EFFECT OF TRAVELLING FIRE ON STRUCTURAL RESPONSE OF A GENERIC STEEL FIRE PROTECTED MOMENT RESISTING FRAME

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ABSTRACT

To simulate a fire inside large compartments, there is a pioneering method called 'traveling fire'. As steel structures are vulnerable to high temperatures, they are normally fireproofed by insulation materials appropriate for a specific duration of time. An investigation is performed here to examine the robustness of a generic four-story moment-resisting steel structure, fireproofed to comply with the one-hour standard curve, when it is subjected to traveling fire. The results show that while no collapse occurs during the 12.5%, 50% and 100%, the structure collapses under the 25% fire size at 75 min. This seems to be in contradiction with traditional belief, where it is assumed that taking into consideration a larger-scale fire in a compartment would increase the safety margin. The investigation performed also underlines that the fireproofing of structures does not necessarily provide adequate resistance under traveling fires.

KEYWORDS

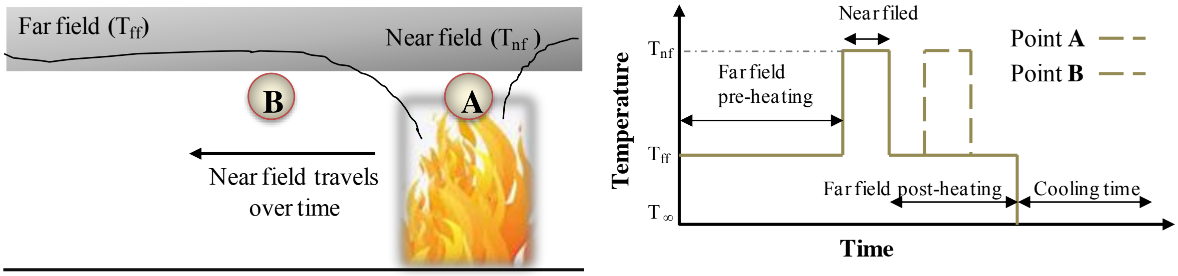
Standard fire curves, traveling fire, steel structures, robustness, fireproofing.

INTRODUCTION

What the natural fire curves and the standard fire curves have in common is that they consider a homogenous temperature throughout a compartment. While this assumption can be justified in small and medium compartments, it is not deemed to be precise in large compartments, as it has been revealed in several tests and observations that large compartments do not burn simultaneously throughout the enclosure (Stern-Gottfried and Rein, 2012b). This is the reason why the application of the natural fire curves is confined to compartments with areas up to 500 m^2, heights up to 4 m, and with no opening on the roof. In addition, highly conductive linings cannot be included to develop the fire curves. Although the majority of buildings designed in the last century fall within the scope of the natural fire curves, very few newly constructed buildings would meet the requirements mentioned (Jonsdottir et al., 2010). In addition, observations from real fires have proved that in large open areas the fire travels either vertically between the floors and/or across the floor plates. Therefore, combustible materials inside a compartment are not burnt concurrently and are consumed at a rate governed by the existing ventilation (Stern-Gottfried et al., 2010). This leads to a non-uniform temperature in the compartment.
There are a number of investigations concerning traveling fire, either vertically or horizontally. However, most of the structural studies mentioned above, however, have assumed that the fire moves suddenly from one zone to next. Besides, only a selected number of different temperatures were assumed in the investigations, which in turn is far from the situation in a real fire. In addition, studies considering the global response of structures under traveling fire are rare. Investigating the global robustness of structures subjected to traveling fire is thus of importance, and is the focus of the current study. From a different perspective, as steel structures are highly vulnerable to fire, they are normally fireproofed using insulation materials or are encased in concrete, both of which materials are less conductive and thus slow down the heat penetration. Nevertheless, the installation of the insulation materials is usually implemented based on the traditional fire curves, such as the standard or natural fire curves. Since there is almost no regulation regarding fireproofing procedures for large open-plan steel composite structures, an investigation is planned here to examine whether a fireproofed steel structure, already designed to meet a standard fire curve, ISO 834, can resist a traveling fire.

HORIZONTALLY TRAVELING FIRE

The influence of non-uniform temperatures on structural members was investigated by Gillie and Stratford (Gillie et al., 2012). They showed that the effect of non-uniform temperature on large open areas could not be ignored. There is also an innovative method for considering traveling fire proposed by Gottfried et al. (Gottfried et al., 2010, Stern-Gottfried and Rein, 2012b, Stern-Gottfried and Rein, 2012a). Based on their model, the temperatures arising from the fire can be divided into two relative regions, near field and far field, as illustrated in Figure 1. The near field temperature (T_{nf}) refers to the region where the combustible materials are being burnt, and hence only a portion of the compartment at any time is influenced. On the other hand, the far field temperature (T_{ff}) refers to the regions that are yet to burn. It was shown in the Gottfried et al.'s work that the far field temperature is higher than the ambient temperature (T_{∞}), as a result of layers of hot gases inside the compartment. The near field size pertains to the available ventilation, and is defined as an input to the model. In addition, the near field temperature depends on the flame temperature, and thus relies on the type of fuel being consumed. Details and assumptions made in this study for modelling of traveling fire are based on what were made in (Stern-Gottfried and Rein, 2012a).



a) Concept of traveling fire b) Near field and far field temperatures

Figure 1 Illustration of traveling fire (Stern-Gottfried and Rein, 2012a, Wang et al., 2012)

CASE STUDY STRUCTURE

The traveling fire methodology is applied here to a generic four-story steel moment-resisting frame designed as a conventional office building. The building consists of four bays of 6 m in each direction with storey height of 3.2 m. An internal frame is selected here to be investigated. Beams and columns are fire-protected for the one-hour standard fire curve, ISO834, after application of the insulation material to the members. The temperature at any one point of the steel sections does not exceed 550 ºC. The one-hour fire resistance was selected in line with the French and British fire code regulations, where it is mentioned that conventional office buildings up to four stories – irrespective of the skeleton properties – shall have 30-60 minutes fire resistance (Holicky M et al., 2005). Table 1 shows the sections used for the studied frame.

Table 1 Section of all members of the generic frame

Story	External Column	Internal Column	Beam
1,2	UC356 × 368 × 153	UC356 × 368 × 202	UB457 × 152 × 67
3,4	UC356 × 254 × 107	UC356 × 368 × 153	UB457 × 152 × 67

The structure is dimensioned for load combinations of 6.5 kPa for dead load and 2.0 kPa for live load. The slab is made of normal-weight concrete with a 100 mm thickness. Grade 43 steel, with yield strength of 275 MPa and Young's modulus of 210 MPa at ambient temperature, is used for the structural analysis. On the other hand, a question can arise whether a three-dimensional model should be considered. To respond, detailed comparisons have shown that there is a close agreement between two-dimensional and three-dimensional models, such as those conducted by Usmani (Usmani, 2005). Here, in the light of these previous studies, a two-dimensional frame is selected for the analysis. Moreover, as the concrete slab has an important role in the fire resistance of a structure, its effective length is also involved in the frame, which is 1000 mm, based on ACI 318-08 (ACI318, 2008).

DEVELOPMENT OF NUMERICAL MODEL

Modelling of Structure

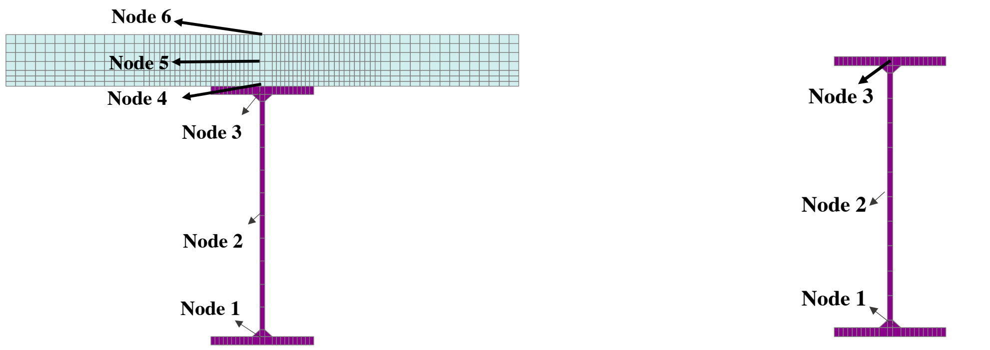
OpenSees (Mazzoni, 2007) is used here to analyse the case study structure subjected to traveling fire. For this, a series of nonlinear dynamic analyses are performed. To model the steel behaviour, a bilinear kinematic stress-strain curve is assigned to the structural elements using Steel02Thermal from the OpenSees materials library. A strain hardening modulus of 1% E is considered to model the inelastic strain range of the material. Young's modulus and yield stress were reduced depending on temperature, in accordance with the reference to Eurocode. In addition, beam-column elements in combination with fiber cross-sections are used to model the cross-sectional areas. Plasticisation of elements over the member length and cross-section is considered as well. Large displacement effects are also taken into consideration through the employment of co-rotational transformation of the geometric stiffness matrix. All connections are assumed to be ideally rigid.

Thermal Analysis

To investigate the robustness of the frame subjected to the traveling fire, the first step is to perform the cross-sectional thermal analysis. For doing this, the SAFIR program, is employed (Franssen, 2005). Since SAFIR is a fiber-based program, the variation of temperature in all of the fibers can be obtained. Results of the temperature history over time are then transferred to OpenSees for performing the structural analysis. In order to reduce the computational time, the time-temperature results of a number of fibers, along with the height of the cross-section, are selected and then cast into OpenSees as shown in Figure 2.
Figure 3 shows the variation of peak flange and web temperatures versus fire sizes of 12.5% to 100%, based on a grid size of 1500 mm. It is understood from the figure that the peak temperature occurs under the application of the 12.5% fire size, while it declines along with increasing fire size. This is because the larger fire size has a higher far field temperature but shorter duration. By contrast, the smaller fire size has a fairly low far field temperature with higher duration.

Gravitational Loading

As the traveling fire is defined in the time domain, all loads have to be defined in the time domain as well. The gravitational loads considered for the fire limit state comprise a combination of 100% of dead load and 50% of live load (BSI, 1991). For the case study here, this was a total line load of 45 kN/m. To work within the realm of time domain, the gravity loads were linearly increased during 5 seconds to reach their final values, and then, for the remainder of the analysis, times were kept unchanged.



a) Beams and slab b) Columns

Figure 2 Fibers selected from thermal analysis by using SAFIR

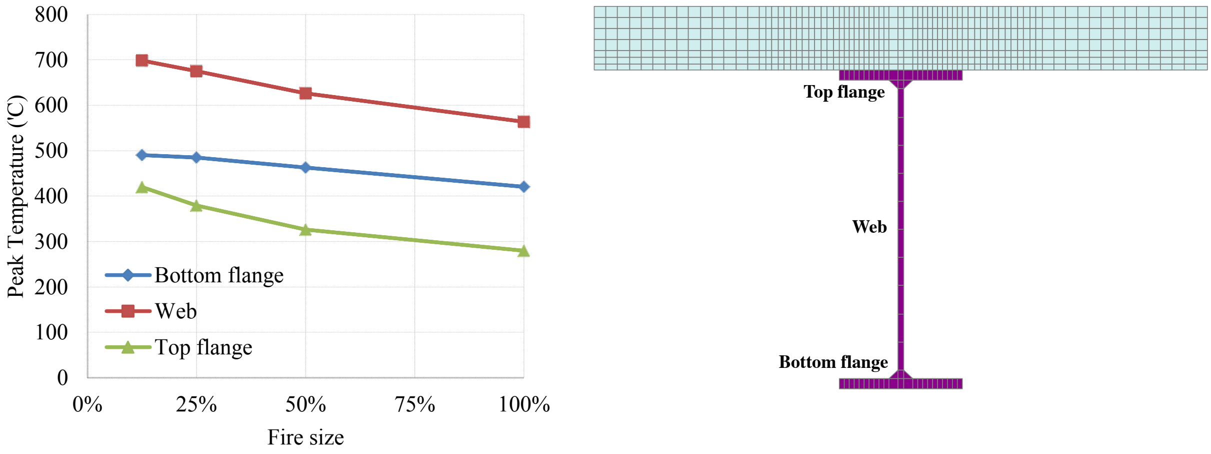


Figure 3 Variation of flange and web peak temperature versus fire size

Application of the Fire

The first story of the frame is subjected to the traveling fire as shown in Figure 4. It is assumed that the fire commences from axis A toward the other axes, while four fire sizes of 12.5%, 25%, 50% and 100% are selected for the analysis. Information for the fire sizes is shown in Table 2, including the heat release rate, the maximum total burning duration, the spread rate, and the near field temperature (which is assumed to be 1200 °C). The grid size (∆x) of 1500 mm is selected; hence each span is divided into four quarters which are supposed to provide adequate resolution for the far field temperature and the total burning time. The exterior sides of the external columns are not exposed to the fire. In addition, the top sides of the beams are not exposed to fire, as it is supposed that the top side is protected by the concrete slab.

The gas phase temperatures for beams and columns are then plotted separately, some of which are shown in Figure 5. These curves clearly show that the temperature variation in longitudinal direction of the case study plan is not constant, and thus, while some nodes are being heated up, some of them are being cooled. This variation in the temperature is important, since it may intensify the collapse risk, because different temperatures can result in different tensile forces in axially restrained beams.

RESPONSE OF THE CASE STUDY STRUCTURE TO THE FIRE SCENARIOS

Robustness Assessment

There are various failure criteria that mostly relate to one structural member, such as the thermal and the strength failure criteria mentioned in ASTM E119 (ASTME119-01, 2001). Most fire engineers would say that a steel member has failed when its temperature goes beyond 550 ºC. While using this failure criterion can provide much simplicity for controlling the stability of a member, its use is arguable for a building structure such as the case study here. Indeed, it is challenging to say that failure of a member – even though it is accepted that for instance at 550 ºC the member has failed – will automatically result in a chain of successive failures in other members. This relates to the definition of progressive collapse, where the robustness of a structure to resist a localized failure is scrutinized. After failure of a structural member, if an alternative load path is found, i.e. the load can be re-distributed to other members, the structural integrity is maintained. Mostly, when two successive columns fail, the load cannot be redistributed, and hence, progressive collapse occurs. In following part, response of the structure to fire sizes of 50% and 100% are not shown since they do not lead to instability of any structural member.

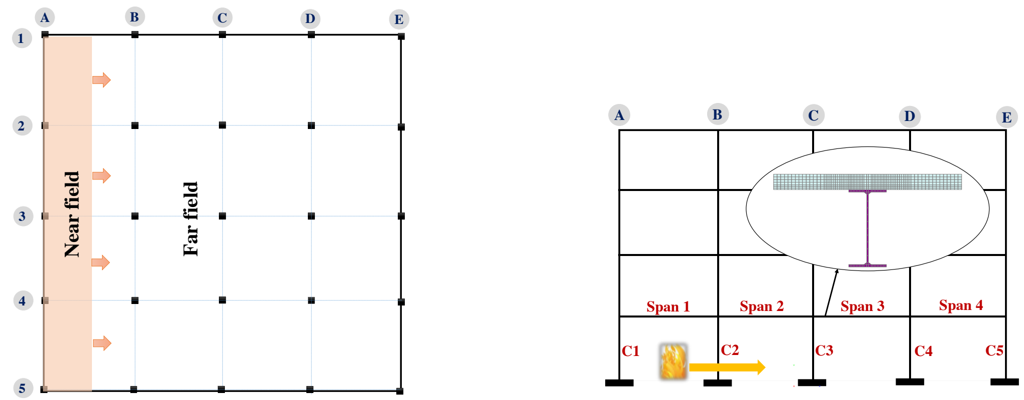


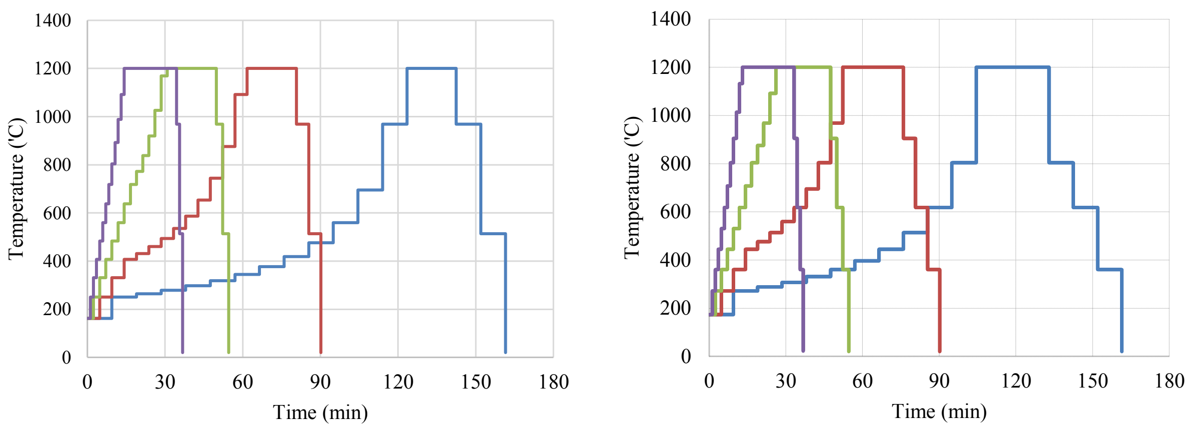
Figure 4 Application of the traveling fire to the case study

Table 2 The size range of the fire

Fire size (%)	A_f **(m²)**	$\dot{Q}$ **(MW)**	t^*_{total} **(min)**	s **(m/min)**	T_{nf} **(°C)**
12.50	72	36	161.50	0.16	1200
25.00	144	72	90.25	0.32	1200
50.00	288	144	54.62	0.63	1200
100.0	576	288	36.81	1.26	1200

Fire Size of 12.5%

Figure 6 shows the response of the structure under the application of the 12.5% fire size. As is seen in Figure 6a (the vertical displacements at the top side of the columns), no global failure occurs. However, some transient instabilities are experienced during the analysis, mainly due to the buckling of some columns. The global stability of the structure can be justified because of the structure's capacity to redistribute loads from the failed elements to other elements. This can better be understood in Figure 6b, where the axial loads of the columns are shown. As is seen, larger distribution of the loads occurs when the fire travels across the story in such a way that columns 2 and 4 tolerate much more axial loads than the other columns. It is observed that, as the temperature rises, column 1 experiences more compression force, resulting in a decrease in the axial load of column 2. However, as the temperature of column 2 increases, its axial force increases while the axial force of column 1 decreases. This state continues until column 2 buckles at 49.18 min, when the temperature is 601.61 ºC (web temperature) and thus, its load is relocated to column 3. Furthermore, when the near field temperature has passed beyond column 2, which means the temperature decreases, it can again carry more axial load. Then, when at 81.71 min (web temperature of 597.97 ºC) column 3 buckles, its load is transferred to the adjacent columns 2 and 4

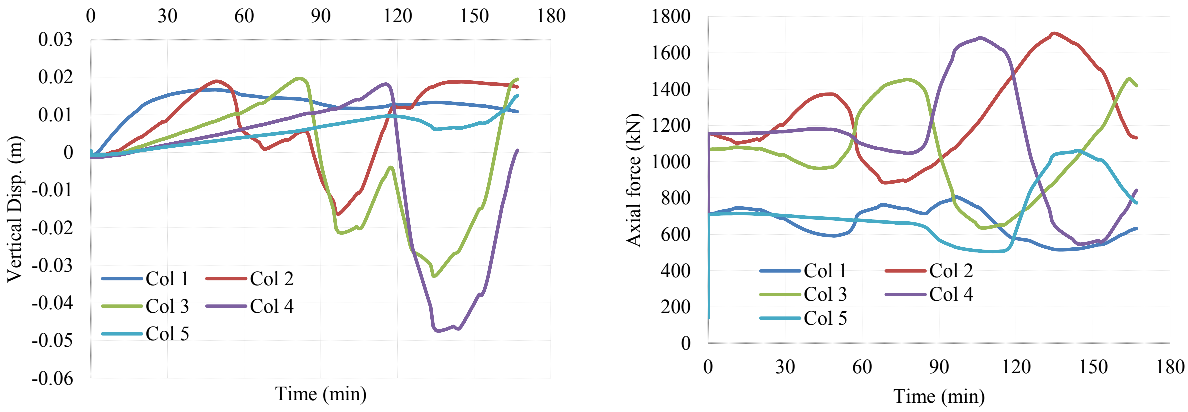


a) First quarter of span 4

b) Column 4

12.50% 25% 50% 100%

Figure 5 Gas phase temperatures

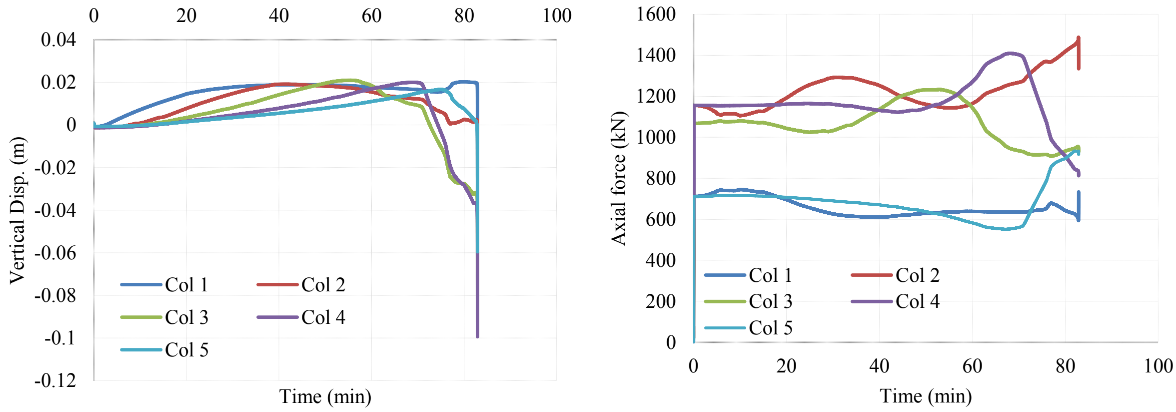


a) Vertical displacement b) Axial `

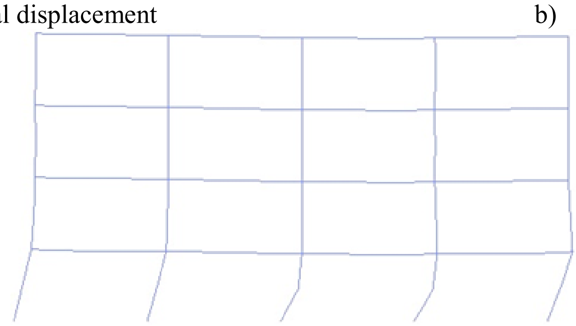
Figure 6. Response of the structure under the traveling fire size of 12.5%

Fire Size of 25%

The response of the structure under the 25% fire size is shown in Figure 7. As seen in Figure 7a, the structure collapses when column 5 buckles at 74.95 min (temperature 458.66 ºC). It can also be seen that the compression force in column 1 increases over the heating phase. However, when column 2 is exposed to higher temperatures, its axial force increases while the axial force of column 1 decreases. This continues until buckling of column 2 occurs at 41.33 min (web temperature of 602.8 ºC). At this moment, column 3 carries the gravitational load that is supposed to be carried by column 2. This continues until the buckling of column 3 at 55.25 (web temperature of 633.11ºC). As with the previous column, the load is transferred to columns 2 and 4. This structural behaviour continues until the bucking of column 5, at which point (based on the temperature of adjacent columns) there is no sufficient alternate path to transfer the loads carried by failed columns. The total collapse occurs at this time, as shown in Figure 7c.



a) Vertical displacement b) Axial force



c) Deformed frame after the application of the fire size of 25% (t= 74.95 min)

Figure 7. Response of the structure under the traveling fire size of 25%

GENERAL CONCLUSION FROM THE RESULTS

As a general conclusion, it is evident that the stability of the frame is largely dependent on the fire size, a point that could not be understood prior to the investigation performed here. The structure remains stable in all of the fire scenarios except for the 25% fire. This conclusion is in contradiction with traditional belief, which deems that the assumption of a larger-scale fire in a compartment is a more-conservative assumption, and one that can thus increase the safety margin. The results of the investigation performed here become more noteworthy when reminded that the structure has already been designed and fireproofed to resist the one-hour standard fire curve, ISO 834. It is also worth noting that while the fire size decreases, the maximum gas temperature increases, as shown in Figure 8. In terms of the structural engineering view, nevertheless, the fire size with the maximum temperature does not necessarily bring about the most critical situation, as was investigated here. In other words, the fire size and the method of application of fire to a large open area can lead to different results, ranging from no damage to total collapse. As there is a low possibility of a uniform fire in a large compartment, it is essential to consider the results of a traveling fire in order to arrive at more accurate results. The results of the investigation performed here are more noteworthy when it is re-stated that the building had already been fireproofed, leading to the presumption that it would remain stable under the possible fires. As there are almost no fire regulations standardized for large compartments, particularly as regards fireproofing, more investigations are thus required to arrive at a better understanding of the application of traveling fire to large compartments.

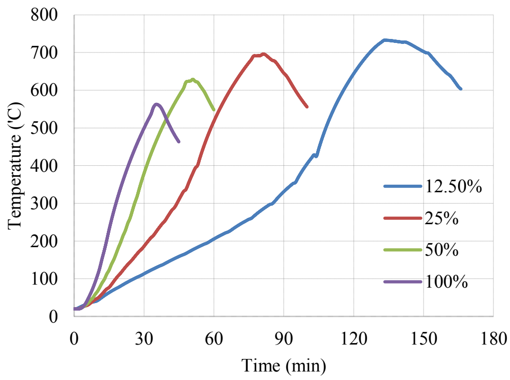


Figure 8. Web temperature of column 4 exposed to various traveling fire sizes

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