ANALYSES OF THE EFFECTS OF FIRE TYPE ON THE RESPONSE OF STEEL STRUCTURES SUBJECTED TO ACCIDENTAL LOADINGS

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ABSTRACT
Structural fire research is fundamental to the safety of buildings, installations and personnel. A good number of fire design of structures is based on the traditional prescriptive approach. However, performance based fire design of structures has also been extensively developed and used by researchers to understand the actual behaviour of structures that are subjected to accidental loadings, such as fire. Most of these fire models used are either based on relevant assumptions that do not actually represent the scenarios under study, or are derived from theoretical and empirical data. These assumptions may not exactly characterise the true behaviour of structures when subjected to such accidental loadings. Recently, there have been real fire trials undertaken to bridge this gap, such as the Cardington fire test, UK and the Health and Safety Laboratory fire test, UK as distinct from the standard laboratory fire tests. It is therefore the intent of this paper to investigate the response of steel-framed structures under thermo-mechanical actions using a variety of fire loadings. To allow for this comparison the frame structure was taken from the literature and the model was analysed using ABAQUS analysis code and then validated against existing data. The most critical fire load was identified and subsequently used to study the performance of steel-framed structures. It is discovered that using the appropriate fire loads, structures can be most efficiently and economically designed. Findings also reveal that if the correct fire load for a given scenario is not properly defined the results from the calculation can be seriously wrong. Moreover, a concise description of the methodology used to derive one of the fire models adopted in the study is also presented.

KEYWORDS: Accidental loading, elevated temperature, fire behaviour, fire type, performance based fire design.

1 INTRODUCTION

The behaviour of steel structures subjected to fire loadings forms a part of the fundamental subject of topical research projects. Fire design of structures has been traditionally evaluated using deterministic techniques where the fire loadings are computed based on simple models described in Eurocode 3, Part 1-2 (2005). Currently, a new technique that can exclusively incorporate the likely uncertainties associated with heat loads and responses of structural members has begun to gain much popularity. This probabilistic method that is often termed "the stochastic technique" has been used by a few researchers that study the reliability of structures (Eamon and Jensen, 2012, Hietaniemi, 2007). However, this new development in research seems to rely more on valid numerical predictions to circumvent the enormous costs that are necessarily associated with experimental works. Recently, most researchers have continued to criticize codified design rules based on the standard time-temperature curve, maintaining that such rules repress innovative fire safety protocols that are likely to be cheaper and more effective. Moreover, in recent times the trend of fire design has begun to move from the long-established prescriptive approach to the more robust performance based approach.

It is remarkable, however, to ascertain the fundamental response of structures subjected to different fire types. Different fire models are adopted in this study to determine the response of structures to different fire loadings. This is required because the moment a fire has begun, it is often difficult to predict its behaviour (Drysdale, 1999, Wighus, 1994, Stern-Gottfried et al., 2010). The response of steel structures to fire attack will depend on the behaviour of the fires, which obviously can be described by the fire models. Donegan (1991) emphasized that the principal mechanism causing collapse in a fire is the release of potential energy when the strength and stability of the structure is reduced by the effects of the fire. It is recognized that the various failure types that occur in steel structures do not occur simultaneously, however more than one phenomenon will be involved until the structure reaches its ultimate limit state (Paik and Thayamballi, 2002).

2 MATERIAL PROPERTIES AT ELEVATED TEMPERATURES

2.1 Thermal and mechanical properties

The material properties of steel at elevated temperatures are required to model the behaviour of structures in fire conditions. The loss of strength and stiffness of structural materials at elevated temperature introduces material nonlinearities into the model thus leading to a more complex analysis. In assessing the performance of structures at elevated temperatures, both mechanical and thermal properties are required to accurately predict their behaviour. Therefore, the thermal and the mechanical properties implemented in this study are based on those adopted by Ufuah et *al*. (2013). The elevated temperature stress-strain curves were developed using the model shown in Fig. 1 in accordance with Eurocode 3, Part 1-2 (2005). Fire limit state design and the modelling procedure implemented in the study are fully described in Ufuah et *al*. (2013). Generally, in the limit state definition safety of structures is guaranteed according to *Eq. (1)* as defined in (Lawson and Newman, 1996):

$$\gamma_f E \leq \frac{R}{\gamma_M} \tag{1}$$

where γ_f and γ_M are the partial safety factors on actions (E) and materials (R) respectively.

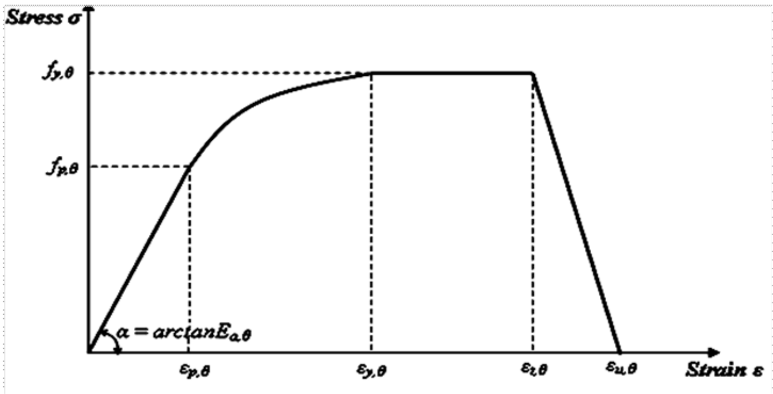

Fig. 1. Stress-strain model for carbon steel

The equilibrium equation and the equivalent thermo-mechanical constitutive model used to determine the strength and behaviour of the beam model are given as follows (Ufuah et al., 2013):

$$\sigma_i = E\varepsilon_{ii} \quad (2)$$

$$\{\Delta\sigma\} = \{D^{ep}\}[B]\{\Delta U_e\} - \{C^{th}\}[M]\{\Delta T_e\} \quad (3)$$

where σ_i is a stress tensor, E is the flexural stiffness, ε_i is strain, $\Delta\sigma$ is the incremental stress, D^{ep} is the elasto-plastic stiffness, B is strain-displacement relation, U_e is nodal displacement, C^{th} is thermal stiffness, M is temperature shape function and ΔT_e is nodal incremental temperature.

3 FIRE LOAD MODELS

The various fire models adopted in this study include those of Bailey et *al.* (1996) fire model, the Health and Safety Laboratory fire model (Thyer et al., 2008), the parametric fire model and the models developed by Ufuah and Bailey (2011). These models were separately implemented to enable the calculation of deformation and axial capacities of structural assemblies. All of these fire models incorporate both the heating and the cooling regimes as depicted in Figs. 2 - 6. For the beams Nos. 1 and 2, the Bailey et al. [11] fire was applied as a non-travelling fire that was assumed to extinguish after about 216 minutes. In the case of the two-dimensional framed structure, the fires were assumed to start in zone I and then migrate seemingly to other zones with time.

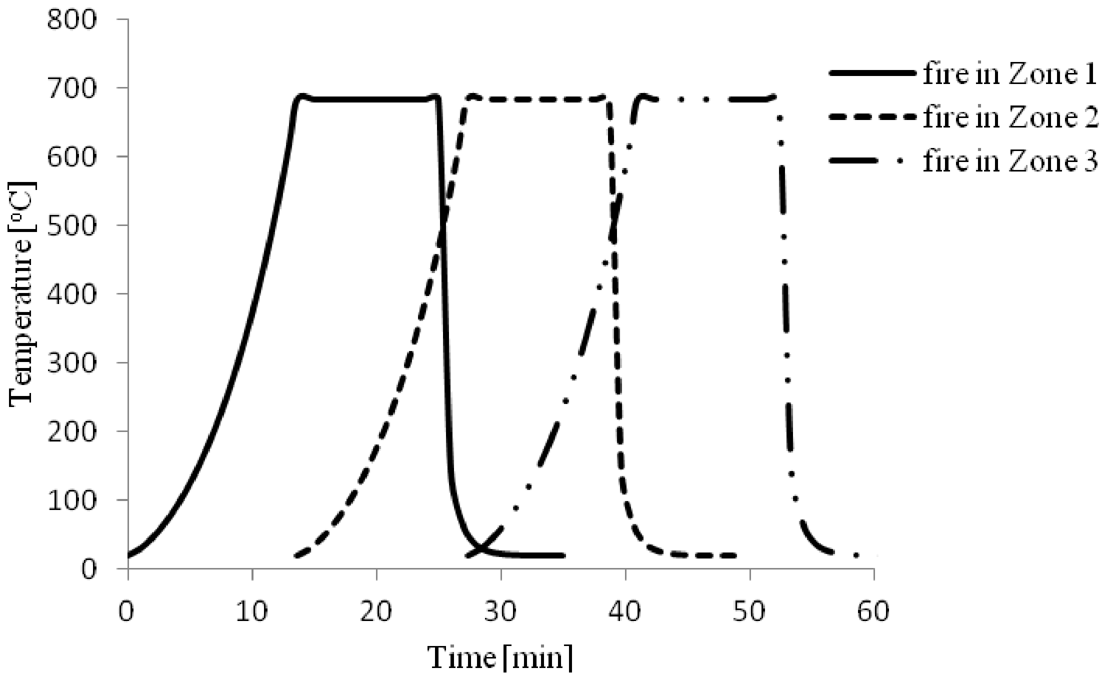

Fig. 2. Fully developed fire applied as travelling fires

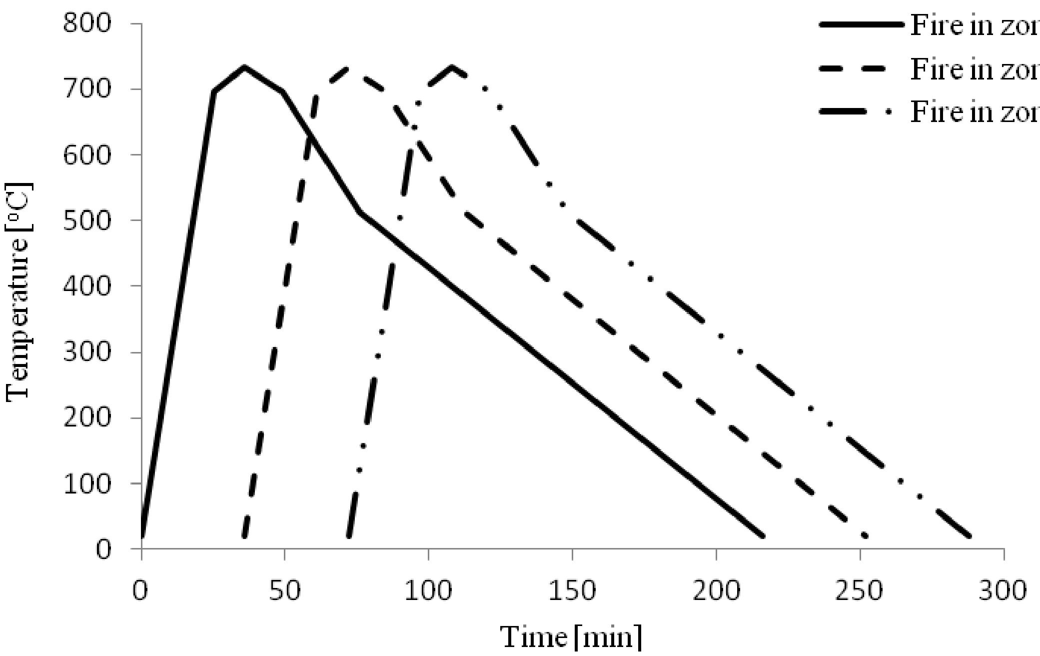

Fig. 3. Time-temperature curves used by Bailey et *al.* [11]

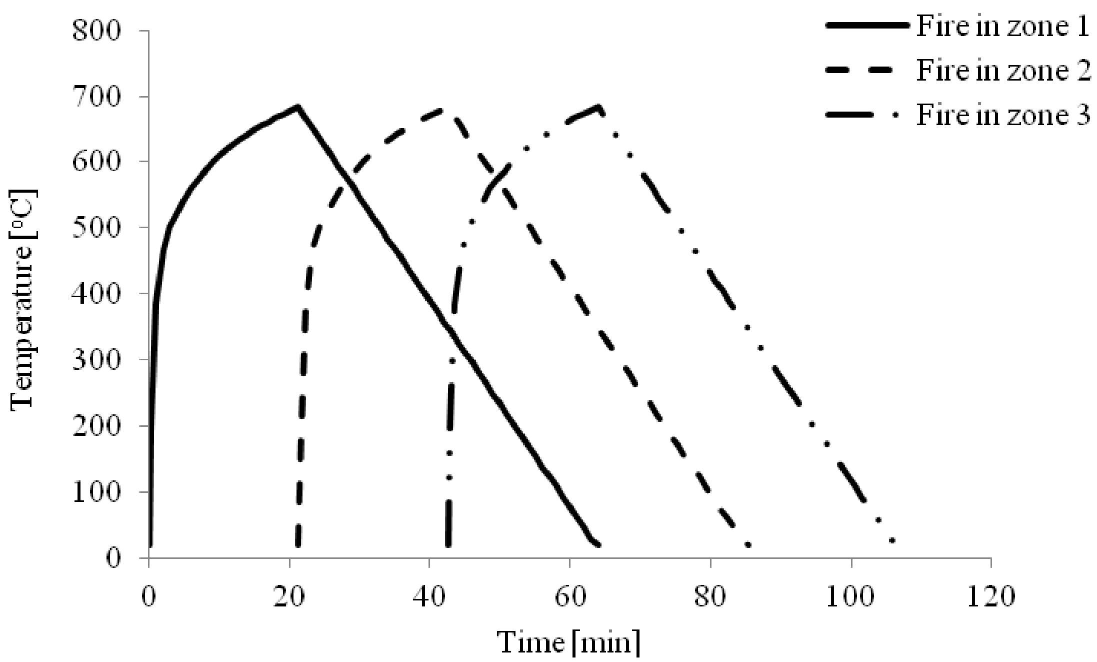

Fig. 4. Parametric fire applied as travelling fires

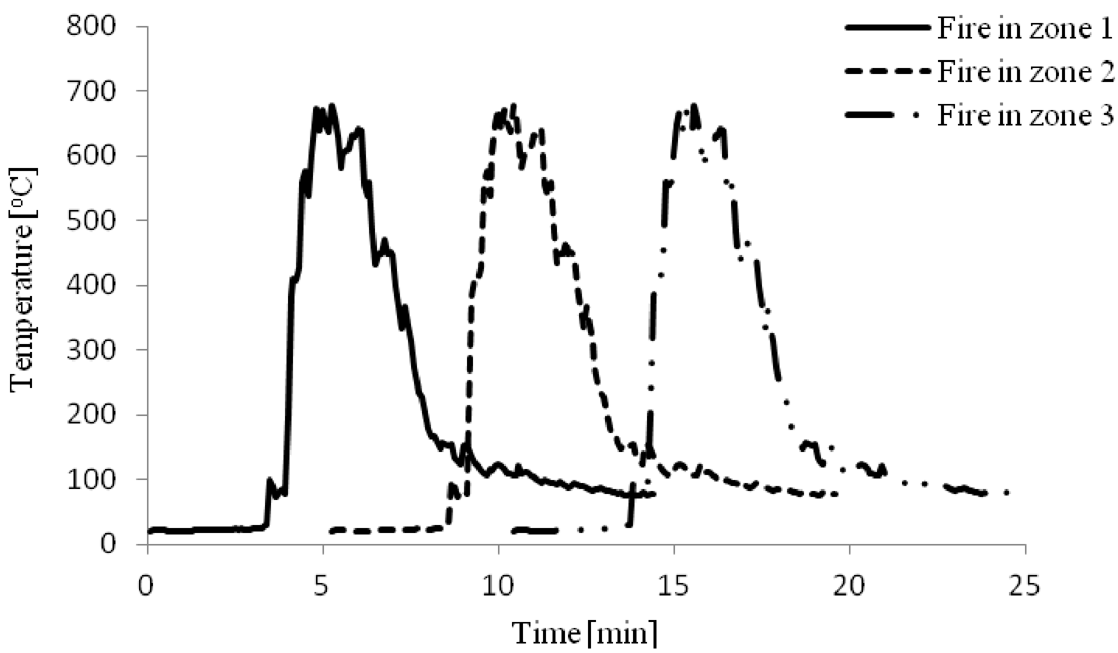

Fig. 5. HSL measured fire applied as localized travelling fires [12]

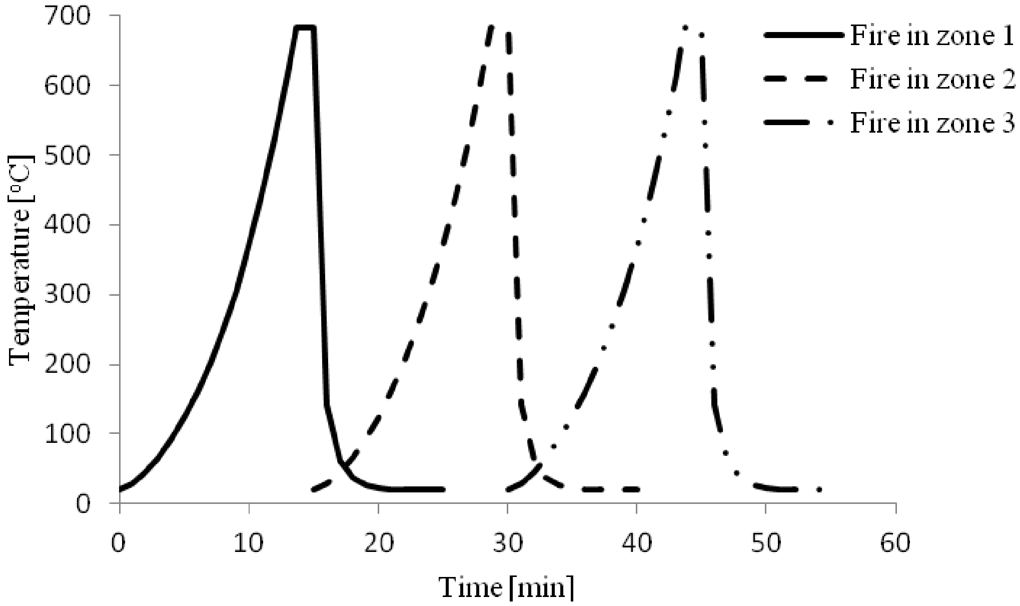

Fig. 6. Localized travelling fires

3.1 Development of localized and fully developed fire models

The localized and fully developed fire models are modelled to represent the fire loading conditions under which the deck at the Health and Safety Laboratory (HSL), Buxton, UK (Thyer et al., 2008) was tested. The flame height correlation of Heskestad (2002) was adopted to determine the fire loads. The power law relation developed by Heskestad (2001, 2007) was used to develop the fire models. This yields a convective fire growth coefficient of 0.0469KW/s^2. The power law implemented in this analysis is a t-squared fire model defined according to *Eq. (4).*

$$Q = \alpha_c (t - t_0)^2 \tag{4}$$

where $_c$ is the convective fire growth coefficient,
Q is the heat release rate, t is time from ignition and t_o represents the virtual time at origin.

A typical design fire curve comprises three stages of the fire process, namely the growth phase, the steady-state phase and the decay phase. These phases can be characterized in terms of the heat release rate using the power law relation. The maximum mass burning rate for spill fires is estimated as one-fifth of the maximum mass burning rate for pool fires (Gottuck and White, 2002). Notably, the model is based on the physical measurements in the trials conducted at the HSL in which 200 litres of fuel was spilled onto the deck (Thyer et al., 2008). Using the correlation of *Eq. (5)* as proposed by Gottuck and White (2002), the surface area of fuel available for burning becomes 72m^2

$$A_p = 1.4V; \quad \text{for pool} < 95 \text{ litres}$$
$$= 3.6V; \quad \text{for pool} > 95 \text{ litres} \qquad (5)$$

where A_p is pool fire area (m^2), V is spilled volume (litres)

However, the properties of most hydrocarbon fuels can be found in the handbook of society of fire protection engineers (SFPE) (Babrauskas, 2002). Given that the fuel is spilled onto the deck without being constrained, the heat release rate is accordingly computed using *Eqs. (6)* and *(7)* (Gottuck and White, 2002).

$$\dot{Q}_{max} = \frac{1}{\,}\dot{m}''A\Delta h_c \qquad (6)$$

$$\dot{m}'' = \dot{m}''_{\infty}[1 - \exp(-k\beta D)] \qquad (7)$$

Where $\dot{m}''$ is the mass burning rate Δh_c is the lower heat of combustion; A is pool fire area; $\dot{Q}_{max}$ is the maximum heat release rate; $\dot{m}''_{\infty}$ is the maximum burning rate and k is the mean beam length corrector-flame attenuation coefficient product.

To evaluate the period in which the fire remains steady for the fully developed phase, the model of *Eq. (4)* was integrated over the requisite time intervals and then equated to the total heat release rate as follows:

$$\int_0^{t_1} \alpha_c t^2 dt + \int_{t_1}^{t_2} \alpha_c t_1^2 dt + \int_{t_2}^{\infty} \alpha_c t_1^2 e^{-\left(\frac{t-t_2}{\mathrm{E}}\right)} dt = \dot{m}''A\Delta h_r \qquad (8)$$

Upon integration and simplification of *Eq. (8)* the following expression was obtained:

$$t_2 = \frac{\dot{m}''A\Delta h_c}{\alpha_c t_1^2} + \frac{2}{3}t_1 - \qquad (9)$$

where is the exponential fire decay time constant, t_1 is initial time to reach maximum temperature and t_2 is the final time at maximum temperature before decay starts.

Having established the heat release rate (HRR) of the pool fire in terms of the t-squared fire model, the temperature of fire plume can now be computed using *Eqs. (10)* and *(11)*. This requires the solution of the three conservation equations for continuity, momentum and buoyancy (Heskestad, 2002).

$$\Delta T_o = 9.1\left(\frac{T_\infty}{g c_p^2 X_\infty^2}\right)^{1/3} Q_c^{2/3}\ z - z_o)^{-5/3} \quad (10)$$

where ΔT_0 = incremental change in temperature, T_∞= ambient temperature, c = specific heat of air, ρ_∞=ambient density, z = height above top of combustible, z_0 = virtual source location, D = diameter of fire source, g = acceleration of gravity and Q_r = convective component of the heat release rate.

$$\frac{z_o}{D} = -1.02 \quad 0.083\frac{Q^{2/5}}{5} \quad (11)$$

4 MODELLING PROCEDURE

4.1 Benchmark study: Bailey et *al.* (1996) model

For the purpose of assessing the suitability of ABAQUS commercial software in this study, structural components such as beams subjected to thermal and mechanical loadings were selected from a previous study. This concept was adopted because of the limitation and cost in carrying out the experimental programme. Generally, the failures observed after a fire with gross yielding of the component parts of redundant structures actually occur during the cooling phase of the fire. To permit a basis for comparison, the benchmark models taken from the literature (Bailey et al. 1996) were modelled in ABAQUS. The models are simple beams having their material and geometric characteristics described in Table 1. The uniformly distributed load in each case corresponds to the load level shown in Table 1, as defined in Eurocode 3, Part 1-2 (2005). The top flanges of the beams were heated such that they experienced 50% of the web and bottom flange temperatures.

*Table 1.*Matrix of benchmark beam models implemented in the study

Beam No.	Effective span (m)	Length (m)	Yield, Y_{20}(MPa)	Modulus, E_{20}(GPa)	Section sizing	Load level	Distributed load(KN/m)	Support type
1	6	6	308	210	356x171x51UB	0.5	30.6	S.S
2	6	6	308	210	356x171x51UB	0.22	6.89	S.S

S.S = simply supported

4.2 Numerical modelling of two dimensional steel-framed structures

The two-dimensional steel-framed structure previously analysed by Bailey et *al.* (1996) is shown in Fig. 7. It comprises internal beams of sizes 610 x 229 x 101UB and columns of sizes 254 x 254 x 132UC, with the connections assumed to be rigid and the entire structural assembly restrained horizontally. The beams were subjected to a load level of 0.6 whereas the internal columns in the fire affected region were subjected to a load level of 0.45. The external columns were loaded with point loads of 475.2 kN each while the internal columns were made to resist 950.4 kN as shown in Fig. 7. The columns were assumed to remain at room temperature and the beams under consideration were heated with the top flanges having 80% of the web and bottom flange temperatures. The elevated temperature mechanical properties were characterised using the model of Fig. 1 as described in Eurocode 3, part 1-2 (2005). The room temperature yield strength and modulus of elasticity used were 275 MPa and 210GPa respectively. The thermal properties adopted in the study are as described in Eurocode 3, Part 1-2 (2005).

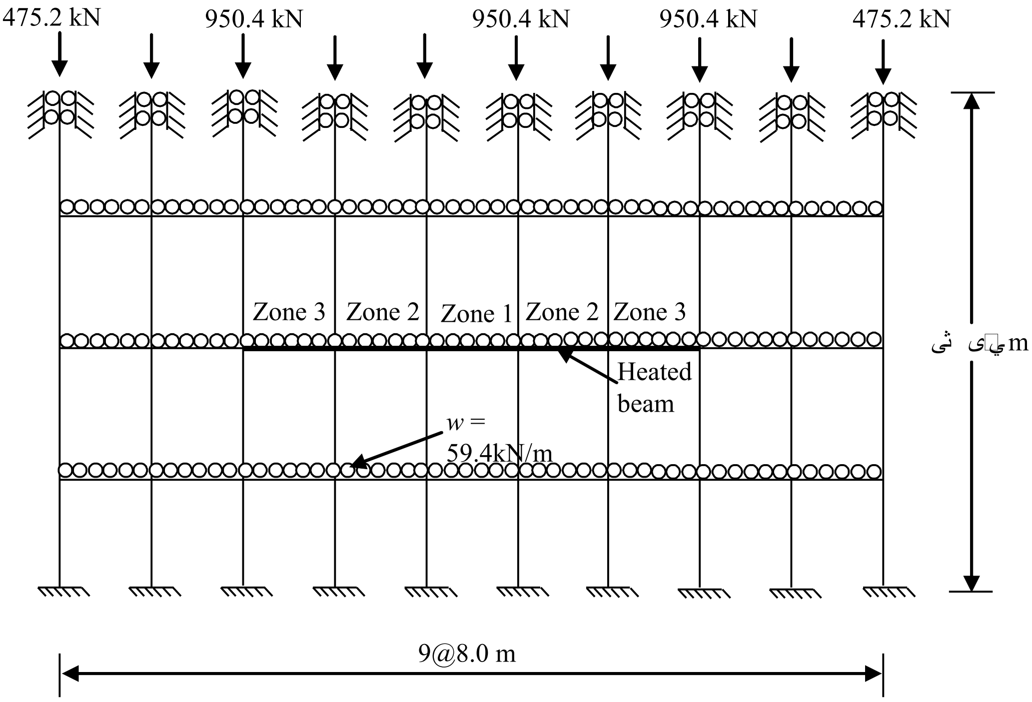

Fig. 7. Two-dimensional frame structure under travelling fire

5 RESULTS AND DISCUSSION

Firstly, the results of the benchmark study are presented in Figs. 8 and 9, followed by those of the steel-framed structure shown in Figs.10 - 12. The results of the mid-span displacement and the axial forces for the beam in the steel-framed structure reveal that fire type can affect the way structures would behave under fire conditions. The Bailey et *al*. (1996) fire seems to have the most significant effect on the performance of the beam. This occurred within the cooling regime at the end of the fire for which the axial capacity was reduced following the plastic deformation experienced in the beam section as seen in Fig. 12. The deformed shape of the frame structure is shown in Fig. 13

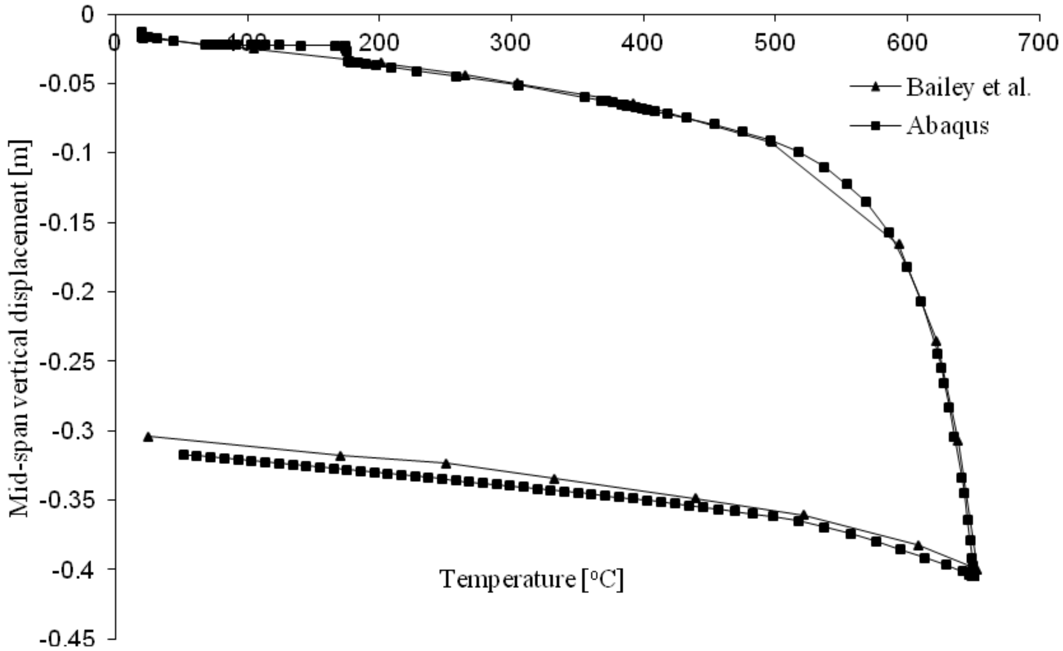

Fig. 8. Temperature-displacement curve for beam No. 1

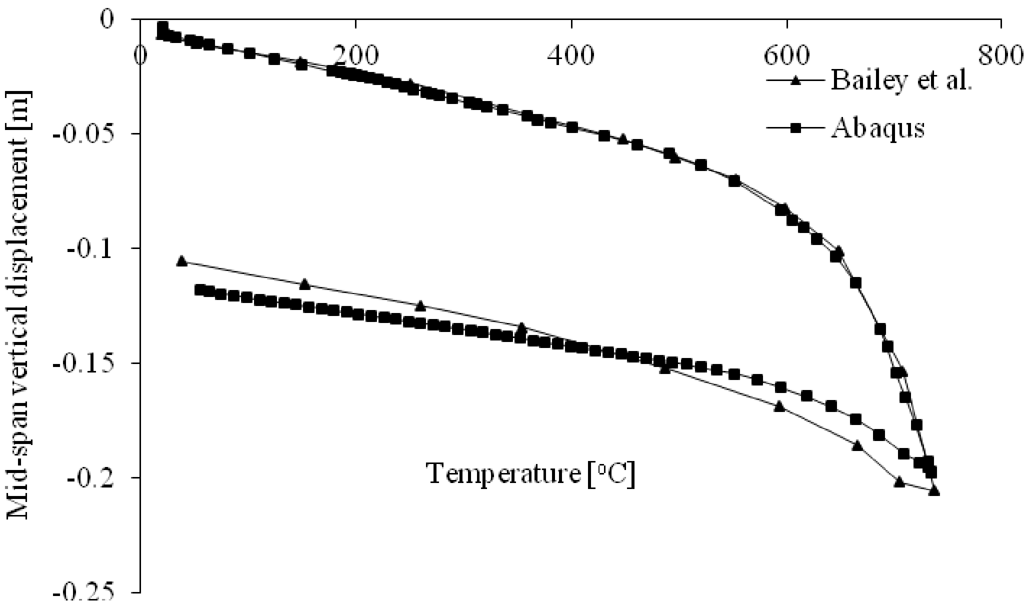

Fig. 8. Temperature-displacement curve for beam No. 2

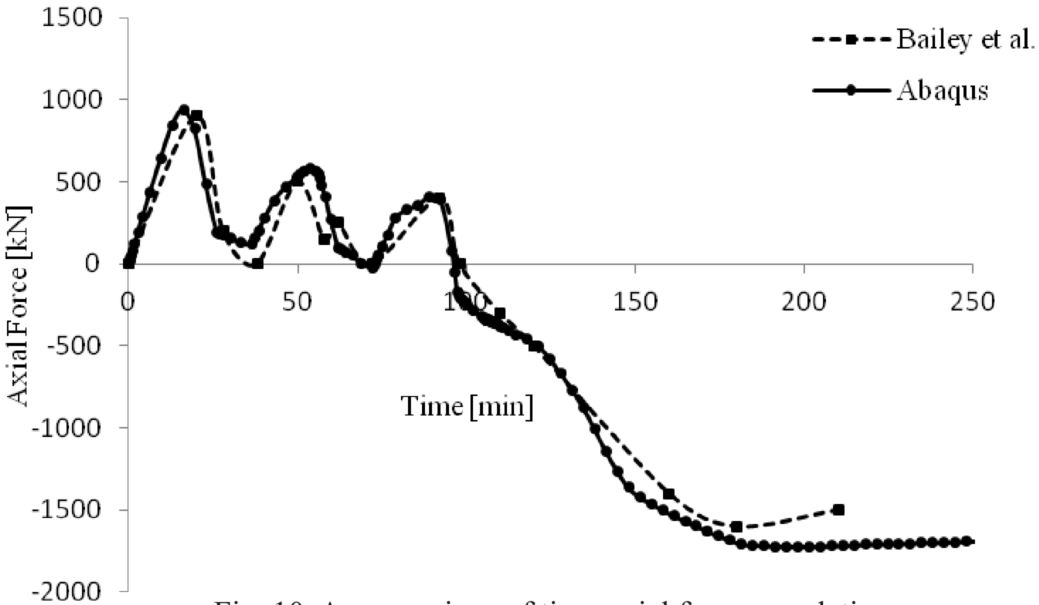

Fig. 10. A comparison of time-axial force correlation

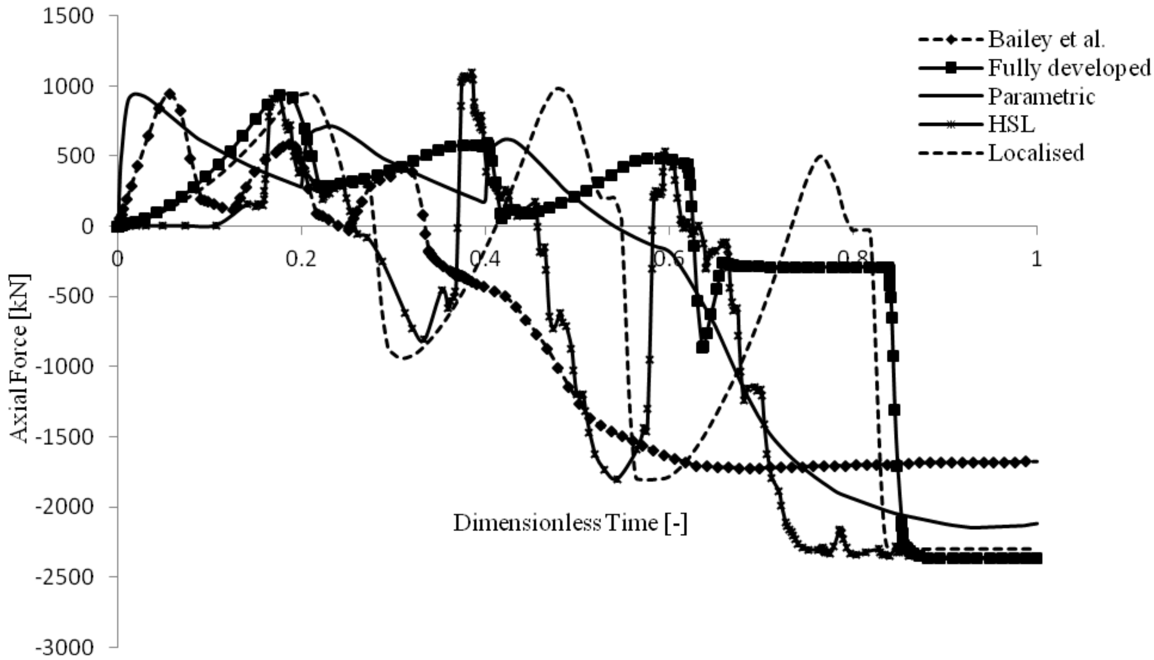

Fig. 11. Dimensionless time-axial force correlation for all fire types

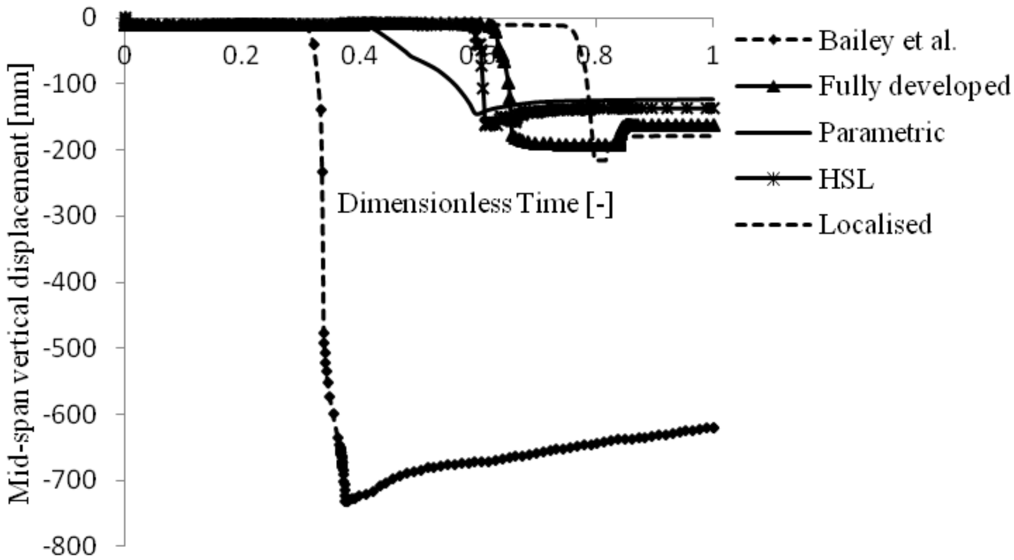

Fig. 12. Dimensionless time-displacement correlation for all fire types

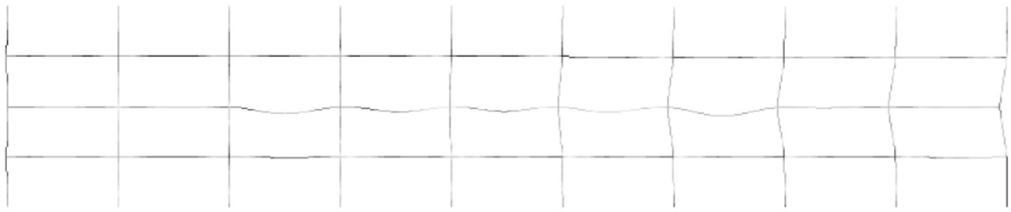

Fig. 13. The deformed shape of the two-dimensional frame structure under spreading fire

6 CONCLUSION

It is important, however, to ascertain the fundamental response of structures subjected to different fire types. Different fire models were adopted in this study to determine the response of structures to different fire loadings. This is required because the instant a fire has begun, it is often difficult to predict its behaviour, particularly for the case of a spreading fire. The response of steel structures to fire attack will depend on the behaviour of the fires, which apparently can be described by the fire models. Incidentally, the performance of a structure subjected to the standard fire load will be different from the case of a natural fire. Donegan (1991) emphasized that the principal mechanism causing collapse in a fire is the release of potential energy when the strength and stability of the structure is reduced by the effects of the fire. The fire limit state design and the modelling procedure implemented in the current study are fully described in Ufuah et *al*. (2013). The elevated temperature stress-strain curves were developed in accordance with Eurocode 3, Part 1-2 (2005).

To permit a basis for comparison, the benchmark models taken from the literature, Bailey et *al*. (1996) were modelled in ABAQUS as a scoping study. The results are presented in Figs. 8 and 9. The results of the mid-span displacement and the axial forces for the beam in the steel-framed structure, shown in Figs. 10

to 12, reveal that fire type can affect the way structures would behave under fire conditions. The Bailey et *al.* (1996) fire seems to have the most significant effect on the performance of the beam.

All the fire types implemented in the study except the Bailey et *al.* (1996) fire resulted in approximately the same residual deformation with the parametric fire producing the least residual deformation. The parametric, fully developed and Bailey et *al.* (1996) fires only started to induce tensile stresses in the beam towards the end of the fire. It is also likely that the HSL fire may have structural implication on the integrity of buildings, particularly in the cooling regime of a fire. It is therefore paramount to consider fire type when considering fire design of structures. Since the deformation capacity of the beams is seen to be much reduced under Bailey et *al.* (1996) fire, it is then indicated that the interplay temperature between two successive regions, as the fire migrates from one zone to another, can be very critical. It should be noted, however, that the interplay temperature in the Bailey et *al.* (1996) fire is 630^0C whereas in the HSL fire, parametric fire, localised fire and fully developed fire, the interplay temperatures are respectively 120^0C, 625^0C, 60^0C and 500^0C. This signifies that most of the vertical displacement of the deck under the HSL and localised travelling fires will be recovered in each stage as the fire moves from one zone to another.

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