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Temperature-time analysis for steel structures under fire conditions

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Abstract. The objective of the paper is to present a method whereby the time required for a steel structure to sustain the effects of a prescribed temperature rise according to real fire curves can be calculated. The method is divided into two parts. The first part deals with the post-yield behaviour of steel structures at elevated temperatures. It takes into account the variation of the properties of steel material with temperature in an incremental elastoplastic analysis so that the safety factor of the structure under certain fire conditions can be assessed. The second part deals with the heat transfer problem of bare steel members in real fire. Factors affecting the heat transfer process are examined and a model for predicting the temperature variation with time under real fire conditions is proposed. This model results in more accurate temperature predictions for steel members than those obtained from previously adopted model.

Key words: fire; temperature prediction; elastoplastic analysis; steel.

1. Introduction

In recent years, the use of bare steel members for building construction has become increasingly popular. Multi-storey carparks and single-storey industrial buildings are common examples of structures composed of unprotected steel members in Australia. This trend will continue, particularly when more technical information related to the behaviour of steelwork under fire conditions is made available to engineers. When designing a steel structure under fire conditions, the assessment of the adequacy of the structure's integrity may be based on a stated period of time within which the members of the structure are exposed to the fire without collapsing. It is therefore necessary for engineers to be able to carry out a fire endurance analysis so that the behaviour of the structure, where the temperature changes with time, can be traced. This is particularly important when performance based fire design codes are gaining increasing attention.

At present, most design codes for fire engineering design provide guidelines or design rules for performing fire endurance analysis (CIB 1983, Standards Australia 1990, BSI 1995). However, the level of sophistication in performing the fire endurance analysis depends on the state of engineering knowledge and design tools available to engineers but is in general limited to simple beams and isolated sub-assemblies. Assessing local fire effects on global structural behaviour is important since the cooler part of the structure may contribute a substantial strength

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resistance to the fire, particularly during the early stage of the fire. Thus, the fire endurance, when considered in the context of a complete structure, may be much different from that when only isolated sub-assemblies of the structure are considered. The analysis of a complete structure under fire conditions is a complex one. This complexity increases when an incremental plastic method is adopted for the analysis as the interaction between imposed design loads and thermal loads becomes extremely difficult to handle when temperature in the members increases. Little information on the treatment of interaction between imposed design loads and thermal loads in an incremental analysis is available. A few studies on the numerical analysis of complete structures under fire conditions have been carried out in the past (Iding and Bresler 1984, Schleich, *et al.* 1985). These studies are mostly based on the Finite Element Method which requires substantial effort in dealing with the discretization of the structural elements and the numerical iteration required to maintain equilibrium.

An alternative approach for calculating the temperature effects on structures composed of unprotected steel members has been proposed by Crozier and Wong (1994). This approach is based on an elastoplastic analysis of steel frames at elevated temperatures where the material yield of elements is accounted for in terms of the stress resultants. In this study, a numerical model for predicting the limiting temperature at which the structure collapses has been established. Geometric nonlinearity as a result of significant lateral external loads, such as wind loads, is not included in the analysis. This assumption is reasonable since the probability of both the design wind loads and fire occurring at the same time is remote. The advantage of this approach is that numerical iteration procedures, such as Newton-Raphson Method, are not required while equilibrium of structure is always maintained.

In this paper, an extension of this work to include the time effects on the temperature of structural elements in a steel frame under simulated real fire conditions is presented. The paper is divided into two parts. Firstly, a method to trace the behaviour of steel structures under real fire conditions is described. The plastic hinge concept in an incremental elastoplastic analysis (Wong 1991) is used to trace the inelastic behaviour of the structure at elevated temperatures. The collapse scenario is based on a specified temperature distribution pattern for local fires which occur in the structure. Details of the treatment of the interaction between internal actions due to temperature rise and imposed design loads are given. Secondly, a heat transfer model to predict steel element temperature is presented so that the time required to achieve a specified temperature distribution in the structure can be calculated. Factors affecting the radiation and convection mechanisms of a fire are examined in detail. This heat transfer model gives a more accurate temperature prediction for steel elements than the one generally adopted by other researchers (Barthelemy 1976, Smith and Stirland 1983). The new model is calibrated against results from simulated real fire experiments.

2. Mechanical properties of steel at elevated temperatures

The mechanical properties of steel at elevated temperatures are specified mainly in terms of the stress-strain relationship, the modulus of elasticity, the yield stress and the coefficient of thermal expansion. The effects of creep are not considered in the present study. Studies on the mechanical properties of steel under increasing temperature have been carried out and verified through experiments by a number of researchers (Kirby and Preston 1988, Lie 1992). Various expressions have been proposed to simulate the variation of mechanical properties of steel with temperature

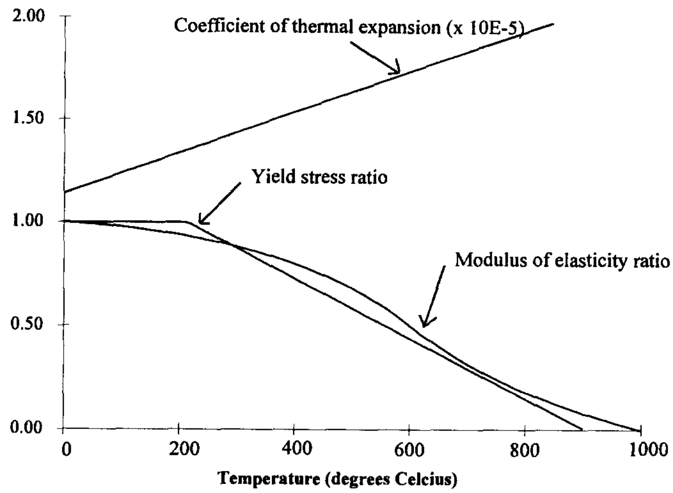

Fig. 1 Mechanical properties of steel at elevated temperatures.

(Brockenbrough 1970, ECCS 1983, Chiou, *et al.* 1991). A detailed description of the variation of mechanical properties of different materials with temperature can be found in Lie (1992).

The mechanical properties of steel at elevated temperatures adopted in the present study are described below and shown in Fig. 1.

2.1. Modulus of elasticity

The stress-strain relationships of steel at high temperatures are related to its value of modulus of elasticity and may be expressed in a form similar to the Ramberg-Osgood formula (Olawale and Plank 1988, Saab and Nethercot 1991). Expressions derived in this manner result in a highly nonlinear problem for analysis. A simplified but practical approach is to approximate each of the nonlinear elevated temperature stress-strain relationships as a bilinear one. This approach has been adopted in most design guidelines or codes (ECCS 1983, Standards Australia 1990, CTICM 1982). In this paper, an equation given by CTICM (1982) for the modulus of elasticity is used and is given as

$$\frac{E(T)}{E(20)} = 1.0 + \frac{T}{2000 \ln\left[\dfrac{T}{1100}\right]} \qquad \text{for } 0\,^{\circ}\mathrm{C} < T \leq 600\,^{\circ}\mathrm{C}$$

$$= \frac{690\left(1 - \dfrac{T}{1000}\right)}{T - 53.5} \qquad \text{for } 600\,^{\circ}\mathrm{C} < T \leq 1000\,^{\circ}\mathrm{C}$$

where T = temperature of steel in °C,
$E(T)$ = modulus of elasticity of steel at T °C, and
$E(20)$ = modulus of elasticity of steel at 20 °C.

2.2. Yield stress

In general, yield stress varies nonlinearly with temperature (ECCS 1983, CTICM 1982, Chiou 1991). A linear expression, based on regression analyses of data and given in the Australian steel code AS4100 (Standards Australia 1990), is adopted in the present study and is given as

$$\begin{aligned} \frac{f_y(T)}{f_y(20)} &= 1.0 \qquad \text{for } 0\ ^\circ\mathrm{C} < T \leq 215\ ^\circ\mathrm{C} \\ &= \frac{905-T}{690} \qquad \text{for } 215\ ^\circ\mathrm{C} < T \leq 905\ ^\circ\mathrm{C} \end{aligned} \tag{2}$$

where $f_y(T)$ = yield stress of steel at T °C.

2.3. Coefficient of thermal expansion

The coefficient of thermal expansion, α, also varies with temperature. Eq. (3) given below is derived from British test data (BISRA 1952).

$$\alpha(T) = (12.0 + T/100) \times 10^{-6} \quad (^\circ\mathrm{C})^{-1} \tag{3}$$

where $\alpha(T)$ = coefficient of thermal expansion at T °C.

Eqs. (1), (2) and (3) encompass the mechanical properties of steel to be used in the current work for the analysis of structures at elevated temperatures. It should be noted that these properties are assumed to be constant at a certain temperature at which the analysis can be carried out in the usual manner.

3. Behaviour of steel members at elevated temperatures

3.1. Internal end actions

When the temperature of a steel member rigidly restrained at both ends is increased, internal end actions are induced as a result of the expansion of the longitudinal fibres of the member. These internal end actions, sometimes termed self-straining actions, are transmitted through the ends of the member and re-distributed as member forces to other parts of the structure. The thermal stress, σ_T, induced at any point in the cross-section of the member due to a temperature rise ΔT is given by

$$\sigma_T = E(T)\alpha(T)\Delta T \tag{4}$$

Eq. (4) represents the state of the member's stress which is constant across the depth of the cross-section if subject to uniform temperature rise but varies if subject to a temperature gradient from $T1$ to $T2$ as shown in Fig. 2. In the latter case where temperature gradient exists, bowing action is induced due to the differential expansion of the longitudinal fibres in the cross-section, resulting in bending moments in the member.

Hence, the internal end actions, in terms of axial force N_T and bending moment M_T, at the member ends can be given as

$$N_T = \int_A \sigma_T \,\delta A \tag{5a}$$

$$M_T = \int_A \sigma_T y \,\delta A \tag{5b}$$

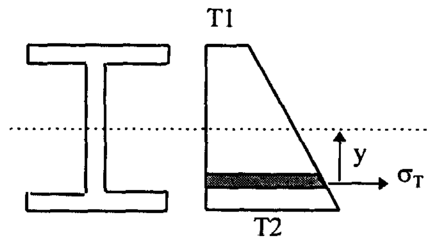

Fig. 2 Temperature gradient across a cross-section.

where A is the cross-sectional area and y is the distance between σ_T and the neutral axis. It should be noted that ΔT given in Eqs. (4), (5a) and (5b) for any point in the cross-section can be expressed as a function of $T1$, $T2$ and y. For $T1 = T2$, $M_T = 0$.

It can be seen that Eqs. (5a) and (5b) are highly nonlinear and numerical techniques, such as the Newton-Cotes closed formula, can be used for the integration.

3.2. Modified end actions

In an elastic matrix analysis for structures, the element constitutive relationship can be written as

$$\boldsymbol{p} = \boldsymbol{kd} + \boldsymbol{p}^o \tag{6}$$

where $\boldsymbol{p}$ is the external load vector, $\boldsymbol{k}$ the stiffness matrix, $\boldsymbol{d}$ the displacement vector, and $\boldsymbol{p}^o$ the vector containing all internal actions, including those given in Eqs. (5a) and (5b).

In an elastoplastic analysis (Tin-Loi and Wong 1989), the values of the elastic internal end actions must be modified according to the state of plasticity of the loaded member. These modified internal end actions used in the elastoplastic analysis are herein termed the Modified End Actions (MEA). Thus, the element constitutive relationship in an elastoplastic analysis can be expressed as

$$\boldsymbol{p} = \boldsymbol{k}^p \boldsymbol{d} + \boldsymbol{v}^o \tag{7}$$

where $\boldsymbol{k}^p$ is the elastoplastic stiffness matrix and $\boldsymbol{v}^o$ is the vector containing the MEA. Explicit expressions for $\boldsymbol{k}^p$ for various states of plasticity for both pure bending and bending-axial interaction yield criteria have been derived by Wong (1991). Treatment of $\boldsymbol{v}^o$ belongs to a class of problem where the same technique for deriving $\boldsymbol{k}^p$ in an elastoplastic analysis can be applied to form $\boldsymbol{v}^o$ from a range of internal end actions induced by, for instance, foundation settlement (Wong 1995) and uniformly distributed load (Crozier and Wong 1995). The technique has also been used by Ueda, *et al.* (1985) for the treatment of internal end actions induced by a temperature rise.

If a displacement vector, $\boldsymbol{q}^o$, is defined as

$$\boldsymbol{q}^o = \boldsymbol{k}^{-1} \boldsymbol{p}^o \tag{8}$$

then the MEA vector, $\boldsymbol{v}^o$, for each element can be derived as

$$\boldsymbol{v}^o = \boldsymbol{k}^p \boldsymbol{q}^o \tag{9}$$

Eq. (9) is derived on the basis of the theory of plasticity. It has four different forms according to the four different states of plasticity (Wong 1991). It should be noted that for the simple case of elastic state where $k^P = k$, Eq. (9) gives $v^o = p^o$.

For a complete structure, the constitutive relationship of the structure can be written as

$$P = K^P D + V^o \tag{10}$$

where P, K^P and V^o are assembled from the corresponding terms in Eq. (7) of all elements, and D is the displacement vector of the structure.

3.3. Member capacity at elevated temperatures

The strength capacity of a member decreases with increasing temperature as a result of the decrease in yield stress, $f_y(T)$, according to Eq. (2). The reduced axial force capacity, N_{TC}, and bending moment capacity, M_{TC}, can be written in a form similar to Eqs. (5a) and (5b) as

$$N_{TC} = \int_A f_y(T) \delta A \tag{11}$$

$$M_{TC} = \int_A f_y(T) y \, \delta A \tag{12}$$

For a section under a specified temperature gradient, Eq. (12) can be integrated numerically after the neutral axis is located.

4. Collapse analysis under elevated temperature

When a structure is on fire, the temperature in some or all of the members rises. As a result, the strength of the structure deteriorates due to the degradation of the mechanical properties of the heated elements. The temperature in those affected members may reach a limiting value at which the structure as a whole collapses under both the imposed design loads and the thermal loads. The collapse temperature for members in a structure may be different from that when the same members are isolated and considered under ideal end conditions.

In performing a traditional plastic analysis, the imposed design loads of the structure are increased proportionally by a common load factor, β. Collapse of the structure is attained corresponding to a collapse load factor β_c. For a safe design, $\beta_c \geq 1.0$ at all time.

The procedure of analysis proposed in the present work follows similar methodology for elastoplastic analysis except that the thermal loading due to temperature rise in each member is applied prior to the application of the static loading. It can be argued that the final state of stresses in the members is independent of the sequence of application of static and thermal loadings so long as each separate loading does not induce collapse of the structure. Therefore, it is assumed here that the sequence of application of static and thermal loadings will not affect the final collapse load of the structure. If a particular temperature distribution for the structure is known at a particular time, the mechanical properties of the elements can be found and an incremental elastoplastic analysis carried out for the structure in order to calculate β_c. To satisfy design requirements for fire endurance analysis, information in two areas needs to be found: (i) the temperature distribution in the structure at a critical time during which the structure is likely to collapse; and (ii) the collapse load factor β_c under such temperature distribution. To obtain information for (i), knowledge of the spatial development of fire is required so that the way the fire is spread can be obtained. Fire spread is a complex process involving thermodynamics and is beyond the scope of the present work. However, the heat transfer process in a fire scenario is important in determining the temperature of steel. A heat transfer model is presented in the following section.

5. Heat transfer model

The calculation of the fire endurance period of a structure requires knowledge of the temperature-time relationship of the fire so that the temperature rise in the element can be traced according to the heat transfer mechanism. A proposed heat transfer model for calculating the temperature rise in steel under real fire conditions is presented below. The model is based on the assumption where no heat loss occurs through openings and through conduction between members.

The energy balance for a bare steel element of uniform temperature (steel is assumed to be a perfect conductor) in a fire without heat transfer to the surroundings is normally written as

$$\dot{q}_r + \dot{q}_c = \frac{c_p V \rho}{A_r} \frac{dT}{dt} = \frac{c_p \rho}{P/A_s} \frac{dT}{dt} \tag{13}$$

where $\dot{q}_r$ – radiative heat transfer per unit area from the fire in W/m^2,
$\dot{q}_c$ – convective heat transfer per unit area from the fire in W/m^2,
c_p, ρ – specific heat (J/kg K) and density (kg/m^3) of the steel respectively,
V, A_r – volume (m^3) and exposed surface area (m^2) of the steel element respectively,
P, A_s – perimeter (m) and area of cross-section (m^2) of the steel element respectively,
dT/dt – rate of change of temperature of the steel element.

The specific heat of steel is temperature dependent and is used in the form given by Barthelemy (1976):

$$c_p = 3.8 \times 10^{-7}\,\theta_s^2 + 2 \times 10^{-4}\,\theta_s + 0.472 \tag{14}$$

where θ_s – temperature of the steel element in °C.

In the widely used simple heat transfer model presented by Barthelemy (1976) and Smith and Stirland (1983) for steel element temperature calculation, the convective heat transfer component is given by

$$\dot{q}_c = h\,(T_g - T_s) \tag{15}$$

where h – coefficient of convective heat transfer, T_g – gas temperature in K, and T_s – steel temperature in K.

The radiative heat transfer component is given by

$$\dot{q}_r = \varepsilon\sigma(T_g^4 - T_s^4) \tag{16}$$

where ε – emissivity of the fire and σ=5.67$\times 10^{-8}$ W/m^2K^4 – Stefan-Boltzmann constant.

Eq. (16) implies that the adopted radiation model assumes the fire to be a hot grey body in a large black enclosure separated by perfectly transparent media (vacuum, air at normal temperatures and pressures). In Smith and Stirland (1983)'s work, the value of emissivity of the fire is artificially varied within a wide range to give acceptable prediction of the steel temperature.

A more realistic heat transfer model taking into account the effects of the presence of combustion products has been constructed in the current work to describe the heat exchange between real fires and steel elements in an enclosure. The proposed model retains the convective component as described above and assumes the coefficient of convective heat transfer h to be constant. To assess the effect of h on the predicted steel temperature, two values of h have been tried in the current work: 15 and 23 W/m^2K.

The radiative component is radically different from the one in the simple model proposed by Barthelemy (1976) and Smith and Stirland (1983). The assumptions underlying the proposed radiative heat transfer model are:

(1) Radiation from the fire is assumed to be emanating from a gas comprising combustion products at a uniform temperature T_g filling a black enclosure at temperature T_s. The assumption of a black enclosure can be justified on the grounds that in case of fire, the enclosure walls are likely to be covered by dirt and soot making it almost a black body. Since the enclosure completely surrounds the gas, the view factor is equal to unity.

(2) The gas in the enclosure is not a black body, and is a mixture of 10% water vapour (H_2O), 10% carbon dioxide (CO_2) and 80% nitrogen (N_2) at a pressure of 1 atm. H_2O and CO_2 emit and absorb radiation at any temperature. N_2 is essentially transparent to radiation at the temperatures encountered in fires.

(3) The enclosure is assumed to be cubical in shape with dimensions $6 \times 6 \times 6$ m. This results in a mean beam length L_e of 3.6 m. The mean beam length is a characteristic dimension of the heat transfer system and represents the thickness of the gas layer which transmits radiation. It is calculated using the equation $L_e = 3.6 V_g / A_e$ where V_g is the total volume of the gas in the enclosure and A_e is the total surface area of the enclosure (Holman 1986).

The radiative heat transfer equation is now written as

$$\dot{q}_r = \sigma(\varepsilon_g T_g^4 - \alpha_g T_s^4) \tag{17}$$

where ε_g – total gas emissivity (emittance) at temperature T_g K and α_g – total gas absorptivity (absorptance) for radiation from the black enclosure at temperature T_s K.

Total emissivity of a gas ε_g is a function of its temperature, partial pressures of the emitting gaseous species multiplied by gas layer thickness (a function of mean beam length L_e) and total pressure (Mills 1992). Total absorptivity of a gas α_g for radiation from a black enclosure is a function of both gas and enclosure temperatures. Figs. 3 and 4 show some of the emissivity and absorptivity data used in the proposed model. These data were reproduced from a computer program for total gas properties by Mills (1992) based on the work by Edwards and Matavosian (1984, 1990).

A computer program was written to test the capability of the proposed model for predicting the temperature of bare steel surfaces exposed to real fire conditions. The results of three test runs based on data provided by Smith and Stirland (1983) are presented below. Polynomials were fitted to the gas temperature data to facilitate calculations.

Run 1	Fire load	15 kg wood/m^2
	Ventilation	1/2
	Steel member	203×203 mm $\times 52$ kg/m column
	P/A_s=180 m^{-1}	

The steel member was exposed to real fire from four sides. Fig. 5 shows the results of temperature predictions by the simple model and the proposed model. The correlation between the observed and predicted steel temperatures is good with regards to the shape of the curves and the onset of maximum temperature. The coefficient of convective heat transfer h for best corre-

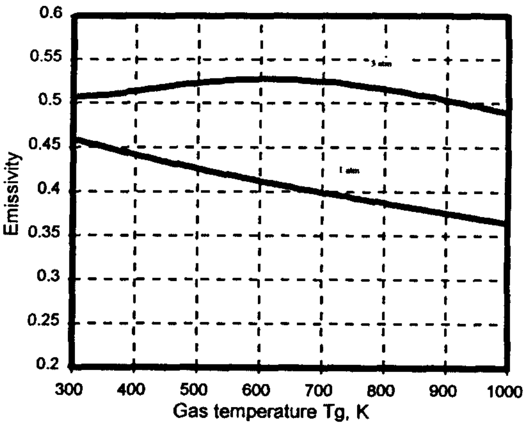

Fig. 3 Emissivity data for a gas mixture of 10% CO_2, 10% H_2O and 80% N_2 at two pressures.

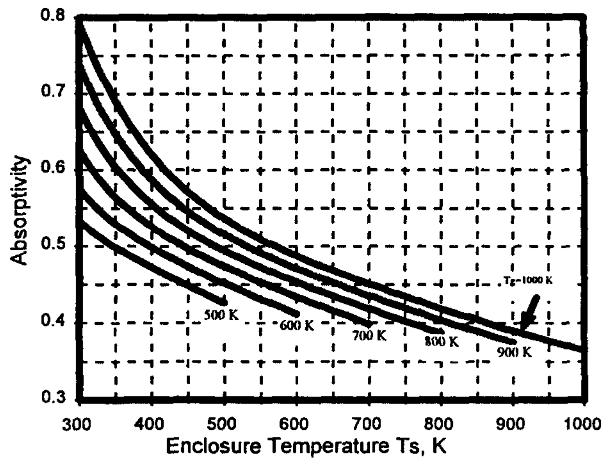

Fig. 4 Absorptivity data for a gas mixture of 10% CO_2, 10% H_2O and 80% N_2 at a pressure of 1 atm.

lation seems to be about 20 W/m^2 K.

Run 2	Fire load	15 kg wood/m^2
	Ventilation	1/4
	Steel member	203×203 mm×52 kg/m column
	$P/A_s = 180$ m^{-1}	

The steel member was also exposed to real fire from four sides in this run. Fig. 6 shows the results of temperature predictions by both the simple model and the proposed model. The proposed model over predicts steel temperatures from $t = 10$ min to $t = 25$ min. The shapes of the curves are different with the onset of maximum temperature being 5 minutes earlier for the proposed model. The deviations from the observed temperatures in this run are difficult to explain at this stage and more runs are required to verify the validity of the model in its present form.

Run 3	Fire load	7.5 kg wood/m^2
	Ventilation	1/4
	Steel member	203×203 mm×52 kg/m column

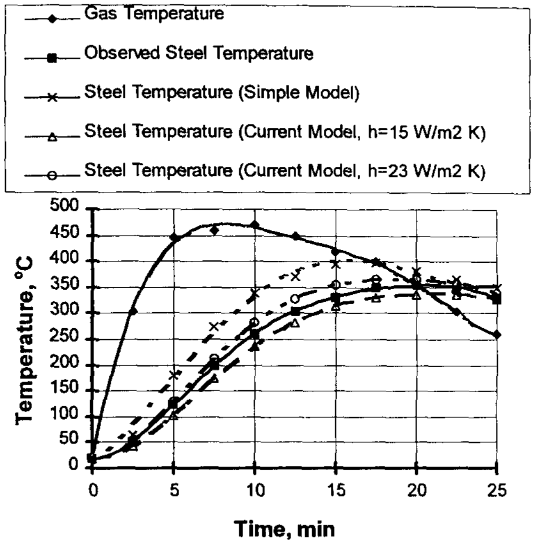

Fig. 5 Observed and predicted steel temperatures for Run 1.

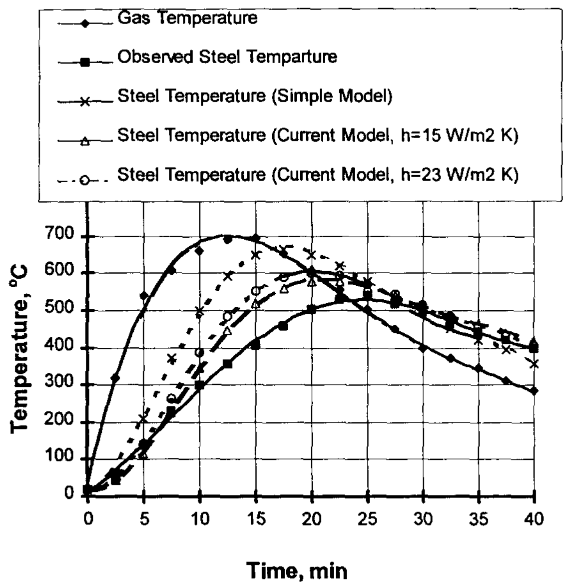

Fig. 6 Observed and predicted steel temperatures for Run 2.

$$P/A_s\text{=}180\ \mathrm{m}^{-1}$$

The steel member was also exposed to real fire from four sides in this run. Fig. 7 shows the results of temperature predictions by the simple model and the proposed model. The coefficient of convective heat transfer h was set at 15 W/m^2 K for this run. The outcome is very similar to Run 1 with the proposed model slightly under predicting the steel temperature.

On the basis of the results obtained in the computer runs, it can be concluded that the proposed model provides much better temperature prediction for bare steel surfaces exposed to simulated real fires than the conventional simple model. Further work is being carried out to re-

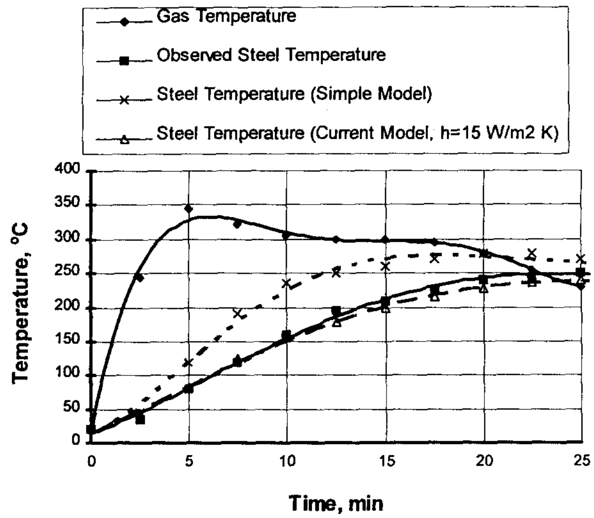

Fig. 7 Observed and predicted steel temperatures for Run 3.

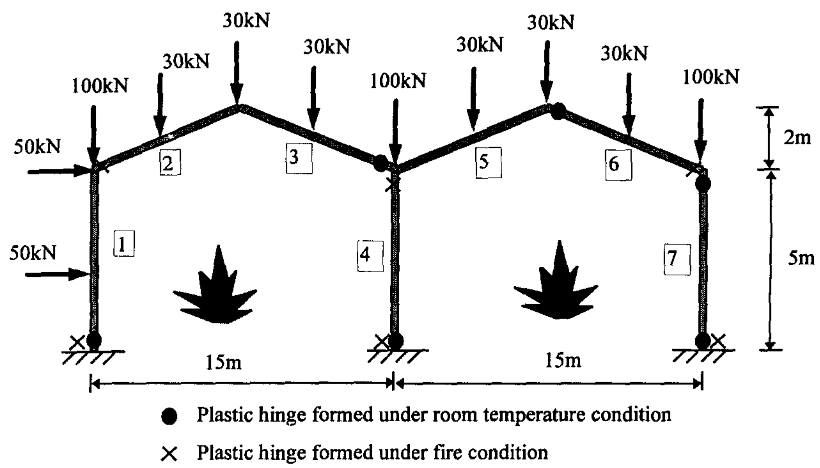

Fig. 8 Pitched-roof portal frame example.

fine the convective heat transfer model. Also more computer runs involving a wide range of simulated fires will be conducted to verify the model.

6. Examples

The numerical examples presented below demonstrate the use of the proposed method for calculating the collapse-temperature-time relationship for bare steel frames under different fire scenarios. Although it is rare that a horizontal load, such as wind load, will co-exist with the maximum vertical design load, this case is also included in the examples for illustration purposes.

6.1. Pitched-roof portal frame

A two-bay pitched-roof portal frame under a set of static loads as shown in Fig. 8 was analysed using the proposed method. The columns are 250UC89 and the rafters 410UB60 I-sections. All members have a yield stress of 260 N/mm^2 at room temperature. Both bays were assumed to be under the same real fire conditions as given in Fig. 6. It was found that at one stage during the fire, members 1 to 4 in one bay had a maximum uniform temperature of 600°C and members 5 to 7 in the other bay of 300°C.

Two analyses were performed on the structure, one under room temperature conditions and the other under the fire conditions described above. The coefficient of convective heat transfer h was set at 23 W/m^2 K. The following observations were drawn from the analyses.

1. The calculated collapse load factors were 3.09 for the structure at room temperature and 1.22 under the prescribed fire conditions. That is, the fire had reduced the strength of the structure by about 60%. A total of 6 plastic hinges occurred before collapse in both cases. Since the collapse load factor of the structure under the prescribed fire conditions was greater than 1.0, the structure was likely to survive total collapse during the fire.
2. If the fire-temperature characteristics followed exactly that of Fig. 6, then the maximum steel temperature of 600°C occurred 20 minutes after the fire started and that of 300°C occurred 8 minutes after the fire started. It can be concluded that the fire on the right started 12 minutes after the fire on the left had started.
3. In carrying out the analysis, the thermal loading was applied prior to the static loading. It was found that one plastic hinge occurred when the thermal loading alone was applied. In this case, the vector containing the modified end actions given in Eq. (10) must be used for the member with the plastic hinge in order to cater for the thermal loading due to the increase in temperature.

6.2. Multi-storey steel frame

The steel frame shown in Fig. 9 was analysed to demonstrate the effects of fire on multi-storey buildings. All members are I-sections and have a yield stress of 350 MPa at room temperature. The yield criterion is based on a linear interaction relationship between axial force and bending moment. Two fire scenarios shown in Fig. 10 were assumed: Case (a) – fire on the middle floor and Case (b) – fire on both the middle and lower floors. In both cases, the members were assumed to be thermally insulated in such a way that the temperature distribution was as shown in Fig. 10. Both cases followed the observed steel temperature curve given in Fig. 6, giving a maximum temperature of approximately 500°C at 25 minutes. Three analyses were carried out: Room temperature, Case (a) and Case (b).

For room temperature analysis, twelve plastic hinges occurred before collapse, resulting in a collapse load factor of 3.09.

For Case (a), a temperature gradient was assumed to exist for the beams below and above the middle floors and the columns were well insulated so that the increase in temperature in the columns was negligible. The collapse load factor for this case was found to be 2.35, a 24% reduction in strength due to the fire.

For Case (b), the beams were assumed to be well insulated from the fire without any significant temperature increase whereas a temperature rise occurred in the columns. The collapse load factor for this case was found to be 1.72, a significant 44% reduction in strength due to the fire.

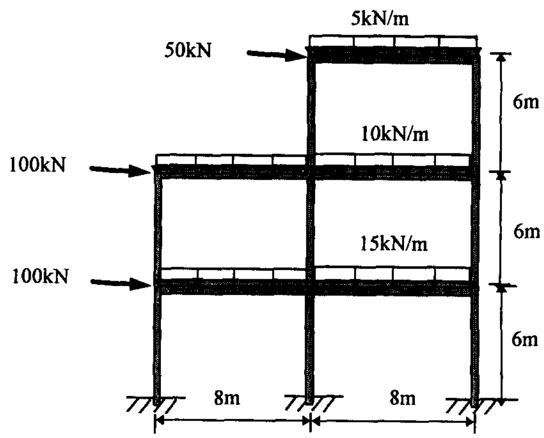

Fig. 9 Multi-storey steel frame example.

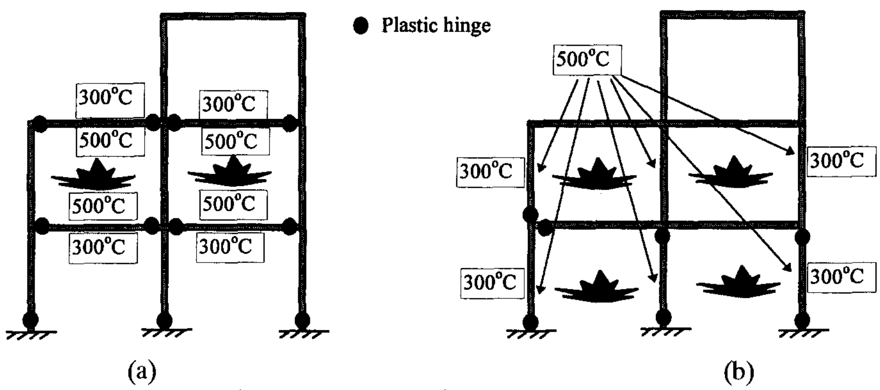

Fig. 10 Fire scenarios for steel frame.

Table 1 Results of analyses for multi-storey steel frame

Fire conditions	Time (min.)	Collapse load factor
Room temperature	0	3.09
Case (a)	25	2.35
Case (b)	25	1.72

It can be said that the effect of fire in Case (b) is more severe than that in Case (a). This is reasonable as the columns provide a major resistance to the pushover action of the horizontal loads and, when weakened by the fire, will reduce the collapse strength of the structure significantly. A summary of the results of the three analyses is given in Table 1.

7. Conclusions

Performance based design codes have become increasingly popular in structural design in which the structure should satisfy the design requirements on the basis of realistic situations. In designing structures under fire conditions, it is therefore more realistic to simulate the real fire conditions when carrying out structural analysis. In this paper, a method to calculate the collapse load factor under prescribed real fire conditions at a specified time is presented. It must be emphasised that in the analysis at a fixed temperature for a specified fire scenario, a linear structural constitutive relationship is always maintained, so that iteration procedures, as used in a number of other methods in high temperature structural analysis, can be avoided while equilibrium can still be maintained.

A new heat transfer model is proposed for the calculation of steel temperature under real fire conditions. The role of gases emanated from fire burning in transmitting heat energy through radiation has been examined in detail, resulting in an improved numerical model for simulating the heat transfer process during fire. When compared with experimental data, the new model shows good results in predicting steel temperature.

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