |
| B60 C120 LC | 60 min | 120 min | W16 × 26 | 7 |

* Equivalent standard fire resistance refers to the different thicknesses of fire protection based on the limiting steel temperature of 550 °C under a standard fire exposure.
compartments. There has only been a limited number of full-scale tests on real buildings carried out worldwide (e.g. Cardington tests). As a result, computational tools are commonly used to assess the structural response of complex buildings under fire conditions. The primary objectives of this study are to analyse the general trends and to compare the outcomes of the model qualitatively for many fire scenarios (i.e. 117, see Table 1). Therefore, for simplicity of the model and computational time, the analyses presented in this paper has been based on the 2D model of a simple steel frame using a finite element software LS-DYNA. Such representation ignores composite action effects between steel beams and concrete slab, and concrete membrane tensile action.

2.1. Multi-storey frame

The multi-storey steel frame considered in this analysis is based on the moment resistant frame published by NIST [25]. 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. The frame is quite regular and does not present any unusual structural characteristics that would make it particularly sensitive to thermal expansion. It is designed according to the American Society of Civil Engineers (ASCE 7-02) standard. The plan layout and the 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 particular internal frame is selected 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 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 a 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).

The steel beams are originally designed, as reported in [25], to support a lightweight concrete floor slab, and act in a composite action through shear studs. However, in this study, due to the 2D representation of the building, the composite action between the beams and the concrete floor slab is not taken into account and, therefore, the structure is considered as non-composite (such as a pre-cast plank construction). This also allows simplifying the study by focusing on the criteria related to the steel members only. However, despite the fact that the concrete slab is not accounted for in the mechanical assessment, the effect of the concrete slab is considered in heat transfer analysis. In different studies on a generic composite structure [18], a high-rise, moment-resisting steel frame [26], and tall composite buildings with a concrete core and perimeter long-span steel beams and trusses [22,27], it was found that a 2D model using beam elements gives a good representation of the global structural response to fire when compared to the 3D model using beam and shell elements. The 2D model in [18] was observed to underestimate the deflections and residual bending moment during cooling, but the overall trends in behaviour were found to be the same. Also, in the latter study [18], in the 2D beam model, load due to the concrete slab was not taken into account, which could have resulted in lower deflections compared to the 3D model.

For the cases investigated in this study, if composite action between the steel beam and the concrete slab was to be considered in the model, it would likely result in lower beam deflections and column horizontal displacements due to the stiffer overall response of the concrete slab. It could also lead to lower tensile axial forces during cooling, and thus affect the observations of this study, particularly for the smaller travelling fires, where structural members in different areas of the floor-plate experience heating and cooling simultaneously for long periods during the fire exposure. In general, the 2D models in previously identified studies [18,22,26,27] 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 primary objectives are to analyse the general trends and to compare the outcomes of the model for the different fire scenarios considered.

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/m² (dead) and 4.79 kN/m² (live). For the roof, design loads are 2.68 kN/m² (dead) and 0.96 kN/m² (live) [25]. In this paper, the unfactored design loads were used as the combination for the fire limit state. The load combination at the fire limit state varies from country to country. However, the factors for the permanent (dead) loads are around 1, and for variable (live) loads typically vary between 0.5 and 1 depending on the code and occupancy. The value of 1 was considered in this study for conservatism. It should be noted that, for the structure examined and the failure mechanisms considered (implicitly due to the beam element representation), it is expected that the
applied loads would only affect the time to failure, but not the failure mechanism itself. In addition, the assumption of non-composite structure results in higher utilisation of the beams at ambient temperature (see Section 3.2) from the original design of the building which is expected to primarily affect the time to failure (i.e. reduce it).

2.2. Fire scenarios

The structural response of the frame subjected to travelling fires (TFM) [3] and uniform design fires, such as Eurocode (EC) parametric temperature-time curves [17], the standard fire (ISO) [17,28], and the SFPE constant compartment temperature design fire [29], is investigated. These fire scenarios have been chosen, as identified in the Introduction, to investigate the effects of different fire exposures, i.e. uniform (traditional design fires) and non-uniform (travelling fires representative of fires observed in real fire accidents).

Every fire scenario is applied, one at a time, on every floor of the frame. In addition, for each fire scenario occurring on Floor 7 the level of member protection and the beam section size is varied to assess the effects of the unevenness in fire protection, and the increase in beam section size in the achieved fire resistance of the frame. These effects are presented and discussed in Appendix A. Therefore, in total, 117 scenarios are investigated. All the investigated scenarios of this study are summarised in Table 1. B60/B120 and C60/C120 in ‘Reference’ column of Table 1 refer to the equivalent standard fire resistance of beams and columns, respectively. In UK, the 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 [32]. In this study, beam failure criteria are primarily used. Therefore, a reduced degree of applied fire protection to the beams, in comparison with the columns, is expected to lead to an earlier failure of the beams.

To represent the travelling fire exposure, iTFM [3] is used. iTFM is the most recent version of the Travelling Fires Methodology (TFM), which was developed by [1,2,15]. The key difference with other fire models for structural design is that TFM considers the non-uniform temperature distribution in the compartment and the long fire durations observed in real fire incidents (e.g. 7 h, TU Delft Faculty of Architecture Building fire [30]). An illustration of a travelling fire is shown in Fig. 2. It should be noted that the exposure is idealised for structural fire design purposes, with the aim of removing complexities and simplifying its use, but equally capturing the key phenomena experienced by non-uniform fires. As it is common with all design fires, the actual fire physics is complex to characterise and, therefore, the methodology is not predictive in nature but developed for parametric design purposes. 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 uniformly distributed fuel and, once alight, burns at a constant rate. Thus, the fire size is governed by the fire spread rate. Each floor of the frame in this study is subject to five TFM scenarios: fire sizes of 2.5%, 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 [3], i.e. spread rates of 1 m/min and 19.2 m/min, respectively. TFM sizes of 10% and 25% have been found to be the worst case scenarios in previous studies [2,19]. 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/m² (80th percentile design value for offices) and 500 kW/m² (typical value for densely furnished places) [2], respectively.

Two Eurocode [17] parametric temperature-time curves are considered, representing short-hot (EC SH) and long-cool (EC LC) fire exposures, based on the study by Lamont et al. [31]. 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 [31] it was found that these two parametric fires resulted in different structural behaviour, and for this reason they are included in our study. EC parametric curves were generated assuming the same fuel load density as for travelling fires (570 MJ/m²), and opening factors of 0.176 m⁰.⁵ (short-hot) and 0.044 m⁰.⁵ (long-cool). These opening factors correspond to 100% and 25% glass breakage (assuming a weighted average window height of 2.5 m), respectively.

The correlation representing the standard fire (referred to as the “ISO standard fire” in this paper) is taken from the Eurocode [17]. The standard fire has its origins in the early 20th century, and forms the basis of fire resistance rating and standards worldwide. In addition, the design fire scenario in the SFPE standard [29] is considered. In the SFPE standard, a constant and uniform compartment temperature of 1200 °C is defined until the burnout time. For different opening factors (i.e. depending on the assumed glass breakage) the burnout time can vary from approx. 15 min to 5 h or longer. For this study, the temperature of 1200 °C is kept constant until the initiation of collapse of the frame, to represent the worst case uniform fire exposure. This fire scenario is referred to as “SFPE”. SFPE fire scenario was chosen to represent the worst case uniform fire in the terms of heating rates.

A set of gas temperatures resulting from different scenarios are 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 [33], used to define gas temperatures in the iTFM, is a function of ceiling height.

It should be noted that this paper does not suggest that such a wide range of fire scenarios needs to be considered by designers when undertaking commercial projects. The aim of selecting many fire scenarios in this work is for in-depth study and comparative purposes that can help in defining better design practices.

2.3. Heat transfer

Fig. 2. Illustration of a travelling fire and distribution of gas temperatures in the near-field and far-field [3].
Steel insulation thermal properties are taken as for high density perlite (thermal conductivity $k_i = 0.12$ W/mK, density $\rho_i = 550$ kg/m$^3$, and specific heat $c_i = 1200$ J/kg K) [34]. Heat transfer to the structural members is carried out assuming lumped capacitance for separate parts of the cross-section (i.e. web and flanges) according to [2,34] as shown in Eq. (2).

$$\Delta T_i = \frac{H_p}{A_i \rho_i c_i \left[ (H_p/2) \rho_i c_i / A_i \right]} (T_g - T_i) \Delta t$$

where $T_i$ is the steel temperature (K), $T_g$ is the gas temperature (K), $\rho_i$ is the density of steel (kg/m$^3$), $c_i$ is the temperature dependent specific heat of steel taken from the Eurocode [11] (J/kg K), and $\Delta t$ is the time step (s).

For beams, the effect of the concrete slab is considered by excluding the top surface of the upper flange, which is in contact with the slab, in the calculation of the heated perimeter. Therefore, an adiabatic boundary condition is assumed for the top surface in the upper flange. Additional heat losses from the steel section to the concrete slab that would result in slightly lower temperatures at the top flange were not considered. The convective heat transfer coefficient at the free surface, the density of steel, and the radiative emissivity at the free surface are considered. The convective heat transfer coefficient is calculated at the ceiling for the same location. Previous research on columns subjected to non-uniform temperature distributions has shown that this is a conservative assumption [37].

2.4. LS-DYNA model

The multi-storey steel frame is modelled using the explicit solver of the general purpose finite element software LS-DYNA (Release 7.1.1). The software was originally developed specifically for highly nonlinear and transient dynamic analysis. LS-DYNA is capable of simulating 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 [38]. Previously, LS-DYNA has been used for the analysis of tall structures [39] and structural arrangements, with bi-linear columns [8], subjected to fire. All the parameters for the model presented in this section were chosen based on mesh density and parameter sensitivity convergence studies. Further details and results of the parameter sensitivity studies for the investigated model can be found in the PhD thesis of the first author [58].

The steel beams and columns are modelled using the Hughes-Liu [40] 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. LS-DYNA has been used for the analysis of tall structures [39] and structural arrangements, with bi-linear columns [8], subjected to fire. All the parameters for the model presented in this section were chosen based on mesh density and parameter sensitivity convergence studies. Further details and results of the parameter sensitivity studies for the investigated model can be found in the PhD thesis of the first author [58].

A thermally-sensitive steel material type MAT 202 formulation, based on Eurocode 3 (EN 1993-1-2:2005) [11], is used for both steel beams and columns, with the default temperature-dependent material properties. The material model considers thermal strains, however strain-hardening is not taken into account. Steel with initial yield stress of 345 MPa, Young’s modulus of 210 GPa [42], and Poisson’s ratio of 0.3 [42] is assigned to all members. Both mechanical and gravitational loads are considered. Simulations are carried out using the explicit solver of LS-DYNA, which uses real-time units to solve the equation of motion. Thus, in order to avoid artificial, dynamic oscillations, the mechanical and gravitational 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 is applied using a formulation that 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 time, the temperature development within heated members is scaled by a factor of 100, which was determined to be an

![Fig. 3. Gas temperature histories at mid-span of Bay 2 (top) and right end of Bay 5 (bottom) for the nine 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 to 9 (right).](image-url)
appropriate scaling factor to control the inertia effects, based on the sensitivity analysis. This means that a parametric curve that would last 120 min in physical time would be applied in 1.2 min in terms of simulation time.

All simulations were carried out on a single PC computer with one Intel Core i7 CPU and 8.00 GB of RAM. Depending on the fire scenario investigated, simulation times for a single scenario ranged from approx. 7.5 h up to approx. 7 days and 9.4 h. The total CPU time taken for the simulations of all 117 different fire scenarios is approx. 4.8 months. Once all the simulations were completed, data on the development of axial forces, bending moments, and displacements in the frame were extracted, and are analysed in the following sections.

2.5. Failure criteria

In recent work [24], we have analysed the generic structural response of the same steel frame in terms of the development of stresses and deflections. Even though these metrics give a useful overview of structural behaviour, they provide no indication of how close the frame is to failure for different fire scenarios. Thus, the aim of this study is to extend previous work and assess structural fire performance in terms of different failure criteria and the corresponding critical member temperatures. Critical member temperature, load-bearing capacity, beam mid-span deflection criteria, and overall frame utilization are considered. The former three criteria refer to the local failure of the structural member (which is the basis of prescriptive fire resistance design) and therefore do not necessarily imply a full frame failure, i.e. partial or global collapse.

It should also be noted that the failure criteria examined in this paper are only concerned with the load bearing fire resistance of a structural steel frame, and not with other fire resistance criteria, such as integrity or insulation, that may sometimes govern fire safety design (e.g. around compartmentation provided for life safety or fire-fighting activities). Additionally, designers need to carefully examine which criteria they adopt in conjunction with other elements of construction that may not be represented in a structural model, but may have their own failure criteria or limits, such as cracking of concrete slabs due to excessive strains leading to compartmentation failure, fire protection materials (for example the strain limits of intumescent paint), deflection heads in compartment walls, fire stops in compartment floors, etc. This paper does not address these additional considerations.

2.5.1. Critical member temperature criterion

In the standard fire test (BS 476-43, ISO 834, and ASTM E119-45) of individual members, the critical member temperature criterion is used as a measure of their performance. A structural member would be exposed to a standard fire curve (if a steel member, typically provided with a fire protection material) and the time taken for the member to reach the prescribed failure temperature would be recorded as the rating of the member (e.g. 30 min or 2 h). Standard fire curves used in BS 476-43, American (ASTM E119-45), and International (ISO 834) standards are very similar, with minor differences in severities, especially for longer fire tests [46]. In ASTM E119 the critical member temperature criterion was first introduced in 1964 [10] for steel beams. Previously, it was required that the member did not collapse, which was judged by the furnace operator. Although, the time when the critical member temperature had been reached was considered of known interest [47]. In 1964, the standard required that the average temperature of the steel beam did not exceed 1200 °F (649 °C) and that the maximum temperature at any location did not exceed 1400 °F (760 °C) during the classification period. In 1967 the unrestrained and restrained beam classifications were introduced to account for the fact that restrained beams usually showed a better performance in fire (i.e. based on the evidence of structural failure) in comparison to unrestrained beams [10]. It was also based on theories that restrained beams performed better due to, for example, redistribution of stresses, restraint developed compressive stresses, developed negative moments, composite action, etc. [10]. For the restrained beams, the temperature criterion was left the same, and for unrestrained beams, the requirements for the average and the maximum temperatures at any location were 1000 °F (538 °C) and 1200 °F (649 °C), respectively. These temperatures were chosen based on the assumption that at 1000 °F (538 °C) steel retains approximately half of its original strength [10], and that the tests carried out at the time showed rapid development of deflections at temperatures higher than 1200 °F (649 °C) [48]. In 1988, the critical member temperatures for restrained beams were decreased by 100 °F (38 °C) and have not been changed since. In the BS 476-8:1972 and BS 476-23:1987 standards, in 1972 and 1978, respectively, similar criteria were introduced for the steel beams protected by suspended ceilings. The member average and maximum temperature limits were 550 °C and 650 °C, respectively, for loaded beams, and for the unloaded beams the maximum temperature limit was 400 °C.

In structural fire design, the critical member temperature has been used as one of the limit states since the first attempts in European standardisation for fire in 1983, and is defined as the temperature at which a structure is expected to be at the point of collapse. In the current Eurocode (EN 1993-1-2 [11]), the critical member temperature limit state is based on the value of member utilization at ambient temperature, assuming a uniform temperature distribution. For the range of utilization factors from 0.2 and 0.8, the critical member temperatures are typically between 500 °C and 700 °C. Critical member temperatures used in the ASTM E119 and BS 476 standards fall inside this range as well. Thus, the lower limiting temperature of 550 °C is commonly accepted [36] as the critical member temperature in structural design, and is used for assessment of structural performance. At 550 °C steel maintains only 60% of its ambient temperature strength because of the thermal degradation of its mechanical properties. For simplicity, the same critical member temperature of 550 °C is assumed in our study. It should be noted that other beams such as beam with holes may have lower critical temperatures but these beams are not included as part of this study.

2.5.2. Critical deflection criterion

The first nationwide attempts at the standardisation of structural fire tests in the USA and UK were in the early 20th century [49]. However, these early standards had no prescribed failure criterion, and the collapse (i.e. failure) relied instead on the judgement of the furnace operator. Ryan and Bender [47] and Ryan and Robertson [50] were the first to attempt to define a general criterion for load failure during the standard fire test. The criterion was defined in terms of a limiting mid-span deflection of the beam of \( L^2/800d \) and a rate of deflection of \( L^2/9000d \) over 1 min, where \( L \) and \( d \) are beam length and section depth, respectively [50]. Authors noted that high deflections might not necessarily indicate structural collapse, and suggested that both previously identified criteria (\( L^2/800d \) and \( L^2/9000d \) over 1 min) need to be exceeded to give a crude practical indication of a load failure. The proposed criteria were empirical and based on the equation for the maximum deflection of the beam (Eq. (3)).

\[
\Delta_{max} = k \left( \frac{L^2}{d} \right) (e_1 - e_2)
\]

where \( k \) is the numerical constant depending on the type of support and loading, and \( e_1 - e_2 \) is the difference between the strains in the planes of the two surfaces of the specimens separated by the distance \( d \).

Ryan and Robertson’s criterion is still widely used today in standard fire tests (BS 476-43, ISO 834, ASTM E119-45) and in the assessment of the structural fire performance of beams. Nevertheless, the critical deflection has been increased to \( L^2/400d \) in ISO 834-1 [44] and ASTM E119-16 [45] and replaced by \( L/20 \) in BS 476-10:2009 [43]. In addition, the limiting rate of deflection of \( L^2/9000d \) over 1 min in BS 476-10:2009 [43] is only considered after the mid-span deflection of the beam of \( L/30 \) has been exceeded. According to Law et al. [15] and
Lamont et al. [51], the deflection criterion of L/20 originates from the standard test furnace, and was chosen to ensure that the furnace was not damaged during testing. In structural fire design of steel structures, the deflection criteria (L/30) has only been included in European recommendations for fire safety of steel structures in 1983 [52]. Furthermore, Dumont et al. [53] have recently noted that European standards for testing (EN 1363-1 [54]) and classification (EN 13501-2 [55]) of structural elements in fire use different interpretations of the same deflection criteria (i.e. critical deflection of $L^2/400d$ and limiting rate of deflection of $L^2/9000d$) to assess the loadbearing performance. According to EN 1363-1 failure occurs when only one of the deflection criteria have been exceeded, while according to EN 13501-2 failure occurs when both criteria are exceeded. Dumont et al. [53] investigated results from 46 fire resistance tests and concluded that this can have practical implications due to differences in recorded failure times and, as a result, different assigned fire resistance ratings for the same elements.

Since the development of these deflection criteria and at present, it is known that large beam deflections do not necessarily indicate the onset of structural collapse as experienced in the Broadgate fire in London and the Cardington tests in the UK [56]. However, large deflections may be important for the integrity of fire protection and fire compartmentation (e.g. fire walls can be breached due to gaps as a result of large deflections). In BS 5950-8 a deflection limit state of L/100 existed for beams which have fire resisting walls under them. In our study the maximum deflection criteria of L/20, $L^2/400d$ and the limiting rate of deflection of $L^2/9000d$ over 1 min are chosen as performance indicators.

### 2.5.3. Ultimate strain

The critical member temperature and maximum deflection criteria are simple measures to assess the structural performance in fire, particularly in the standard fire test. However, they do not give an indication or relation to the actual load-bearing capacity of the structure, and the occurrence of collapse is uncertain. Structural member failure occurs at a member level (i.e. neglecting localized failures) once the stresses that develop within the member due to fire exceed its load-bearing capacity. Therefore, for the purposes of this study, as one of the performance indicators, structural failure is based on the calculated failure of structural members, i.e. the ultimate strain, during the computational LS-DYNA analysis. Failure in the LS-DYNA analysis is defined according to the Eurocode EN 1993-1-2 material model, and takes place in any member when the ultimate strain of 0.2 for steel is exceeded. Failure of the first element is considered as an indication of collapse (as commonly assumed in prescriptive guidance). Progressive collapse of the frame was not explicitly considered as part of the analyses conducted.

#### 2.5.4. Utilization

Utilization is considered here as the ratio of the load carried by the member and its load carrying capacity. Unlike critical member temperature and deflection criteria, it gives an indication of how close the member is to structural failure. In EN 1993-1-2 [11], member utilization is used to determine the critical temperature of a member. However, it is based on uniform temperature distribution and the pre-applied loading at the accidental fire limit state. The thermally induced forces that develop during a fire, due to non-uniform temperature distribution, restrained thermal expansion, and thermal bowing or other effects, are not taken into account. The utilization analysis used as a performance indicator in this study is based on the work by Garlock and Quiel [57], where the yield capacity is represented by plastic axial load and moment (P-M) interaction curves for steel sections with non-uniform through-depth temperature distributions. The P-M curves are determined by moving the neutral axis (i.e. axis of zero strain) through the depth of the section and integrating the stresses at each location to find the axial load $P$ and bending moment $M$ according to Eqs. (4) and (5), respectively [57].

$$P = \int_A \sigma_t \, dA$$

(4)

$$M = \int_A \sigma_t \, z \, dA$$

(5)

where $\sigma_t$ is the yield stress as a function of the temperature distribution, $A$ is the cross-section area, and $z$ is the distance to the reference axis. LS-DYNA uses a geometric centroid as the reference axis for the calculation. Thus, for an easy comparison with computational results, the geometric centroid is used as the reference axis to calculate the bending moment for the P-M curves in this study as well. Temperature dependent, steel yield strength reduction factors are taken from the Eurocode [11] as in the LS-DYNA analysis. The same sign convention as in [57] is used. Positive bending moment represents tension in the top flange and positive axial force represents compression.

Illustration of P-M curves at ambient temperature and during the 25% travelling fire occurring on Floor 8 is shown in Fig. 4. P-M curves are shown for the beam mid-span in Bay 3 and the element that failed first, i.e. Bay 5 beam left end, in the terms of ultimate strain criteria. In this study, based on these curves, and the axial force and bending moment output from the computational analysis using LS-DYNA, the utilization of each element is calculated at every time step.
3. Results and discussion

3.1. Peak temperatures

A comparison of the peak temperatures that develop within the heated steel beams for different fire exposures (travelling fires and uniform fires) and the time to reach them are shown in Fig. 5. For the ISO (standard) and SFPE (constant compartment temperature) fires, the temperature is shown at the corresponding beam fire resistance time, i.e. at 60 min (B60) and at 120 min (B120). The critical temperature used in the design of this fire protection was 550°C. However, beams B120 under standard fire exposure, after 120 mins, only reach a temperature of 400°C. This is because different relationships were used for the calculations of the fire protection thickness and the thermal response of the beam. For the former, a very simple empirical equation was used (Eq. (1)) that assumed uniform heating of the beam, while in the heat transfer calculations (Eq. (2)) for the structural analysis separate temperatures were calculated for the flanges and the web. This is a conservative approach, and it is not uncommon for fire protection providers to overdesign fire protection materials based on a simplified assessment rather than a detailed thermal response assessment. It should be noted that the use of different relationships for the fire protection thickness and heat transfer calculations does not affect the results on the structural response, as the main aim of this work is to compare the structural response and failure criteria between the different cases investigated in this study.

As could be expected, for both beams B60 and B120, the SFPE fire results in higher temperatures than the ISO fire by 190°C and 110°C, at 60 min and 120 min, respectively. In general, the results indicate that between fires with fire decay period (i.e. TFM (travelling fires) and EC (Eurocode parametric fires)) 2.5–10% travelling fires lead to the highest peak temperatures of up to 700°C (B60) and 400°C (B120). The same observations were made in other studies on the thermal response of steel [2] and concrete [2,15] structures. Small TFM sizes lead to higher temperatures mainly because of longer heating durations, compared to other fires [3]. The lowest temperatures develop within beams exposed to EC SH (Eurocode short-hot) fire. They are 367°C and 142°C for B60 and B120, respectively. On the other hand, the time taken to reach the peak temperatures is shorter for large TFM scenarios and uniform fires (58–105 min) than for small TFM fires (up to 758 min). The time to reach critical member temperature of 550°C is also shorter for larger TFM fires (e.g. 48% TFM) than for the smaller ones (2.5% TFM) [2].

3.2. Utilization

A plot of the evolution of utilization of the frame with time for the 10% TFM (travelling), EC SH (Eurocode short-hot), and ISO (standard) fires is shown in Fig. 6. For the beams on the floor subjected to fire, utilization increases with time. Beam elements in the proximity of the connections to the columns experience the highest utilization because of the development of high bending moments, as connections in the analysis are assumed fixed. The results also indicate significant increasing utilization in the beams in the floors directly above and below the fire floor, although they are not heated. In the floors further away from the fire there is no significant change in beam utilization. This is due to the development of high axial forces in the floors adjacent to the fire floor. Similarly, the results show significant redistribution of the forces, i.e. utilization, in the columns up to 2 floors away from the fire floor. The increase in utilization of internal columns connected to Bay 3 on the fire floor is smaller than for the end columns for the 10% TFM and ISO fires. For example, for the 10% TFM, the utilization in internal columns increases by 37% (from 17 to 54%) while in the end columns it increases by up to 59% (from 27 to 86%). For the ISO fire the increase in the utilization of internal columns is only 4%. The axial load on the
columns during the fire does not increase significantly and their changes in utilization are mainly affected by the development of bending moments.

A comparison of the change in utilization of the beams with time for the different fire scenarios is shown in Fig. 7. It shows, as identified in the previous paragraph, that for all fire scenarios, the beam locations that are close to connections are under the highest utilization. However, during cooling, the utilization of the connections decreases, and the utilization at the beam mid-span increases. For small TFM scenarios (i.e. 5% and 10% TFM), the numbers of the beams under high utilization are smaller and more localized close to the location of the near-field. For uniform fires, the level of utilization in different bays at the same time is similar. As the travelling fire size increases, the patterns of utilization along the fire path approach that of the uniform fires. In general, for all fire scenarios, different sections of the beams (e.g. connections and mid-span) or columns are under significantly different levels of utilization as a result of the distribution of bending moments and axial forces. This is illustrated in Figs. 6 and 7.

A change in the typical range of utilizations along the beam with time and the 20th, 40th, 60th, and 80th percentiles are shown in Fig. 8. At ambient temperature, 80% of the beams is at a utilization of 50% or lower. For the case illustrated, the 80th percentile utilization increases up to 90% in Bay 4 as the fire travels along the frame. In order to allow an easy comparison of all fire scenarios for each member in the frame (i.e. beams and columns), the mean and the maximum were taken as the representative values of its utilization. The maximum value indicates how close the member is to yielding. Cumulative density functions (CDF) were then plotted to give an indication of the overall frame utilization and performance in fire. CDF illustrate what percentage of each member in the frame is under the utilization equal to or lower than the specified value. Fig. 9 provides an illustration of the change of CDF with time for a frame exposed to the 48% TFM on Floor 2. The lower and upper bounds show CDF of the average and peak (i.e. maximum) utilization, respectively, of different members. It should be noted that even at ambient temperature peak utilization of the frame is relatively high This is due to the assumptions about the applied loads and the non-presence of the concrete slab (see Section 2.1). That is, the beams were originally designed as part of the composite structure. Thus, they have relatively small sections for the assumed applied loads and non-composite behaviour in the investigated model resulting in a relatively higher utilisation than normal. Approximately 30% of the members have a peak utilization of 90%. 10% of the members with a peak utilization between 40 and 50% are the floor 9 beams. These beams are subjected to smaller, permanent, and variable loads than beams on the
floors below. Normally, with time and increasing temperature, the utilization of the frame increases, as illustrated in Fig. 9. The peak utilization factor values that are close to 1 indicate the number of the members in the frame that have yielded or are close to yielding.

Comparisons of the CDF of the maximum at any time, mean, and peak utilization of the members (as shown in Fig. 9) in the frame for different fire scenarios and fire floors are shown in Figs. 10 and 11. The fire durations considered in Figs. 10 and 11 are from the beginning until the end of the fire (cases with no ultimate strain failure) or until the ultimate strain failure has been reached. Fig. 10 shows that the frame, subjected to any fire scenario on floor 9, is under the lowest level of utilization, in comparison to fires occurring on other floors. This is because the beams at this location, at ambient temperature, are under the lowest utilization, as identified previously, and have a lower level of restraint to expansion. Axial forces that develop on fire floor 9 are up to 240 kN lower than on other fire floors [24]. Fires on floor 9 mainly affect the minimum utilization factor of the frame. This factor decreases as the number of the fire floor decreases. Similarly, the upper limit of the average frame utilization is highest for fires on Floor 1. Even though the CDF functions appear to be similar for fires occurring in other floors, especially Floors 1 to 8, the highest peak utilizations and the largest number of yielded members occur for the frame subjected to...
fires on Floors 6 to 8. For ISO and SFPE (constant compartment temperature) fires Fire floor 7 results in the largest utilization. 80% of the members are at the normalised utilization of 0.63 while for the cases with other fire floors it is approximately 0.5. Therefore, these results indicate that the frame subjected to fires on Floors 6 to 8 is likely to be closer to the onset of collapse. On these floors beams are connected to columns that have the lowest load-bearing capacity, i.e. the smallest sections.

Fig. 11 shows the same results as Fig. 10, but gives a clearer comparison of the effect of fire scenario on the CDF of frame utilization. For all fire floor cases, the uniform fire scenarios (EC LC, ISO, and SFPE) that have a long duration result in the highest average frame utilization. They are followed by the smallest travelling fire scenarios (i.e. 2.5% TFM). With increasing size and, thus, duration of the TFM fires, the highest average frame utilization reduces, and is lowest for the 48% TFM and EC SH fires. This is likely because uniform, large duration fire scenarios result in higher temperatures, and therefore lead to greater overall thermal expansion of the fire floor, leading to higher stresses in members compared with shorter fires. For travelling fire scenarios, it is likely a function of the peak and average temperatures that develop within the members, and the level of restraint from the adjacent members as the fire travels along the fire path. In addition, the 2.5%–10% TFM fires result in high tensile axial forces on fire floor even during the fire exposure [24]. However, the results indicate that for the TFM fires, in comparison with the uniform fires, a higher percentage of members is under higher peak utilization (the CDF curve is shifted towards right which indicates higher utilizations). Similar observations were made by Law [19] on the utilization of concrete structures subjected to travelling fires and uniform fires. He compared the
performance of the frame subjected to ISO, EC SH and LC fires and to 2.5\%, 5\%, 10\%, 25\%, 50\%, and 100\% travelling fires separately. Comparing uniform fires and travelling fires, Law [19] determined that the ISO fire and the 5\% and 10\% TFM to be the most severe exposures, respectively.

3.3. Failure

The ultimate strain and three deflection (\(L/20\), \(L^2/400d\), and \(L^2/9000d\) over 1 min) failure criteria are considered in this paper as structural performance indicators. High deflections might lead to damage to the fire protection materials, fire suppression systems, or compartmentation, leading to faster heating or further fire spread, but these effects are not addressed here. For the ISO (standard) and SFPE (constant compartment temperature) fires, the considered fire duration is until the ultimate strain failure, while for other fire scenarios, the calculated total fire duration is considered, unless ultimate strain failure is reached prior to that. In this study, ultimate strain failure indicates the initiation of local or global collapse of the frame (for more details refer to [58]).

Buckling of the columns does not occur in any of the considered cases, likely because they were set to have a higher level of protection than beams (i.e. equivalent to 120 min of standard fire exposure) because beam failures are primarily examined in this paper. Therefore, columns do not reach sufficiently high temperatures to cause significant thermal expansion and additional thermally induced axial forces, which could lead to buckling. In addition, the utilization of the columns at ambient temperature is significantly lower than the utilization of beams (by up to 40\%). Thus, the structural member and frame failure, for the frame considered in this study, is mainly governed by the thermally induced loads generated as a result of the restrained thermal expansion of the beams, as in [59–61].

The results of different failure times for all fire scenarios are shown in Fig. 12. Based on the ultimate strain criterion, failure for the 2.5\%, 10\%, and 25\% TFM (travelling) and EC LC (Eurocode long-cool) fires occurs only when floors 5 to 8 are exposed to fire. For these floors, the heated beams are connected to the weakest column sections, which are not able to resist the thermally induced forces. Similar observation has been made in the study by Khorasani et al. [23] on a tall, steel, moment resisting frame. They found the upper floors to be more likely to reach the ‘formation of plastic hinges’ and ‘rate of deflection’ limit states than...
lower floors, because of smaller structural member section sizes in the upper floors. Fig. 12 indicates that failure in most cases occurs close to the internal connection in Bay 5 following the pull-in of the end columns. Failure times increase as the fire size decreases and are similar for EC LC and 25% TFM. Even though the CDF of frame utilization indicates that the frame subjected to EC LC fires is under the highest utilization in comparison to other fires, ultimate strain failure only occurs on fire floors 6 and 7. On the other hand, for the ISO, SFPE and 5% TFM, failure occurs for almost all fire floors. This failure time decreases on fire floors 6 to 8 by up to 30 min. No failure is observed for EC SH (Eurocode short-hot fire) and 48% TFM for all the aforementioned failure criteria. Due to relatively short duration of these fire exposures, members do not reach sufficiently high temperatures that would lead to structural failure as for other fire scenarios. Under the ISO fire the frame fails beyond the 60 min fire resistance of beams, at approximately 100 min.

The deflection related failure criteria of \( L/20 \), \( L^2/400d \), and rate of \( L^2/9000d \) over 1 min correspond to 0.455 m, 0.593 m, and 0.026 m/min, respectively. Based on the first two criteria (i.e., \( L/20 \) and \( L^2/400d \)), failure occurs for all fire scenarios, except for the fires on the top and bottom floors of the frame. Comparing the natural fires (EC and TFM), the 25% TF appears to be the worst, with failure occurring after 60 min of the fire exposure, followed by the 10% TFM (70 min), the EC LC fire (80 min), 5% TFM (1.5 h to 2 h), and the 2.5% TFM (2.5 h to 4 h). The third criterion on deflection rate is also exceeded for most fire cases, except for the uniform fire scenarios. For the EC LC the deflection rate limit is only exceeded for fire floors 6 to 8, as for the ultimate strain criteria. For the ISO and SFPE fires on floors 6 to 9, frame ultimate strain criteria is reached without the development of large rates of beam mid-span deflections. For all the failure criteria, the results indicate that there is no single fire scenario that would represent the worst case. For different fire exposures, failure occurs on a different range of floors subjected to fire. In terms of the ultimate strain criteria, the natural fires EC LC and 25% TFM lead to the lowest failure times (100 min), but only for floors 6 to 8. On the other hand, 5% TFM leads to failure for a larger range of fire floors (Floors 1 to 8), but the failure takes place only after a relatively large fire duration of approximately 5.5 h.

A comparison of the corresponding web temperatures of the elements that failed at the time of failure, according to different failure criteria, is shown in Fig. 13. The element temperatures at the ultimate strain failure are between 600 °C and 740 °C, except for the 2.5% TFM occurring on floor 5. For the limiting deflection and the limiting deflection rate failure criteria, failure temperatures are in the ranges of 450–700 °C and 320–700 °C, respectively. For the uniform fire scenarios, the temperatures of the elements when failure occurs are very similar. The difference for the fires occurring on the same floors ranges only between 2 and 33 °C. This difference is even smaller (0.04–6.8 °C) when the average temperatures of the members where failure occurred are compared. Changes in failure temperatures on different fire floors subjected to uniform fires are mainly due to the different levels of restraint, and in all cases are above 550 °C. Thus, it is likely that for EC SH and 48% TFM fires, failure does not occur under any failure criteria because the peak average member (i.e. beam or column) temperatures during the fire are not high enough to reach this critical value. For EC SH and 48% TFM the peak average member temperatures that develop
during the fire are 367 °C and 480 °C (peak localized web temperature is 559 °C), respectively. In the previous work, it was observed that structural response of the frame subjected to 48% TFM is similar to that under uniform fires, due to the relatively large fire size and the structural geometry.

For the travelling fire scenarios, the range of element temperatures when failure occurs is larger than for uniform fires, with the temperatures being as low as 100 °C for a slow travelling fire 2.5% TFM (ultimate strain failure criteria). 2.5% TFM case is discussed further in Section 3.5. In addition, travelling fires, unlike uniform fires, result in highly non-uniform temperature distributions in the compartment at different times of the fire exposure. Therefore, it is difficult to judge the critical member temperature for travelling fires by just looking at the localized temperatures of the elements that failed. These critical member temperatures are analyzed further in Section 3.4.

It should be noted that deflection criteria are typically evaluated at beam mid-span because they originate from testing simply supported members, and therefore this location was selected for the discussion of failure times and the temperatures reported in this study as well. However, in a realistic redundant structure, the beam mid-span may not always be the location of the maximum deflection of the beam. Comparison of failure times for the different failure criteria considering the whole beam span length with failure times considering beam mid-span only and the locations along the beam where failure first occurs are shown in Fig. 14. The results highlight that for the majority of the cases, the failure times are very close for any location in the beam, and when only mid-span is considered. Though, in general, failure occurs earlier in locations of the beam other than the mid-span and indicates that the maximum deflections do not necessarily develop at beam mid-span. This is in particular the case for deflection criterion of L²/400d. The failure time difference for the 2.5% TFM occurring on floor 7 is 6.4 h. Fig. 14 illustrates that for the uniform fires, the deflection criteria is first exceeded at beam mid-span or very close to it. However, for travelling fires, the critical deflection locations are further away from beam mid-span by up to 3.3 m (36.5% of the beam length) towards the fire origin.

3.4. Critical member temperatures

As already identified previously, travelling fires result in non-uniform and transient temperature distributions in a compartment. Thus, failure cannot simply be related to the temperature of the single element that failed (by element, it is inferred here a beam element in LSDYNA that forms part of a structural member and not the whole structural member). Other factors, such as the peak temperature of any element in the compartment at the time of failure, whether the element failed during the heating or cooling phases, and the average member temperatures, need to be considered as well. A comparison of the temperature of the failed element with the peak temperature in the element until the failure time, and the peak temperature of any element in the compartment when failure occurs, are illustrated in Fig. 15. Both localised element and average structural member (i.e. beam or column) temperatures are investigated. The results indicate that, for most of the cases, the elements fail during the heating of the failed element. Element failure during cooling occurs for the small travelling fire scenarios only, i.e. 2.5% and 5% TFM. In most cases, the failed elements are
located in the bays that experience heating, that is, when the near-field of the travelling fire is approaching or is in the same bay. Comparison of the temperature of the failed element and peak temperature of any element in the compartment when failure occurs, demonstrates that in only a few travelling fire scenarios the element that failed is the hottest element at that time. The differences between the temperature of the failed element and the peak temperature of any element in the compartment are up to 460°C (2.5% TFM), 160°C (5% TFM), 90°C (10% TFM), and 80°C (25% TFM). Peak average structural member temperatures in the compartment, when failure occurs, show a closer link

Fig. 14. Comparison of failure times for deflection criteria considering beam mid-span only and any location along the beam (left) and illustration of the locations where deflection criteria are first exceeded along the beam span (right).

Fig. 15. Comparison of failed element/average member temperature with peak element/average member temperature until failure and peak temperature of any element/member in the compartment at and until failure. Element temperatures (top) and average member/bay temperatures (bottom).
to the failed elements. For approximately half of the fire scenarios, irrespective of the fire size, failure occurs in the bay with the highest average member temperature in the compartment at that time. Fig. 16 shows that for the majority of the cases including the EC (Eurocode) fires, failure occurs while the peak temperatures in the frame members are still increasing. When the peak temperature in the frame members begins to decrease, that is, when the near-field has reached the far end of the compartment, failure occurs for a range of fire floors subjected to 25% TFM. In general, the peak element and the structural member temperatures in the compartment at failure time are between 550 and 700°C, and 400 and 625°C, respectively.

Fig. 17 shows the comparison of the time to failure with the time to reach the critical member temperature in the compartment (left), failed element (middle), and failed beam (average structural member temperature) (right). In two figures on the right only the TFM scenarios are compared. Data points plotted at the time of 1000 min indicate that failure did not occur or the critical member temperature was not reached depending on the axis.

3.5. Location of peak member temperature

In previous work [3] on the thermal response of structures subjected to the iTFM, the location of the peak structural member temperature along the fire path for different fire sizes has been investigated, and was found to occur towards the end of the fire path. In Fig. 18 these locations are compared with the locations of the elements for which failure occurs under the ultimate strain and deflection failure criteria. In addition, the locations of peak member temperature along the fire path at the time of failure are shown as well. In this figure, the locations of the critical deflections are illustrated considering the whole span of the beam, rather than just its mid-point. The results indicate that, for the deflection criteria, failure tends to occur in the first few bays of the frame and that there is no correlation with the location of the peak member temperature, especially for the 5% and 10% TFM (travelling) fire scenarios, though failure does occur close to the location of the peak member temperature in the compartment at the time of failure. This is likely because large deflections start to develop in beams at temperatures as low as 300 °C and are not directly linked to the loss of material strength but are a result of the thermally induced strains [56]. On the other hand, failure due to the ultimate strain criterion tends to occur towards the end of the fire path within the region where peak member temperatures in the compartment develop. Peak member temperatures in the compartment at the time of failure are within the same region as well. In only one case, i.e. 2.5% TFM on fire Floor 5, failure is located in the first bay of the frame, but the location of peak
membertemperatureinthecompartmentistowardstheendofthefire path. The element in this case fails due to high tensile axial forces which occur during the cooling of the member at a temperature of 100 °C (see Section 3.3). For small travelling fire sizes, structural members are exposed to long durations of far-field heating or cooling even after the near-field has progressed to other areas in the compartment.

Fig. 19 shows the numbers of cases that failure occurred at different locations along the fire path. The critical deflections for all the uniform fire scenarios tend to occur in the end bays of the frame, while for the travelling fires in the internal bays (2.5–10% TFM) and end bays towards the end of fire path (25% TFM). The ultimate strain criterion in most cases is exceeded in members connected to columns C2 and C5. They are close to the end columns and after the pull-in of these columns large bending moments are generated around columns C2 and C5. The locations of the highest utilization for uniform fires are concentrated around the latter columns as well. For travelling fires, they are located close to columns C3, C4, and C5 where higher peak temperatures are reached in the members. In general, these results indicate that the location of peak temperature in the compartment could likely be used to identify critical structural members in the compartment for the structural ultimate strain criterion. However, further studies considering different geometries and 3D effects need to be conducted to confirm this.

4. Conclusions

Critical temperature criterion is widely used in the assessment of structures subjected to fire as a failure indicator. However, it is unclear how representative it is of the actual structural failure, particularly when non-uniform fires such as travelling fires are considered. In this study, the effect of fire exposure (uniform and non-uniform fires) and fire floor location on ultimate strain, utilization, beam mid-span deflection, and critical temperature limit states of the multi-storey steel frame have been investigated.

The comparison of various failure criteria (ultimate strain, utilization, mid-span deflection, and critical temperature) show that there is no consistency between them, revealing a far more complex problem than reported in the literature. The simulations show that for the structure analysed, the long uniform fires result in the highest average utilization, while short-hot parametric fires and 48% travelling fire result in the lowest utilization. The most severe fire scenarios in the terms of the shortest failure times were 25% travelling fire (deflection and ultimate strain) and long-cool parametric fire (ultimate strain). For most cases examined, deflection failure criteria are reached before the ultimate strain criterion, and at lower structural member temperatures. Out of three deflection criteria examined, results indicate the limiting deflection of $L/20$ to be the most conservative and the limiting deflection of $L^2/900d$ to occur closest in time to ultimate strain criterion. On the other hand, the limiting rate of deflection of $L^2/9000d$ over 1 min is not always reached prior to the ultimate strain criteria, and is the least conservative. In addition, contrary to the typical assumption, the maximum deflections are found to not necessarily develop at beam
mid-span. This is particularly the case for travelling fires, which heat the structure non-uniformly and, therefore, the structure experiences a more irregular deflection pattern.

In general, the results indicate that there is no single fire scenario which would represent the worst case. For different fire exposures, failure occurs on different floors. Column section size change on different levels of the frame are found to be important in defining the weakest floors. In most travelling fire scenarios, failure occurs during the heating, but for small travelling fire, ultimate strain failure occurs during cooling. Therefore, a range of different fire scenarios both uniform and non-uniform should be considered in the structural fire design, in order to capture all different likely structural responses and failure modes. This could be done by considering probabilistic analyses, as in [4–6], and/or selecting a range of travelling fires and uniform fires that would include both short and long fire exposures, similar to the fire scenarios considered in this study.

The simulations reveal that there is no relationship between the time to reach critical temperature and the actual failure time. This means that the critical temperature criterion does not predict accurately the structural failure in time, space or failure mode. Therefore, heat transfer analysis alone is not sufficient for structural failure prediction and advanced structural analysis is necessary. Nevertheless, it is shown that the use of critical temperature is conservative for simple steel frames such as the one studied in this work. The determination of the critical temperature value that is appropriate for conservative design is not a trivial task and should consider all potential failure modes.

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Appendix A. Effect of fire protection and beam section size

The effect of different levels of applied fire protection (i.e. thicknesses) to the steel beams and columns, and the increased beam section size, i.e. sectional capacity, on the failure time, the corresponding temperature of the failed element, and the frame utilization are shown in Figs. A1 and A2. The frame with both beams and columns designed for 120 min of the ISO fire resistance (B120 C120) only fails when subjected to the SFPE (constant compartment temperature) and ISO (standard) fire scenarios after 150 min of exposure. It indicates a 3 times higher fire resistance than for a frame with B60 C120. In the majority of the cases, the frames with the columns with the lower level of fire protection (B60 C60) and larger beam cross-section (B60 C120 – LC) tend to reach failure at earlier and later times, respectively, in comparison to B60 C120 but within a similar range (up to approx. 20 min differences). However, there are cases where lower column protection leads to an improved fire resistance (e.g. 2.5–10% TFM (travelling fires) – deflection criteria, ISO – all criteria), and larger beam section leads to earlier failure time (e.g. 25% TFM – $T_\text{c}^\text{2}/9000\text{d}$). It should be noted that members with a larger section could result in increased thermally induced forces, which in some scenarios could prove more critical in comparison with a smaller section. The most significant differences in failure time occur for 2.5–10% TFM (up to 200 min or even 370 min – 2.5% TFM). The critical temperatures of the failed elements in comparison to the B60 C120 are either higher or lower irrespective of the level of fire protection, section size or failure criteria. They range between 350 °C and 750 °C. The maximum differences are up to 130 °C. In most cases, the increased beam section size, protection and decreased column protection result in higher element temperatures at the time of failure.

The CDF of the maximum at any time, mean, and peak utilization of the members in the frame, which are shown in Fig. A2 (see Appendix A), indicate that the increased beam section size results in the highest improvement of frame utilization. This is followed by the increased beam protection (B120 C120), but the difference is relatively small. B60 C60 result in the most severe utilization rates for uniform fires, but for travelling fires utilization of the frame is similar to that with increased member fire resistance. For the 5% TFM, the 25% TFM and the EC SH (Eurocode short-hot), the frame with increased column protection even shows slightly higher maximum frame utilization. The reason for this is most likely the higher rigidity of the columns, with the higher protection resulting in higher axial restraint, and thus greater axial forces that develop within heated beams. Therefore, depending on the fire scenario, a higher level of fire protection for different members within the frame will typically either lead to an enhanced, or sometimes reduced, fire resistance.

Fig. A1. Comparison of failure times (left) and corresponding failed element web temperatures (right) for the cases with different levels of protection assumed for beam and columns and larger beam cross-sections with the base case (C120 B60).
Fig. A2. The effects of level of fire protection and beam section size on the CDF of maximum at any time average (lower bound) and peak (upper bound) utilization of members in the frame.

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