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Strength and Deformation Capacities of Simple Floor Steel Beams under various Protection Schemes in Fire Conditions

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Abstract— **Beam-column members are vital elements of construction found in most buildings that incorporate simple and complex structural designs. These elements are primarily designed to support both flexural and axial loadings, with the tendency to demonstrate a different behaviour under fire conditions. However, large displacements are often experienced by these elements when subjected to elevated temperatures thus making the analysis to become more complex. Fewer numerical studies regarding fire analysis of partly protected beam-column members are available before now. Therefore, it was considered paramount to conduct a sensitivity study to quantify the means by which simple beam-column members can be economically protected so that structural integrity could be achieved within the fire resistance period. To accomplish this, a performance based methodology was developed and the fire behaviour was defined by implementing the standard fire model. Then, the thermal and structural computations were characterised as sequentially coupled. The numerical models were validated against test data and good agreement was reached. Consequently, a parametric investigation was carried out to establish the optimum passive fire protection strategy to adopt for the structural element. Findings revealed that provided there is sufficient lateral restraint to the beam web, it is safe to protect the bottom flange of simple beams while leaving the upper flange and web unprotected. This study has therefore provided a guide to fire analysis of elements of construction that may incorporate passive fire protection systems.**

Index Terms— **Elevated temperature, floor steel beams, lateral restraint, passive fire protection**

I. Introduction

THE performance of steel beam-column members at elevated temperature has been the subject of topical research projects, which have been investigated both numerically and experimentally [1]-[4]. However, a few that have studied the elevated temperature behaviour of steel structural elements using analytical procedure have actually considered simple structures as it is often difficult to carry out analytical calculations in fire conditions. Fire design of structures has been traditionally evaluated using deterministic techniques where the fire loadings are computed based on the simple models described in Eurocode 3 [5]. Currently, a new technique that can fully incorporate the likely uncertainties associated with the loadings and responses of the 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 [6], [7]. These two approaches have been found to be useful in evaluating critical response parameters for fire design of structures. Moreover, the new development in research seems to rely more on valid numerical predictions to circumvent the enormous costs that are necessarily associated with experimental works.

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. Moreover, the traditional prescriptive approach to design is based on individual member analysis to determine the strength and behaviour under various loading conditions. This approach has been effectively applied in the design of most of the existing buildings. In the case of universal beam steel sections, the critical element is recognised to be the bottom flange. Nevertheless, the web can be assumed critical for laterally unrestrained sections, which can often result in lateral-torsional buckling under compressive loadings [1].

Contemporary experimental and numerical studies have also shown that it is possible to protect only the critical structural members of a structure subjected to fire, such as columns while leaving most of the beams unprotected [8]-[10]. This is based on the assumption that under large displacement the reinforced concrete slab will utilise the inherent tensile membrane action as a carrying-capacity to support the load even when the internal beams have yielded completely [8], [9]. Nevertheless, in circumstances where tensile membrane actions do not exist the behaviour will be such that the slabs would yield completely without any significant reserved strength. Hence, it becomes very paramount to evaluate the behaviour of steel beams under fire conditions. In this paper, a performance based methodology is developed to determine the most economical way by which the element of a structure, namely the beam-column member, can be protected in the event of an accidental loading such as a fire.

II. Elevated Temperature Material Properties

An easy way to comply with the journal paper formatting requirements is to use this document as a template and simply type your text into it. The material properties of steel at elevated temperatures are required to model the behaviour of structures in fire conditions. When a steel structure is exposed to fire, the temperature of the element will increase from room temperature to a high level, thereby leaving the material to degrade in strength and stiffness. The loss of strength and stiffness at elevated

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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 needed to accurately predict their behaviour.

The thermal properties that are of interest to the fire engineer include thermal conductivity, specific heat, thermal strain and thermal expansion. Conversely, the mechanical properties that are of concern to the design engineer include the potential yield strength, the ultimate tensile strength, the strain hardening and the stress-strain curve. The empirical models that are implemented to determine these elevated temperature material properties illustrated in Figs. 1 and 2 are given in Eurocode 3, Part 1-2 [5].The stress-strain model for carbon steel at elevated temperatures is illustrated in Fig. 3 whereas the stress-strain-temperature characterisation of carbon steel based on the stress-strain model of Fig. 3 is presented in Fig. 4. The matrix of test specimens used in the study is presented in Table 1.

TABLE 1 Test specimens geometric and material properties

Test No.	Effective span (m)	Yield stress (MPa)	Point variable load (N)
1	4.5	Flange =284	44150
		Web =306	
2	4.585	Flange =411	46730
		Web =399	
3	4.585	Flange =250	32540
		Web =277	
4	4.465	Flange =408	33920

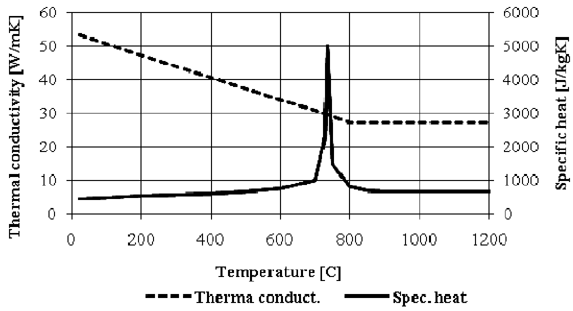

Fig. 1 High temperature thermal properties of carbon steel

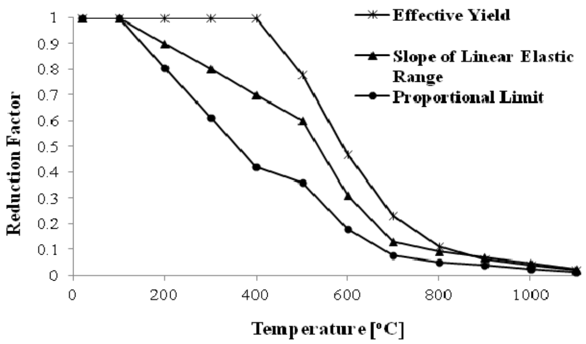

Fig. 2 Mechanical properties reduction factors for carbon steel at elevated temperatures

Each test beam has an exposed length of 4.0 m with simply supported ends. It should be noted that test beam 1 has two loading points whereas test beams 2, 3 and 4 each has four loading points. The end conditions for each of the test beams could permit translation once the beams were heated thereby limiting the magnitude of deflection and increasing the failure time. It is notable to recognise that the mechanical performance of representative steel can be greatly affected by the chemical composition and physical characteristics surrounding its formation. In fact, carbon content has been known to influence the structural behaviour of most steels [11]. At temperatures above 400°C mild steel begins to lose its strength and continues to reduce in strength at an approximately steady rate up to around 800°C when the rate of strength loss flattens out as illustrated in Fig. 2. However, the reduction in the stiffness and nonlinearity in behaviour of mild steel may start at temperatures below 400°C as recommended in Eurocode 3 [5].

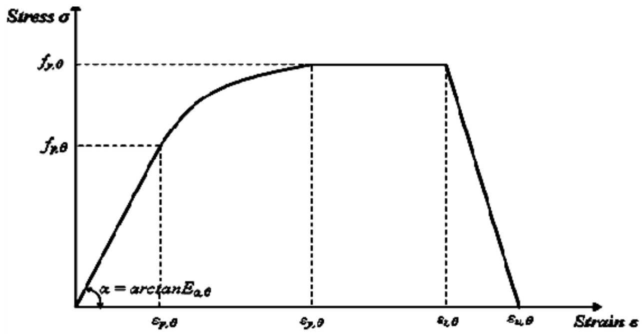

Fig. 3 Stress-strain model for carbon steel

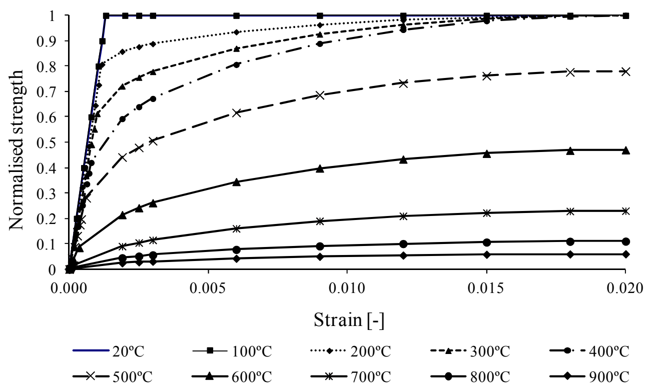

Fig. 4 Stress-strain-temperature curves for carbon steel

III. FIRE LIMIT STATE DESIGN

Generally, in the limit state definition safety of structures is guaranteed when [12]:

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

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

However, at the fire limit state the partial safety factors for permanent and variable actions are put to unity to account for the likelihood of reduced loading in the event of a fire thereby leading to the design value of the actions adjusted as follows [5]:

$$E_{f_i,d,t} = G_d + \psi_1 Q_{k,1} + \psi_2 Q_{k,2} \tag{2}$$

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where ψ_1 and ψ_2 are load combination factors depending on the limit state under consideration, and G and Q are the permanent (dead) and variable (live) loads respectively.

Hence, load level at exposure time in fire condition can be expressed in Equation (3) as given in Eurocode 3 [5] and by Lawson and Newman [12]:

$$\eta_{f_i,t} = \frac{E_{f,d,t}}{R_d} = k_{y,\theta}\left(\frac{\gamma_M}{\gamma_{M,f_i}}\right) \leq \frac{R_{f,d,t}}{R_d} \qquad (3)$$

where $\eta_{f,t}$ is load level at exposure time in fire conditions, $E_{f,d,t}$ is the actions effect at the fire limit state, R_d is the design value of resistance, γ_M is the partial safety factor on material, $\gamma_{M,fi}$ is the partial safety factor on material in fire conditions, $k_{y,\theta}$ is the reduction factor for effective yield.

IV. MODELLING PROCEDURE

The analysis was based on finite element method with a general-purpose commercial code-ABAQUS [13], which has the capability to calculate the thermal and mechanical responses of structures. The large displacement effects of the non-linear finite elements are considered using an updated Lagrangian formulation [13], [14]. To model the thermal field, the standard ISO 834 [15] temperature curve was used to define the fire load applied to the requisite surfaces of the beam. Once the temperature profile in the heat transfer analysis was determined, it was then sequentially coupled with the structural analysis by setting the temperature values at the nodes in the structural model, thus accounting for the effects of thermal strains in the model. The multi-linear stress-strain-temperature curves for the structural model shown in Fig. 4 describes the non-linear behaviour of the material in fire conditions. The plasticity is calculated by the von Mises yield criteria and the associated flow rule [11], [13].

All the beams are 254x146x43UB and subjected to various point loads with a load level of 0.6 [16]. The loads were computed using elastic design in accordance with BS5950, Part 8 [17]. The segmented concrete toppings, nominally 650mm wide and 130mm thick, were characterised as non-structural and their influence on the resultant deformation of the beams was insignificant in all the analyses. The test beam is shown in Fig. 5 while the representative test furnace with a typical test specimen is presented in Fig. 6. However, to avoid stress concentration the total variable load at each loading point was divided equally amongst nine nodes. The passive fire protection is implicitly modelled such that the member being protected will remain stable within the fire resistance period. Based on this assumption, a parametric study was conducted to evaluate the most practical means by which simple beam-column elements in buildings can be economically protected.

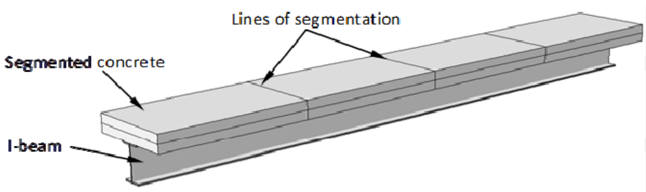

Fig. 5 The beam model carrying segmented floor slabs

The modelling of heat transfer involves the application of heat energy to a surface. Obviously, the transient temperature field shows both temporal and spatial variations in which the three mechanisms of heat transfer are involved. However, the thermal conductivity value of steel does not show significant spatial variation so that a homogeneous thermal conductivity k can be adopted as follows:

$$k\left(\frac{\partial^2 T}{\partial x^2}+\frac{\partial^2 T}{\partial y^2}+\frac{\partial^2 T}{\partial z^2}\right)+Q(x,y,z,t)-\rho c\frac{\partial T}{\partial t}(x,y,z,t)=0 \qquad (4)$$

The correct boundary conditions necessary to solve (4) relate to initial temperature T, internal heat generation $Q = f(q)$ and heat fluxes q_r as given in (5) and (6).

$$T(x,y,z,t=0) = T_0(x,y,z) \qquad (5)$$

$$\begin{aligned} &k\left(\frac{\partial T}{\partial x}N_x + \frac{\partial T}{\partial y}N_y + \frac{\partial T}{\partial z}N_z\right) + q(x,y,z,t) \\ &+ h_c(T_s - T_\infty) + \sigma\varepsilon VF(T_s^4 - T_f^4) \end{aligned} \qquad (6)$$

where:
T is the current temperature, $Q(x,y,z,t)$ is the rate of internal heat generation, t is the time, ρ is the density and c is the specific heat; N_x, N_y and N_z are the direction cosines orthogonal to the boundary surface, h_c is the film coefficient, T_s is the surface temperature assumed uniform through the thickness, T_f is the fire temperature, T_∞ is the temperature of the surrounding medium, ε is the surface emissivity of the material, σ is the Stefan Boltzmann constant (= 5.667×10^{-8} $Wm^{-2}K^{-4}$) and VF is the configuration factor.

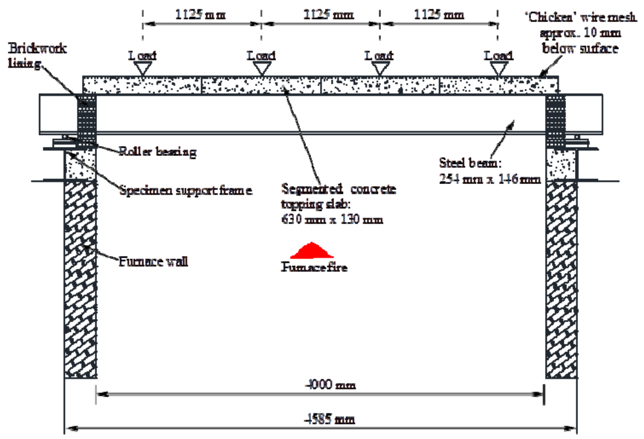

Fig. 6 Longitudinal section for the simply supported floor beam test 2 assembly[16]

The transport properties, which comprise convective heat transfer coefficient and radiative emissivity utilised in the numerical calculations for the thermal field are 25 W/m^2K and 0.7 respectively. The configuration factor was assumed unity by disregarding the position and shadow effects. This

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temperature field was then sequentially applied to the mechanical model as a forcing function to compute the stresses and strains using incremental plasticity theory with temperature-dependent mechanical material properties. The material model assumed for the elastic analysis of steel structures typically follows the von Mises yield criterion. Different analysis methods can be implemented to trace the equilibrium path in an incremental procedure. ABAQUS [13] uses Newton-Raphson method to approximate the incremental stress in nonlinear elasto-plastic problems. The equilibrium equation and the equivalent thermo-mechanical constitutive model used to determine the strength and behaviour of the beam-column model are given as follows:

$$\sigma_i = \mathrm{E}\varepsilon_i \tag{7}$$

$$\{\Delta\sigma\} = \{D^{ep}\}[B]\{\Delta U_e\} - \{C^{th}\}[M]\{\Delta T_e\} \tag{8}$$

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.

V. Parametric Investigation

The critical member of a rolled steel joist in fire conditions is the bottom flange as specified in Eurocode 3, Part 1.2 [5]. However, little information is available to characterise the increase in the carrying capacity of the beam when this element is fully or partially protected. Providing that the beam is fully restrained against lateral-torsional buckling, the strength and deformation capacities of the beam can be greatly enhanced by protecting the bottom flange of the floor beam. A series of analyses was conducted based on previously validated model [14] to quantify the strength and deformation characteristics of the beam under various protection schemes. Test beam TB1 was selected and a comprehensive series of sensitivity study was conducted. The strategy involves lone protection of the beam bottom flange, web, top flange or a combination of any two of these parts in a single analysis run.

VI. Results and Discussion

The results of the numerical computations are presented in the following section. Firstly, the tested specimen and results of the validated models are presented Figs. 7 to 12. Then, the results of the parametric study are shown in Figs. 13 and 14. The experimental and the numerical results compared reasonably well. The difference in the results only began to appear as the material became more nonlinear at a much high temperature. Based on this verification, a sensitivity study was conducted to examine the most probable and economic way by which the beam could be protected. The selection included "no protection", only web protected" and "only bottom flange protected". This strategy was adopted as a preliminary study to ascertain the behaviour of the element under partial protection scheme.

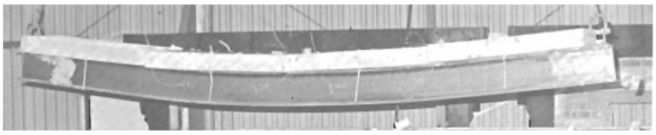
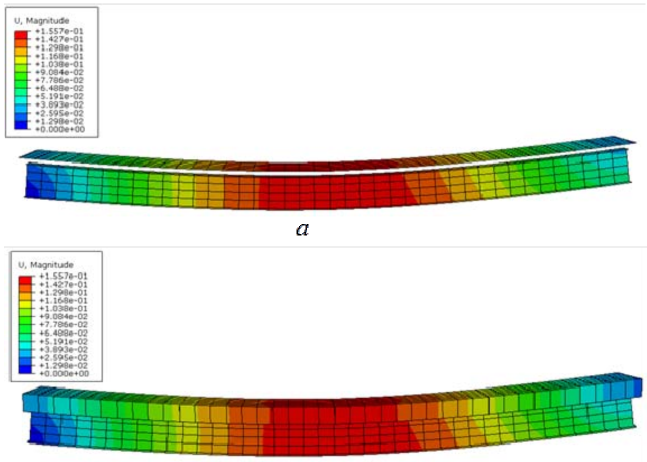


Fig. 7 A 4.5 m long beam tested at the Warrington Research Laboratory [16]

The results of the mid-span displacement and the axial forces for the beam demonstrate that the beam bottom flange is most critical and thus should be fire-protected. Upon protection of this critical section, the midspan deflection reduces considerably and the axial capacity is also enhanced. By protecting the beam web only, the bottom flange is placed in compression, though the mid-span displacement can also be improved only after attaining a higher temperature. It can be surmised that provided there is lateral restraint to the beam web such that torsional buckling can be prevented, it is safe and most economical to protect only the beam bottom flange in the event of a fire.



a

b

Fig. 8 Deflection profile for the numerical calculation of test beam 1 (*a*) Shell element (b) Rendered shell element thickness

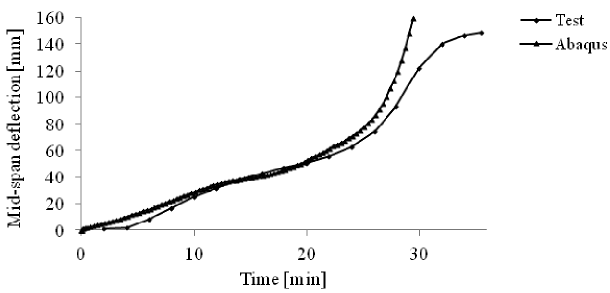


Fig. 9 Mid-span deflection for test beam 1

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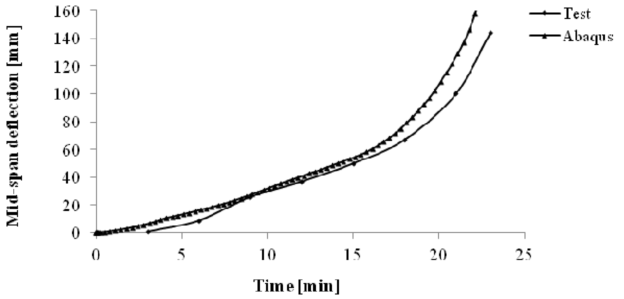

Fig. 10 Mid-span deflection for test beam 2

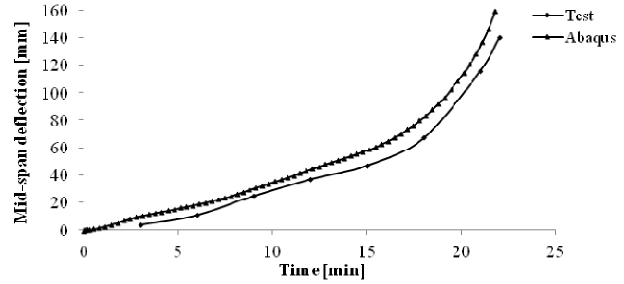

Fig. 11 Mid-span deflection for test beam 3

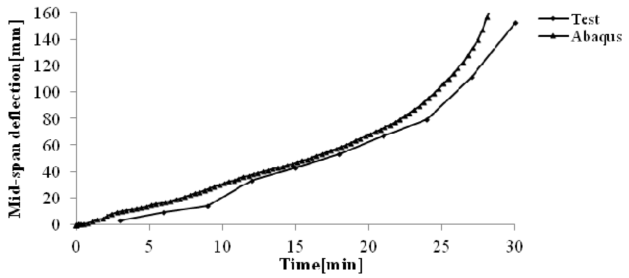

Fig. 12 Mid-span deflection for test beam 4

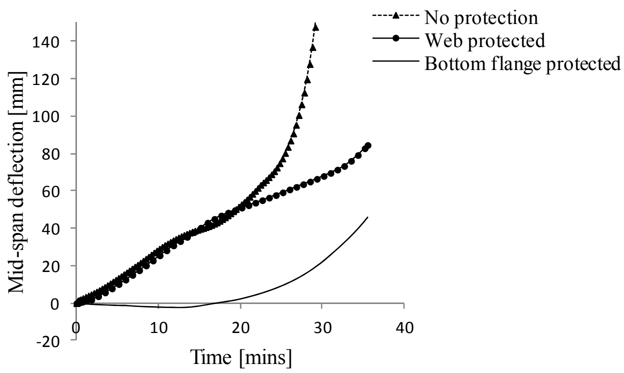

Fig. 13 Mid-span deflection for test beam 1 for different protection schemes

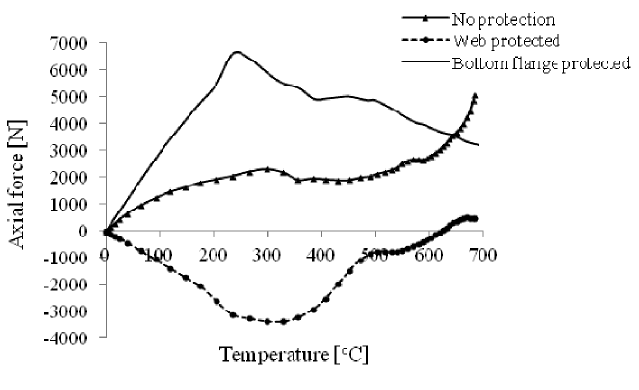

Fig. 14 Axial force in test beam 1 for different protection schemes

VII. CONCLUSION

The strength and deformation characteristics of steel beam-columns subjected to lateral pressure under heat loads were investigated by a series of numerical computations. The results demonstrate that the flexural strength of the beams depends very much on the particular scheme adopted for fire protection as described in this paper. Findings reveal that the application of fire protection material to the web alone would limit the vertical displacement of the beam and put the bottom flange in compression. However, the protection of the beam bottom flange alone will considerably limit the vertical displacement and thus enhance the carrying capacity. It is proposed that only the bottom flange of a beam can be protected to enable the utilisation of the post-yield strength in fire conditions.

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