In this study, structural response of a seismically designed steel moment-resisting frame subjected to travelling fire is investigated. This is to determine the structural strength of a generic frame designed for an extreme load when subjected to fire as another extreme load in addition to quantifying the effect of travelling fire size on its collapse behaviour. In this study, using the concept of travelling fire, and calculating the thermal field applied to structural elements, a generic frame was examined against a family of fires travelling across its first floor. In this regard, the resolved far-field gas temperatures dependent on the distance to the centre of fire were considered in order to calculate the temperature at the unprotected steel members. Analysis results revealed that fire size can deeply affect the total collapse time of a frame so that by reducing the fire size to a half or a quarter, collapse time increases by 19% and 62%, respectively. It was also suggested that columns of such structures should be designed against travelling fire considering the effect of load redistribution by which axial forces of columns might be doubled compared to the nominal loads applied to them prior to fire.

Notation

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A$</td>
<td>cross-section of beam</td>
</tr>
<tr>
<td>$A_f$</td>
<td>floor area of fire (m$^2$)</td>
</tr>
<tr>
<td>$c_s$</td>
<td>temperature-dependent specific heat of steel (J/kg K)</td>
</tr>
<tr>
<td>$E$</td>
<td>modulus of elasticity</td>
</tr>
<tr>
<td>$H$</td>
<td>floor to ceiling height (m)</td>
</tr>
<tr>
<td>$H_p$</td>
<td>heated perimeter of beam</td>
</tr>
<tr>
<td>$h_c$</td>
<td>convective heat transfer coefficient (W/m$^2$ K)</td>
</tr>
<tr>
<td>$L$</td>
<td>total length</td>
</tr>
<tr>
<td>$L_f$</td>
<td>fire length (m)</td>
</tr>
<tr>
<td>$L_g$</td>
<td>gravitational load for the fire limit state</td>
</tr>
<tr>
<td>$n$</td>
<td>number of zones</td>
</tr>
<tr>
<td>$Q$</td>
<td>total heat release of fire (kW)</td>
</tr>
<tr>
<td>$Q'$</td>
<td>heat release rate per unit area (kW/m$^2$)</td>
</tr>
<tr>
<td>$q_f$</td>
<td>fuel load density (MJ/m$^3$)</td>
</tr>
<tr>
<td>$r$</td>
<td>distance from the centre of the fire (m)</td>
</tr>
<tr>
<td>$s$</td>
<td>spread rate (m/s)</td>
</tr>
<tr>
<td>$T_g$</td>
<td>gas temperature (K)</td>
</tr>
<tr>
<td>$T_{max}$</td>
<td>maximum ceiling jet temperature (°C)</td>
</tr>
<tr>
<td>$T_s$</td>
<td>steel temperature (K)</td>
</tr>
<tr>
<td>$T_w$</td>
<td>ambient temperature (°C)</td>
</tr>
<tr>
<td>$t_b$</td>
<td>burning time (s)</td>
</tr>
<tr>
<td>$t_i$</td>
<td>time the fire spends at one node location</td>
</tr>
<tr>
<td>$t_{total}$</td>
<td>total burning duration</td>
</tr>
<tr>
<td>$x_f$</td>
<td>far-field distance</td>
</tr>
<tr>
<td>$\Delta x$</td>
<td>grid size</td>
</tr>
<tr>
<td>$\Delta t$</td>
<td>time step</td>
</tr>
<tr>
<td>$\Delta t_a$</td>
<td>increase of unprotected steel members’ temperatures</td>
</tr>
<tr>
<td>$\theta$</td>
<td>radiative and reradiative emissivity of the material and gas combined</td>
</tr>
<tr>
<td>$\epsilon_s$</td>
<td>thermal elongation</td>
</tr>
<tr>
<td>$\psi$</td>
<td>steel temperature (°C)</td>
</tr>
<tr>
<td>$\rho_s$</td>
<td>density of steel (kg/m$^3$)</td>
</tr>
<tr>
<td>$\sigma$</td>
<td>Stefan–Boltzmann constant (W/m$^2$ K$^4$)</td>
</tr>
</tbody>
</table>

1. Introduction

Fire, earthquake and sudden structural element loss are the potential extreme hazards that might affect some structures. Among these potential hazards, earthquake is well addressed in the codes (IBC, 2009; UBC, 1994), as the field of earthquake engineering and seismology is quite mature because of the lessons learnt through the many past earthquakes. Although a very interesting and relevant topic, sudden structural element loss, which can trigger progressive collapse (as reported for the first time in the collapse of Ronan Point apartments in 1968 (Griffiths et al., 1968; Pearson and Delatte, 2005)), has only received attention after the 11 September 2001 terrorist attack on the World Trade Center in New York.
(Asgarian and Hashemi Rezvani, 2012; Hashemi Rezvani and Asgarian, 2012; Izzuddin et al., 2008; Khandelwal and El-Tawil, 2011; Khandelwal et al., 2009; Kim and Kim, 2009; Vlassis et al., 2008). Today guidelines provide regulations for the design of certain types of structures against progressive collapse (ASCE, 2005; GSA, 2003; NIST, 2007; UFC, 2005). When it comes to fire, the history is longer. A standard fire curve was introduced in 1917. It is still in use today, despite its major drawbacks (Babrauskas and Williamson, 1978). Main problems with the standard curve are that the differences in fuel load for different occupancy types, the size of the fire compartment or the ventilation conditions are not accounted for in this building to fire (Law et al., 2011). To address these issues, parametric fire curves were introduced. Unlike the standard fire temperature–time curve, these curves depend on the fuel load, inertia of linings and ventilation condition of a fire compartment (CEN, 2002). Although parametric curves are more realistic than standard fire curves, they are unsuitable for application in large, open-plan spaces that are often seen now. EN 1991-1-2 (BSI, 2002) states that these curves are only valid for compartments with floor areas up to 500 m² and heights up to 4 m; the enclosure must also have no openings through the ceiling and the compartment linings are restricted to having a thermal inertia between 1000 and 2200 J/m² s¹/² K. This means that highly conducive linings such as glass facades and highly insulating materials were not taken into account when the curves were being developed (Law et al., 2011). Although the great majority of buildings designed in the twentieth century fall within these limitations, only 8% of newly constructed buildings would meet the above requirements (Jonsdottir and Rein, 2009). This implies that parametric curves cannot be applied in order to assess the fire resistance of modern buildings.

Nonetheless, the most important shortcoming of these methods is the assumption of uniform burning and homogeneous compartment temperature regardless of the compartment size (Stern-Gottfried and Rein, 2012a, 2012b) as it is shown that temperature conditions are non-uniform in most compartments (Stern-Gottfried et al., 2010b). Tests have also shown that there is a high degree of temperature variations even within small compartments (Stern-Gottfried et al., 2010a; Welch et al., 2007). Besides, the major fires at the Interstate Bank in Los Angeles in 1988 (Routley, 1988), the One Meridian Plaza in Philadelphia in 1991 (Routley et al., 1991), World Trade Center towers 1, 2 (Pitts et al., 2005) and 7 (Gross and McAllister, 2005) in New York in 2001, the Windsor Tower in Madrid in 2005 (Fletcher et al., 2007) and the Faculty of Architecture building at TU Delft in 2008 (Zannoni, 2008) have shown that fires tend to travel around large compartments as flames spread, burning over a limited area at any one time rather than burning uniformly. These fires have been labelled travelling fires. It is also worth noting that major fire incidents can continue for many hours. For instance, the Interstate Bank fire burnt a little shy of full 4 h, at which point it was controlled by firefighters. Some extraordinary fires such as the fire at the One Meridian Plaza persisted for around 19 h before it was finally brought under control by sprinklers. These examples not only show the travelling nature of fire but also reveal that fires may have durations that are well in excess of the time periods associated with the traditional design methods. This is also because of the assumption of uniform burning in a compartment. Therefore, the traditional methods may underestimate exposure times as compared to the length of real fires, which in turn could affect the structural response. Even though there are important differences between what the traditional methods assume and what real fires are, in the past it was assumed that those traditional methods were conservative and suitable for engineering design. However, recent observations have shown that travelling fires are more dangerous to structures than design fires suggested by the traditional methods (Jonsdottir et al., 2010; Law et al., 2011). In addition, considering recent advances in structural analysis and modeling techniques, it is worth determining the true performance of structures exposed to travelling fire rather than just designing a single member.

Response of structures to fire has recently been the subject of several investigations. Fang et al. (2013) developed a temperature-independent approach framework for the practical design-oriented robustness assessment of multistory steel/composite structures against localised fire that was event-independent. Fang et al. (2011) examined the robustness of steel-composite building structures subjected to localised fire and found that overall system failure can occur at temperatures that are much greater than the conventional design criterion based on the axial force in the affected column, the temperature of which is reducing back to the ambient temperature. Besides, it was shown that depending on the level of loading, the upper ambient floors may provide an alternative load path for the fire affected floor. Fang et al. (2012) discussed the realistic modelling of multistory steel-composite car parks under a typical vehicle fire scenario, where emphasis was given to robustness assessment accounting for ductility of the floor systems subsequent to column buckling. Models employing more detailed two-dimensional (2D) slabs were recommended for extreme loading conditions, particularly when grillage models were found to predict global failure. Kodur and Dwaikat (2009) showed that the load level has a significant influence on the fire response of a restrained steel beam. They mentioned that at higher load levels, smaller axial compressive forces develop because of the accelerated development of catenary action. They also stated that increased axial restraint has a detrimental effect on the fire response as it generates higher axial compressive forces. Such large axial compressive forces lead to lateral buckling, which in turn results in larger deflections. It was also revealed that the rotational restraint, when uncoupled from the axial restraint, can enhance the fire response of steel beams. However, when coupled with axial
restraint, the enhanced effect of the rotational restraint is counterbalanced with the detrimental effect arising from the axial restraints. Nevertheless, the combined effect of both types of restraints increases the fire-induced restraint forces in beams.

Moreover, Dwaikat and Kodur (2011) developed a performance-based methodology for fire design of restrained steel beams. They showed that higher values of axial restraint on steel beams lead to earlier occurrence of yielding in the steel, which in turn leads to increased deflection at the early stages of fire exposure. The increase in fire-induced deflection leads to the development of tensile catenary force in the beam and this tensile force improves the fire resistance of steel beams. However, it was discussed that the connections have to sustain increased tensile forces developed in the catenary phase. Sun et al. (2012b) developed a robust combined static-dynamic procedure by which the collapse mechanism of structures under different fire scenarios was investigated. The results showed that for an unbraced frame, the different loading ratio and beam section can generate different collapse mechanisms. The lower loading ratio and larger beam section resulted in higher failure temperature, which triggered the global collapse of the frame. It was also revealed that the bracing system is helpful in preventing the progressive collapse of the frame.

Sun et al. (2012a) also investigated the collapse behaviour of braced steel frames exposed to fire and concluded that statically indeterminate cross-bracing, in which a tension diagonal can always compensate for the buckling of the compression diagonal, would be a much more structurally efficient way of designing a hat truss to combat progressive collapse. Also, it was revealed that a vertical bracing system can not only increase the lateral restraint of the frame, reducing the pull-in effect of the columns, but also can effectively prevent the switching of the collapse mode from local to global.

Travelling fire as a new line of research in the field of structural fire engineering has enjoyed some attention in recent years. Jonsdottir et al. (2010) compared the steel temperatures resulting from travelling fires to those of traditional methods. The results indicated that the traditional parametric temperature–time curve tends to underestimate the maximum steel temperature. Furthermore, more severe conditions were predicted for small travelling fires than for large uniform fires, both for unprotected and protected steel. The maximum steel temperature was predicted for fire sizes between 5% and 10% for protected steel, whereas it was found to be independent of the fire size for unprotected steel. For unprotected steel, the travelling fire methodology predicted 65–95% higher steel temperatures compared to the parametric fire. Röben et al. (2010) studied structural behaviour during a vertically travelling fire. It was shown that a time delay between floor fires would affect the global response of high-rise structures. It was also concluded that generally neither simultaneous nor vertically travelling fires are worst-case scenarios as they result in different structural responses, each of which might be the worstcase.

Stern-Gottfried et al. (2010a) developed a performance-based methodology using travelling fires for structural analysis by which the response of a concrete frame was investigated. It was shown that travelling fires with sizes between 10% and 25% of the floor area induced the highest rebar temperatures. These temperatures were higher than the maximum temperatures induced by the equivalent parametric fires and similar to those induced by a 100 min standard fire. This finding demonstrated that considering only uniform fires in structural design may not always be as conservative as previously assumed. Ellsbury and Bailey (2011) investigated the performance of a post-tensioned concrete floor for a horizontally travelling fire and showed that horizontally travelling fire scenarios and the inter-zone time delay, affect the time-deflection behaviour considerably over the duration of the fire.

Law et al. (2011) studied the influence of travelling fires on a concrete frame. Analysis results demonstrated that travelling fires had a more severe impact on the performance of this structure than the Eurocode parametric fires. Therefore, it was suggested that the Eurocode fires cannot be considered conservative and the fires of medium duration and size were the most severe in terms of their impact on the structure. Stern-Gottfried and Rein (2012a, 2012b) also introduced dynamics of travelling fires as the way by which structures can be designed for more severe conditions produced by fires. In addition, they developed a method by which the resolved temperature–time curve of travelling fires can be sketched.

However, the great majority of the studies listed above have assumed that fire moves suddenly; that is, it jumps from one part of a floor area to the next, after each burning time. Furthermore, they considered only one or two far-field temperatures and assumed time delays for investigating the effect of travelling fire on structures, although these assumptions are not considered realistic based on the dynamics of fire (Stern-Gottfried and Rein, 2012a). On the other hand, although researchers have studied thermal fields applied to structural members, no-one has looked at the global response of structures. As steel moment-resisting frames (MRFs) are widely used worldwide, the aim of this study is to investigate the performance of a generic steel MRF, which is designed against earthquake and is exposed to travelling fire. Through such investigation, the effect of fire size on the fire resistance of the studied frame will be determined. Further, the probable modes of failure and the sequence of failure progression leading to global collapse of the frame will be determined. On the other hand, the capacity of the frame to redistribute the load carried by failed structural elements to other elements will be measured to investigate the ways by which designers can optimally design the whole of a building against fire rather than just designing a single element exposed to high temperatures.
In the following, the adopted travelling fire methodology is first explained and then the way in which the temperature-time curve of structural elements is calculated and applied is clarified. A generic seismically designed steel MRF is then studied under the defined fire using this method. It should be noted that in this study, the main aim has been the understanding of the response of a 2D primary steel frame, exposed to various fire sizes travelling across the floor plan. The main limitation of the chosen methodology has been ignoring the potential fracture of the connections, and not considering the redistribution occurring by the composite concrete slab into the third dimension. These issues are currently under investigation.

2. Travelling fire
The methodology used in the current study for the evaluation of the response of structures against travelling fire is based on that provided in the references (Law et al., 2011; Stern-Gottfried and Rein, 2012a, 2012b), which is different to that of Clifton (1996). The main difference is that here the effect of all travelling fire scenarios on the structural response is investigated instead of the worst case being determined based on the highest temperature achieved in the structure.

As fire travels across a floor, the fire-induced thermal field is divided in two regions: the near field and the far field. The near field is the burning region of the fire and where structural elements are exposed directly to the flames and experience the most intense heating. The far field is the region remote from the flames where structural elements are exposed to hot combustion gases (the smoke layer) but experience less intense heating than those exposed near field to the flames. Unlike traditional methods, the travelling fire methodology does not assume a single, fixed fire scenario but rather accounts for a whole family of possible fires, ranging from small fires travelling across the floor plate for long durations with mostly low temperatures to large fires burning for short durations with mostly high temperatures. Using a family of fires enables the methodology to overcome the problem that the exact size of an accidental fire cannot be determined a priori. This range of fires allows identification of the most challenging heating scenarios to be applied to the structure as input for the subsequent structural analysis. It should be noted that, since for unprotected steel members the highest temperatures do vary a lot, the influence of all travelling fire scenarios on the structure should be checked. This is because of the effect of far-field temperature on the global performance of the structure. Each fire in the family burns over a specific surface area, denoted as \( A_f \), which is a percentage of the total floor area, \( A \), of the building. Conventional methods, however, only consider full-size fires, which are analogous to the 100% fire size in this methodology. All other burning areas represent travelling fires of different sizes, which are not considered in the conventional methods.

2.1 Burning time
Considering a family of fires, this methodology assumes that there is a uniform fuel load across the fire path and that the fire will burn at a constant heat release rate per unit area. From this, the total heat release rate is calculated by Equation 1.

\[
\dot{Q} = A_f \dot{Q}^0
\]

The local burning time of the fire over area \( A_f \) is calculated by Equation 2.

\[
t_b = \frac{q_f}{\dot{Q}^0}
\]

2.2 Near-field and far-field temperatures
The near field is dominated by the presence of flames. The maximum possible heating in a structural element would result from direct contact with the flames. These temperatures have been measured in small fires in the range of 800 to 1000°C (Audouin et al., 1995) in larger fires up to 1200°C (Drysdale, 2011). The far-field temperature decreases with distance from the fire. The maximum exposure to hot gases results when the structural element is on the exposed side of the ceiling. Temperatures at the ceiling are therefore used in this methodology. The experimental and theoretical work by Alpert (1972) provides full expression and the coefficients that are valid for an axisymmetric, unconfined ceiling jet as a function of radial distance from the fire centre. The correlation is given in Equation 3.

\[
T_{\text{max}} - T_{\infty} = 5.38 \left( \frac{\dot{Q}/r}{H^2} \right)^{3/2}
\]

Alpert gives a piecewise equation for the maximum ceiling jet temperatures to describe the near-field \((r/H \leq 0.18)\) and far-field \((r/H \geq 0.18)\) temperatures, but only the far-field equation is used here. The methodology assumes the near field to be at the flame temperature and does not use the expression given by Alpert. If the results of the above equation exceed the specified near-field temperature at any point, they are capped at the flame temperature. It is worth noting that in the current study it is assumed that columns have the same temperature along their length, which is determined based on the distance from the fire centre.

2.3 Spatial discretisation
It is assumed that the fire extends the whole width of the building and travels in a linear path along the structure’s length. Other fire paths are possible, but results shown in Law et al. (2011) demonstrate that they do not greatly alter the structural response. Thus a single linear path is chosen for this further
development of the methodology. As the far-field temperature is assumed uniform along the width of the building but varies along its length for the assumed linear path, the problem is treated as one dimensional (1D). Thus the far-field temperature for any given fire size can be calculated at any position in the structure by its distance from the fire. This discretisation is similar to the strips examined by Clifton in his large firecell model (Clifton, 1996).

The fire is assumed to travel at a constant spread rate, $s$, across the floor plate. This is calculated by Equation 4 and is related to the burning time and the fire size.

4. \[ s = \frac{L_f}{t_b} \]

To track the fire location over time and enable calculation of the far-field temperature at various distances, the building is broken up into numerous nodes, each with a fixed width $\Delta x$ (also referred to as the grid size). Each node has a single far-field temperature at any given time. As the fire travels across the floor plate, nodes go from being unburnt, to being on fire, to completely burnt out. Figure 1 illustrates the 1D discretisation of the building. The far-field distance ($r = x_{ff}$) is taken from the fire centre to the node (centre of zone) being examined (node $i$).

In order to resolve the movement of the fire adequately, the time step, $\Delta t$, is determined by Equation 5.

5. \[ \Delta t = \frac{\Delta x}{s} \]

This definition allows the time step to capture the movement of the fire from one node to the next. The time the fire spends at one node location, $t_i$, is the sum of the travel time across the node plus one local burning time. The whole node is assumed to start burning when the leading edge of the fire enters from the near side. The whole node is assumed to be burnt out when the trailing edge of the fire passes the far side. This is given by Equation 6.

6. \[ t_i = \Delta t + t_b \]

As the fire travels across $n-1$ nodes (the initial condition has node 1 burning at $t=0$), the total burning duration, $t_{total}$, is the travel time across the rest of the floor plate plus one burning time. Having this, plus noting that $n = L/\Delta x$, means that the total burning duration is given by Equation 7.

7. \[ t_{total} = t_b \left( \frac{L - \Delta x}{L_f} + 1 \right) \]

3. The case study structure

To investigate the effect of travelling fire on the response of seismically designed steel MRFs, a generic four-storey framed building was designed for a high seismic activity zone. The building was square in plan and consisted of four bays of 6 m in each direction with the storey height of 3·6 m. The plan and elevation view of the frames are shown in Figure 2 and Figure 3. In the design process, the gravity loads were assumed to be similar to those in common residential buildings. To design the members against earthquakes, equivalent lateral static forces were applied to all storey levels. The dead and live loads of 3·5 and 5·0 kN/m$^2$, respectively, were used as gravity loads in all storeys. To study the robustness of the building, an internal frame was analysed under load. This frame is depicted in Figure 2 by the dotted line. S275 steel, with a yield strength of 275 N/mm$^2$ and Young’s modulus of 210 kN/mm$^2$ at ambient temperature, was used for both beams and columns. The steel material properties at elevated temperatures are

Figure 1. Spatial discretisation and associated parameters (Stern-Gottfried and Rein, 2012b)
degraded according to CEN (2005). Section sizes of the beam and columns of the studied frame are shown in Table 1. In the current study all structural elements are assumed to be unprotected. The main reason for this assumption is that the case study structure is designed to resist another extreme load event; that is, earthquake. The aim is to find out how much resistance can be expected of structures that are designed against high seismic forces when subjected to travelling fire and those structures are not necessarily protected for fire.

### 4. Methodology of robustness checking

To investigate the structural response of the case study steel MRF exposed to travelling fire, eight scenarios including fire sizes of 12.5%, 25%, 37.5%, 50%, 62.5%, 75%, 87.5%, 100% were defined in which the fire developed according to the predefined fire size and travelled across the first floor of the structure. In each scenario, the response of the structure was investigated to determine the probable modes of failure, sequence of failure, fire resistance time and the capacity of the frame in bridging over the loads carried by the defected structural element to intact parts. In this study, two measures were used to determine the capacity of the frame exposed to fire, including the load-bearing capacity of structural elements and the mid-span deflection of beams. For the former, the failure was defined as buckling of columns during non-linear dynamic analyses, whereas for the latter, the failure was defined by mid-span deflections exceeding $L/20$ in which $L$ is the beam span (Kodur and Dwaikat, 2007) at which the beam is not capable of transferring loads and plastic hinges are completely formed. It is worth noting that buckling of columns is identified, while vertical displacement of the tops of columns suddenly decreases as temperature increases.

### 5. Development of the numerical model

#### 5.1 Modelling of structure

OpenSees (Mazzoni et al., 2007) finite-element program was used to model and analyse the structure subjected to travelling fire. A series of non-linear dynamic analyses was performed for an internal frame of the case study structure, as shown in Figure 2 (with dotted lines) and Figure 3. To model the steel behaviour, a bilinear kinematic stress–strain curve was assigned to the structural elements using Steel02Thermal from the library of materials available in OpenSees. A transition curve was provided for this material at the intersection of the first and second tangents to avoid sudden changes in the local stiffness matrices formed by the elements and to ensure a smooth transition between elastic and plastic regions. This class of material is derived by modification of the existing steel material class ‘Steel02’ to include the temperature-dependent properties according to EN 1992-1-2 (CEN, 2002) carbon steel at elevated temperature. The definition of the parameters of Steel02Thermal is the same as that of Steel02 in OpenSees command language manual. A strain hardening modulus of 1% $E$ was considered for the member behaviour in the inelastic range of strains. This behaviour, together with other material properties used for modelling steel, is shown in Figure 4. Young’s modulus and yield stress were reduced depending on temperature in accordance with CEN (2005). For all structural elements, beam–column elements in combination with fibre cross-sections were used to model the cross-sectional areas.

<table>
<thead>
<tr>
<th>Storey</th>
<th>External column</th>
<th>Internal column</th>
<th>Beam</th>
</tr>
</thead>
<tbody>
<tr>
<td>1,2</td>
<td>UC356 × 368 × 153 / C2 107</td>
<td>UC356 × 368 × 202 / C2 153</td>
<td>UB457 × 152 × 67</td>
</tr>
<tr>
<td>3,4</td>
<td>UC356 × 254 × 107 / C2 153</td>
<td>UC356 × 368 × 153 / C2 152</td>
<td>UB457 × 152 × 67</td>
</tr>
</tbody>
</table>

Table 1. Section of all members of the generic frame
Plastification of elements over the member length and cross-section was considered as well. Large displacement effects were also accounted for through the employment of corotational transformation of the geometric stiffness matrix. All connections were assumed to be ideally rigid. It is also worth noting that the numerical model is a 2D model that does not consider the full three-dimensional modelling of concrete slab on the top flange of beams. However, to reduce the impact it might have on the results, the temperature in the upper flanges of the beams below the slab is reduced. Besides, as fires occupy full width and travel along the frame length, a 2D model is adequate; hence load-sharing with parallel frames is not considered. The thermal elongation of steel fibre is calculated according to Equation 8.

$$\varepsilon_s(\theta) = -2.4 \times 10^{-4} + 1.2 \times 10^{-5} + 0.4 \times 10^{-8} \theta^2$$

for $20^\circ C \leq \theta \leq 750^\circ C$

$$\varepsilon_s(\theta) = 11 \times 10^{-3}$$

for $750^\circ C \leq \theta \leq 860^\circ C$

$$\varepsilon_s(\theta) = -6.2 \times 10^{-3} + 2 \times 10^{-7} \theta$$

for $860^\circ C \leq \theta \leq 1200^\circ C$

## 5.2 Model verification

OpenSees capability to model structures has been verified previously at ambient temperature (Asgarian and Hashemi Rezvani, 2012; Asgarian and Jalaeefar, 2011; Asgarian et al., 2010, 2012; Hashemi Rezvani and Asgarian, 2012, 2014) and at elevated temperature (Jiang and Usmani, 2012; Usmani et al., 2012) against several experimental observations and analytical calculations. As this capability has been verified, no verification analysis was performed in this study.

## 5.3 Gravitational loading

As the travelling fire is defined in the time domain, all loads including gravitational loads have to be defined in the time domain as well. The gravitational loads considered for the fire limit state comprised a combination of dead and live loads according to Equation 9 (CEN, 2002).

$$L_g = DL + 0.5 LL$$

where DL is dead load and LL is live load.

For the case study structure, this worked out to be a total line load of $36 \text{kN/m}$ to work within the realm of the time domain. The gravity loads were linearly increased during 5 s to reach their final values, and after that, for the remainder of the analysis time kept unchanged.

## 5.4 Thermal loading

On gravity loads reaching their maximum at $t=5$ s, loading was kept unchanged for a further 5 s to avoid dynamic excitation. Afterwards, thermal loading of the structural element was started in accordance with the predefined fire scenarios. The fires were assumed to initiate at the left end of the frame shown in Figure 2, occupy the full width and burn along its length over time as illustrated in Figure 3. Travelling fire scenarios together with fire size and area, the heat release rate, the total burning duration and the spread rate are shown in Table 2 based on the grid size, $\Delta x=1.5 \text{ m}$. As explained in the previous sections, the total burning time increases as the fire size decreases. The family of fires created was used to generate transient gas phase temperature fields across the structure. The temperature fields were then used as input to calculate the resulting temperature of unprotected steel structural elements. Owing to high conductivity of the material, the unprotected steel members’ temperatures were calculated by a lumped mass heat transfer method, as given by Buchanan (2001) and shown in Figure 4.

![Figure 4. Structural steel behaviour, where $F_y$ is the yield strength, $\varepsilon_y$ is the yield strain and $\varepsilon_u$ is the ultimate strain](image)

<table>
<thead>
<tr>
<th>Fire size: %</th>
<th>$A_i: m^2$</th>
<th>$Q: MW$</th>
<th>$t_{total}: \text{min}$</th>
<th>$s: \text{m/min}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>12.5</td>
<td>72</td>
<td>36</td>
<td>161.50</td>
<td>0.16</td>
</tr>
<tr>
<td>25</td>
<td>144</td>
<td>72</td>
<td>90.25</td>
<td>0.32</td>
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<tr>
<td>37.5</td>
<td>216</td>
<td>108</td>
<td>66.50</td>
<td>0.47</td>
</tr>
<tr>
<td>50</td>
<td>288</td>
<td>144</td>
<td>54.62</td>
<td>0.63</td>
</tr>
<tr>
<td>62.5</td>
<td>360</td>
<td>180</td>
<td>47.50</td>
<td>0.79</td>
</tr>
<tr>
<td>75</td>
<td>432</td>
<td>216</td>
<td>42.75</td>
<td>0.95</td>
</tr>
<tr>
<td>87.5</td>
<td>504</td>
<td>252</td>
<td>39.36</td>
<td>1.10</td>
</tr>
<tr>
<td>100</td>
<td>576</td>
<td>288</td>
<td>36.81</td>
<td>1.26</td>
</tr>
</tbody>
</table>

**Table 2. Details of family of travelling fires**
in Equation 10. It is worth noting that for the current study, temperature increase was calculated for every single second.

\[ \Delta t_s = \frac{H_p}{A \rho_v c_s} \left[ b_h(T_g - T_s) + \sigma c(T_g^4 - T_s^4) \right] \Delta t \]

It should be noted that the majority of previous works performed on travelling fire took a single far-field temperature for each fire size, independent of distance. Besides, they just determined the worst case according to the highest temperature reached in the structure examined. However, by the method explained here, this study uses spatially varying far-field temperatures to be carried into the heating calculations by which the true performance of the generic steel MRF was investigated. While this results in a larger volume of information to be passed to the structural analysis, it provides a more accurate representation of the fire dynamics for each scenario, which is particularly important in the analyses of whole frame behaviour.

For the travelling fire scenarios studied in this work, the fuel load density, \( q_f \), was assumed to be 570 MJ/m². The heat release rate per unit area, \( \dot{Q}' \), was taken as 500 kW/m². So, accordingly, the burning time, \( t_b \), was 19 min. Besides, the maximum value of 1000°C was chosen here for the near-field temperature to represent the averaged conditions. Furthermore, the grid size was assumed to be \( \Delta x = 1.5 \) m, which means that for the generic frame, the total number of zones was \( n = 16 \). In each zone, the temperature was calculated based on the distance between the centres of the zone being examined and that of travelling fire. For each fire scenario and with five columns per frame, 21 temperature-time curves were defined and applied to the structure. Considering the influence of the composite slab on the temperature distributions within the beams, it was assumed that the temperature of the top flange of a heated beam was 70% of the temperature of the bottom flange. Figure 5 depicts the way travelling fire fronts sweep across the studied structure.

To provide some insight into the transient gas phase temperature fields across the structure for different fire sizes and accordingly the resulting unprotected steel temperature of structural elements, the transient gas phase temperature fields of the second quarters of beam 1 and beam 2 are depicted in Figure 6 and Figure 7, respectively, whereas Figure 8 and Figure 9 show the resulting steel temperature of the so-called zones. According to these figures, it is shown that there is a longer total burning duration for smaller burning areas.

### 6. Response of the frame to travelling fire

In this section the structural response of the generic steel MRF exposed to travelling fire is presented in accordance with the predefined family of fires listed in Table 2.
columns, survives because of its capacity to transfer loads carried by the failed structural elements to the intact parts of the frame. As is shown in Figure 11, large load redistribution occurs while the fire travels across the floor. It is evident that as the temperature increases, column 1 experiences compression owing to restraints and this in turn decreases the axial load carried by column 2. This continues until the buckling of column 1 occurs at $t = 374$ s (730°C) at which time, the majority of its load is transferred to column 2. This transfer on the other hand leads to a decrease in the axial load of column 3. At time equal to 1150 s when column 1 goes from
the near-field temperature to the far-field temperature, it can carry more loads so its axial load increases, whereas the axial force of column 2 decreases. This condition remains until \( t = 1776 \text{ s (740°C)} \) at which column 2 buckles. This causes another redistribution of loads carried by column 2 to the adjacent member's column 1 and column 3, which have adequate capacity to withstand the excess loads. This behaviour can be likened to a moving localised fire, which at a particular point in time only involves one column at the most. So, the critical temperatures of columns are nearly the same depending on how the loads are redistributed in the frame. However, the buckling of the above-mentioned columns is transient and the aforesaid trend continues until the end of the fire and the frame survives the travelling fire of size 12.5%, albeit based on the force-based perspective considered here. Figure 12 depicts the mid-span deflection of beams for the current fire case. As is seen, the critical time of the frame at which the mid-span deflection of one of the beams reaches \( L/20 \) is \( t = 3430 \text{ s (775°C)} \). So, from the mid-span deflection perspective, it can be reported as the failure mode of the structure.

6.2 Fire size of 50%

Figure 13 and Figure 14 show the forces in columns 1 to 5 and the stability behaviour of the frame plotted against time when a travelling fire of size 50% occurs in the first floor of the frame. It can be seen that the compression force in column 1 increases during the initial heating stage, until it reaches its buckling load at \( t = 374 \text{ s (730°C)} \). Beyond this point, the compression force of column 1 is reduced. This causes little change to the compression force in column 3. Most of the load transfer is from column 1 to column 2. Column 2 continues to carry the excess loads until its buckling happens at \( t = 626 \text{ s (708°C)} \). The frame stays stable until plastic hinges have formed at the ends of all beams adjacent to column 2. At this point complete collapse of the frame occurs. From the mid-span deflection perspective, on the other hand, the critical time of the frame for this scenario is \( t = 767 \text{ s (703°C)} \) at which...
beam 2 reaches the mid-span deflection of 30 cm based on Figure 15.

### 6.3 Summary of results for collapsed scenarios

Since all the fire sizes travelling across the first floor except the first scenario lead to global collapse of the case study frame base on the force-based perspective, in this section a summary of the observed results is discussed. Figure 16 depicts the deformed shape of the frame while exposed to travelling fire of size 62.5%. Figures 17–19 illustrate vertical displacement of the top node against time for columns 1 to 3, respectively, for the travelling fire scenarios in which global instability of the frame was observed. As all fire scenarios were initiated from the left side of the frame, column 1 starts to buckle prior to other columns. According to these diagrams, it is evident that for columns 2 and 3, increasing the fire size expedites their buckling. However, this is not the same for the first column, so that it buckles at the same time and temperature ($t = 374$ s and 730°C, respectively) in all fire scenarios.

Figure 20 and Figure 21 depict the mid-span deflection of beams 1 and 2 against time for various fire scenarios. As mentioned previously, the limit state for mid-span deflection is $L/20$. According to these diagrams it is evident that as the fire size increases, mid-span deflection of both beams increases but the point is that beam 2 reaches its limit state prior to beam 1 cases where the fire size is equal to or greater than 50% of the floor area. This is attributable to the higher axial restraints affecting the internal spans preventing free elongation and as such forcing deflection out of plane.

### 7. Discussion of the results

Tables 3 and 4 summarise the data obtained by analysing the case study frame exposed to travelling fires of different sizes. In these tables the critical times and temperatures at which columns 1, 2 and 3 buckle are listed. Further, the time and averaged temperature of beams at which mid-span deflections of the first two bays of the frame reach their limit state ($L/20$) are stated. It should be noted that although it is evident in the following tables that the average critical temperature of beam 2 for reaching mid-span deflection limit state in fire size of 50% is less than the others, the current discussion ignores it because the mid-span deflection only represents one failure mode, not the collapse of the structure.

Based on the summary diagrams and the results listed in Table 3, it can be inferred that while the time at which column 1 buckles does not vary among predefined scenarios, it fluctuates between 62% and 29% for columns 2 and 3, respectively. Besides, it can be understood that while for column 1, the critical temperatures of various sizes do not vary, they have up to 11% and 1% difference for column 2 and column 3, respectively. However, it should be noted that buckling of column 3 just shows up at the fire scenarios greater than 50% of the floor area.

#### 7.1 Collapse time and collapse temperature ratios

Collapse of a structure occurs when there is no alternative path to transfer the loads that are carried by columns of a particular bay to the adjacent bays. So, for the travelling fires investigated in this study, buckling of column 2 while column 1 has already buckled is defined as the collapse of the structure.
Figure 16. Deformed shape of the generic frame exposed to travelling fire of size 62.5%: (a) buckling of column 1; (b) buckling of column 2; (c) buckling of column 3 and collapse of the frame

Figure 17. Vertical displacement of the top of column 1

Figure 18. Vertical displacement of the top of column 2
This is important because it provides a better understanding of the performance of the case frame, especially its collapse behaviour when subjected to various fire sizes. Accordingly, Figure 22 shows the collapse time and temperature ratios of the predefined scenarios of travelling fire against the fire size according to Equation 11. This can also be compared to the fire resistance time based on traditional methods.

In the current investigation, eight scenarios are studied; \( S = \{1, 2, \ldots, 8\} \) and collapse time and temperatures are in accordance with the data summarised in Table 3. Accordingly, except for the first travelling fire scenario (12.5%) in which no permanent instability occurs, it is evident that as the travelling fire size increases, the critical time and temperature for the occurrence of collapse in the first span decrease and increase, respectively.

As is seen, a travelling fire size equal to the whole floor area has the lowest collapse time; hence it has the collapse time ratio of unity. On the other hand, the collapse temperature ratio of unity belongs to the fire size of 25%. It is clear that by reducing the fire size to half and quarter, collapse time increases by 19% and 62%, respectively. These ratios, on the other hand, emphasise the importance of studies on the probability distribution of fire sizes occurring in different types of buildings as well as defining different limit states according to the mean fire size in order to design buildings cost-effectively.

7.2 Load increase factor

The load increase factor (LIF) can be calculated by using Equation 12 and the parameters defined in Figure 23.
Figure 24 illustrates this factor for all columns of the first storey of the case study frame against fire size.

12. \[ LIF = \frac{\text{Peak value of internal force}}{\text{Initial steady-state value of internal force}} \]

It should be noted that in Figure 24 only axial forces of columns are considered. This is because the bending moment of these structural members is not dominant in the collapse of the frame. Besides, the initial steady-state value of the axial force associates with the state of the structure at which only gravitational loading is applied and no fire is yet initiated.

According to Figure 24, except for columns 4 and 5, which did not experience near-field temperatures according to the collapse time of the frame, it can be said that column 1 in the last scenario and column 3 in the fourth scenario have the lowest and highest factors, respectively, 1.16 and 2.11. This is exactly what is expected in the structures wherein a large load redistribution occurs because the central elements have more alternative paths to transfer the loads; hence, larger redistribution of the loads occurs before collapse. Furthermore, generally it is shown that as the fire size increases this factor decreases. These findings suggest that columns of such structures should be designed against travelling fire actions considering the effect of load redistribution, by which axial forces of columns might double compared to the nominal loads applied to them based on the building codes. This, on the other hand, can explain the reason why column 2 buckles in lower temperatures in smaller fires. In other words, the critical temperature of column 2 is dependent on the loads transferred to it from columns 1 and 3. So, since the load magnification factor increases by the reduction in the fire size, it can be expected that column 2 buckles in such a way.

### Table 3. Critical times of the generic frame exposed to travelling fire

<table>
<thead>
<tr>
<th>Fire size: %</th>
<th>Column 1</th>
<th>Column 2</th>
<th>Column 3</th>
<th>Beam 1</th>
<th>Beam 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>12.5</td>
<td>374</td>
<td>1776</td>
<td>3944</td>
<td>—</td>
<td>3430</td>
</tr>
<tr>
<td>25</td>
<td>374</td>
<td>851</td>
<td>—</td>
<td>1079</td>
<td>—</td>
</tr>
<tr>
<td>37.5</td>
<td>374</td>
<td>708</td>
<td>—</td>
<td>926</td>
<td>949</td>
</tr>
<tr>
<td>50</td>
<td>374</td>
<td>626</td>
<td>—</td>
<td>848</td>
<td>767</td>
</tr>
<tr>
<td>62.5</td>
<td>374</td>
<td>579</td>
<td>776</td>
<td>802</td>
<td>668</td>
</tr>
<tr>
<td>75</td>
<td>374</td>
<td>553</td>
<td>698</td>
<td>744</td>
<td>614</td>
</tr>
<tr>
<td>87.5</td>
<td>374</td>
<td>540</td>
<td>639</td>
<td>709</td>
<td>578</td>
</tr>
<tr>
<td>100</td>
<td>374</td>
<td>524</td>
<td>599</td>
<td>683</td>
<td>548</td>
</tr>
</tbody>
</table>

### Table 4. Critical temperatures of the generic frame exposed to travelling fire

<table>
<thead>
<tr>
<th>Fire size: %</th>
<th>Column 1</th>
<th>Column 2</th>
<th>Column 3</th>
<th>Beam 1</th>
<th>Beam 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>12.5</td>
<td>730</td>
<td>740</td>
<td>730</td>
<td>—</td>
<td>775</td>
</tr>
<tr>
<td>25</td>
<td>730</td>
<td>655</td>
<td>—</td>
<td>848</td>
<td>—</td>
</tr>
<tr>
<td>37.5</td>
<td>730</td>
<td>700</td>
<td>—</td>
<td>848</td>
<td>711</td>
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<td>708</td>
<td>—</td>
<td>848</td>
<td>703</td>
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<td>62.5</td>
<td>730</td>
<td>713</td>
<td>643</td>
<td>847</td>
<td>704</td>
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<tr>
<td>75</td>
<td>730</td>
<td>720</td>
<td>648</td>
<td>847</td>
<td>711</td>
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<tr>
<td>87.5</td>
<td>730</td>
<td>726</td>
<td>649</td>
<td>846</td>
<td>717</td>
</tr>
<tr>
<td>100</td>
<td>730</td>
<td>727</td>
<td>650</td>
<td>845</td>
<td>720</td>
</tr>
</tbody>
</table>
8. Conclusion
In this study, the structural response of a generic seismically designed MRF exposed to travelling fire was investigated to examine the effect of fire size and its travelling nature across the floor plate on the robustness of the structure. In this regard, resolved far-field gas temperatures dependent on the distance from the centre of fire were considered to calculate the unprotected steel temperatures as the fire passes through the compartment. Analysis results revealed the following findings.

- Although fire size does not affect the time and temperature at which the first failure mode happens, it can influence the second failure mode (collapse of the first bay) considerably. Observations for the case study frame showed differences of 62% and 11% for time and temperature, respectively.
- As the travelling fire size increases, the critical time and temperature of the second column for the occurrence of collapse at the first span decrease and increase, respectively.
- For the travelling fire scenarios over 50%, the second span reaches its deflection limit state prior to the first one because of higher restraints applied to it when heated.
- Fire size can greatly affect the collapse time of the frame. In the case study frame it was observed that by reducing the fire size to half and quarter, collapse time increases by 19% and 62%, respectively. As the probability of occurrence of fires of different sizes is yet unknown, the only important conclusion that can be reached is that a probabilistic approach is required in combination with a sophisticated fire analysis in order to analyse and design structures cost-effectively against travelling fires of a certain size.

- It was observed that as fires of various sizes travel across the first floor, affected columns of the case study frame have to carry loads 1·16 to 2·11 times greater than the loads they were carrying before the fire ignition. It is therefore suggested that such columns should be designed in accordance with

![Figure 22. Collapse ratios of a family of fires](image)

![Figure 23. Parameters for calculation of dynamic increase factor](image)

![Figure 24. Load increase factors](image)
the LIF, which represents the mechanism by way of which loads are redistributed within a frame. Such a mechanism can lower the critical temperature of structural members leading to the instability of the structure.

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Rezvani and Ronagh

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