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Numerical modelling of high strength steel beams at elevated temperature

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Abstract

High strength steels are increasingly common in structural engineering applications owing to their favourable strength to weight ratio, excellent sustainability credentials and attractive physical and mechanical properties. However, these grades are under-used in structures owing to a lack of reliable information relating to their structural performance, particularly at elevated temperature. This paper presents a review of high strength steels in structural applications including the key design considerations. Particular focus is given to the lateral torsional buckling response of laterally unrestrained beams. A finite element model is developed to investigate this behaviour at ambient and elevated temperature. A series of beams between 500 and 4500 mm in length are studied in order to develop buckling curves which are comparable with current design provisions. At ambient temperature, it is shown that all of the buckling curves currently included in Eurocode 3 Part 1-1 give unsatisfactory and potentially unsafe predictions. In elevated temperature conditions, the buckling curves presented in Eurocode 3 Part 1–2 depict the behaviour reasonably well but, at relatively high slenderness values, the standard does not always provide a safe prediction. Revised bucking curves are proposed for high strength steel beams for laterally unrestrained beams made from high strength steel.

Keywords

High strength steel; Numerical modelling; Lateral torsional buckling; Unrestrained beam; Fire; Elevated temperature; ABAQUS.

1. Introduction

High strength steels (HSS) are defined as materials with a nominal yield strength of 460 N/mm^2 or above. They have been increasingly used in recent years due to their high strength-to-weight ratio and efficiency. When employed in appropriate applications, HSS can offer environmental benefits owing to the reduced material usage which also leads to aesthetic advantages as architects and designers tend to favour lean and unobtrusive structural elements. Besides these advantages when compared with ordinary steel, the main disadvantages of HSS include the greater initial cost and the high yield strength to Young's modulus ratio which can lead to stability issues, as well as more limited deformation capacity.

The most important feature of HSS for structural design is the high strength-to-weight ratio, which leads to smaller section sizes for similar load capacities compared with using ordinary mild steel. This helps to reduce the dead load acting on the structure and allows long spans to be achieved. A lighter structure requires smaller foundations as well as shorter transportation and construction times. Hence, HSS can offer a very economically efficient solution when used in appropriate applications. In addition to cost advantages, the reduction in material weight requirements also lead to reduced CO_2 emissions and energy use both in terms of steel production and transportation.

An excellent example of an appropriate application for HSS is the Friends Arena Stadium in Solna, Sweden, which was built using grade S690 HSS. This led to savings of 15 % in weight and €2.2 million in cost compared with using S355 steel [1, 2]. Another example is the Oresund Bridge between Sweden and Denmark which saved €22 million by using HSS rather than mild steel for some components [1, 3]. As mentioned before, aesthetic considerations and environmental efficiency are further reasons for using HSS since the thin and lightweight elements are architecturally desirable.

The main concern which engineers must consider when specifying HSS in structural components is the increased likelihood of stability problems when compressive stresses are present. HSS has a higher yield strength compared with mild steel but a similar Young's modulus, which can lead to serviceability and stability issues owing to the limited deformation capacity of HSS. Even if the overall buckling strengths of high strength steel elements are improved when compared with the ordinary steel elements on a non-dimensional basis, buckling is still the major problem for HSS elements because of the higher yield ratios. For loading conditions causing inelastic deformations, such as fire and earthquake, this high yield ratio and limited deformation capacity can be particularly important. Moreover, as stated before, HSS sections tend to be thinner than ordinary steel sections, for the same load-carrying capacity, and hence are more likely to experience buckling.

In fire conditions, unprotected steel members experience a loss of strength and stiffness with increasing temperature owing to the rapid temperature development in the structural members which is exacerbated by the high thermal conductivity of steel and high surface area to volume ratio of the section. Thus, recent years have seen an increase in research activity in the area of fire engineering for steel structures, especially after high profile events such as the collapse of the World Trade Centre towers in 2001. Although the design of normal steel structural members at room and elevated temperatures is well known, there is lack of information about the design of high strength steel structures at elevated temperatures.

Therefore, it is important and necessary to investigate the behaviour of high strength steel structural elements under elevated temperature conditions. Whilst some researchers have investigated the response of HSS structural members, the focus has mainly been given to the ambient temperature behaviour, particularly on subjects such as material characteristics (e.g. [4-7]), residual stresses (e.g. [4-7]), flexural buckling (e.g. [4, 8, 9]), local buckling (e.g. [5, 6]) and beam behaviour (e.g. [6, 7, 10]). The behaviour of HSS members under fire conditions has received considerably less attention until recently. Previous studies have mainly focussed on the material properties at elevated temperature (e.g. [11-15]), column behaviour [16] and connections [15] with little attention given to beams.

In this context, the current paper is concerned with the behaviour of HSS beams during fire conditions. These elements can be susceptible to lateral torsional buckling (LTB) if sufficient lateral restraint is not provided. The LTB behaviour of normal strength steel beams both at ambient and elevated temperature is well understood and incorporated into design codes [e.g. 17, 18]. However, HSS beams have not received the same amount of attention in terms of their lateral-torsional buckling resistance under fire conditions, mainly as they are still quite a novel construction form. It is only in recent years, when the demand for buildings to be environmentally and economically sustainable has continued to grow, that HSS has really become an attractive solution.

Despite the different mechanical properties of high strength and normal strength steels, including their response to elevated temperature, the Eurocode design approach is identical, including in fire conditions. In this context, the aim of the current study is to investigate the lateral-torsional buckling behaviour of HSS beams with a yield strength of 690 N/mm^2 at elevated temperatures. Numerical simulations are performed on high strength steel beams with different buckling lengths under fire conditions using the materially and geometric non-linear finite element analysis software ABAQUS [19]. The results obtained from the numerical analyses are validated against available test data and then compared with the Eurocode predictions [18] to assess the applicability of current design specification for laterally unrestrained HSS beams in fire conditions.

2. High strength steel in structures

2.1 Strengthening mechanisms

The key to strengthening steel is to restrict or reduce the movement of metallographic imperfections known as dislocations through the material [1]. Inelastic deformations are due to the movement of these dislocations. Therefore, by restricting their mobility, the dislocations require more stress to move through the iron crystal lattice resulting in an increase in yield strength [20]. The movement of dislocations can be reduced by the presence of alloying elements in the form of solute atoms (e.g. molybdenum) or precipitates (e.g. molybdenum carbides), grain boundaries or other discontinuities.

Commercial HSS typically achieves this through a combination of strengthening mechanisms. These include:

- Grain refinement - the process of producing a microstructure with fine grains which, in turn, results in more grain boundaries. Fine grains lead to an increase in yield strength because the greater number of grain boundaries tends to relax the movement of dislocations through the material.
- Solid solution strengthening –when the movement of dislocations is slowed through active distortion of the iron crystal lattice.

- Precipitation hardening – increasing the strength due to the precipitates directly obstructing the motion of dislocations as opposed to indirectly through distorting the iron crystal lattice as in solid solution strengthening.
- Strain hardening – introduction of dislocations into the crystal lattice through plastic strain.

2.2 Production routes for HSS

There are a number of different production routes for HSS materials using a variety of heat treatments and rolling procedures to achieve the most desirable properties for a particular application. The addition of alloying elements such as carbon or niobium can add substantial strength also. HSS is traditionally hot rolled in the austenitic region which is typically above 900 ^{o}C. The steel is then cooled at a specific rate to achieve the desired mechanical properties [1].

The structural steel grades in Europe (e.g. S355N, S460Q, etc.) are generally denoted by an S at the beginning of their designation followed by the minimum yield strength in N/mm^2 and then the production route/delivery condition, where N, Q, M and C are used for materials that are normalised (N), quench and tempered (Q), thermo-mechanically rolled or thermo-mechanically control processed (M) and cold-formed (C), respectively. More information on these processes is available elsewhere [1].

2.3 Material properties of HSS at ambient temperature

For high strength structural steels with a yield strength greater than 460 N/mm^2, the quench and tempering heat treatment method is the most common production process employed. The European material standard for hot-rolled structural steel [21] includes seven different HSS quench and tempered (Q) grades, namely S460Q, S500Q, S550Q, S620Q, S690Q, S890Q and S960Q. The current study focusses on the quenched and tempered HSS grade S690Q which is selected because it is the highest strength grade currently included in Eurocode 3 and is currently available from steel producers in the UK and Europe. The chemical composition and minimum mechanical properties of S690Q, in accordance with the European material standard [21], are presented in Table 1 and Table 2, respectively. In Table 2, the term So refers to the gauge length of the specimen.

Typical stress strain curves for S460 and S690 steels are illustrated in Fig. 1. The decrease in ductility with increasing strength can be seen in Fig. 1, which was taken from a recent study into HSS beams at ambient temperature [10], where it was shown that the fracture strain for the S460 flat sample is around 0.23 whereas the equivalent value for S690 is around 0.13. From a design perspective, this low ductility can have a negative effect on the deformation capacity of the material unless carefully considered in design.

2.4 Material properties of HSS at elevated temperature

The mechanical properties of steel including the yield (f_y) and ultimate (f_u) strengths as well as elastic modulus (E) decrease with increasing temperature. This reduction is typically represented in design codes through the use of reduction factors which indicate how a particular property diminishes with increasing temperature, relative to its original value at ambient temperature. The Eurocode does not present reduction factors specifically for HSS and designers are directed to use the same values as for normal strength steel, although many researchers have shown that these materials behave in a different manner when subjected to elevated temperatures (e.g. [1, 14, 15]).

In Eurocode 3, Part 1-2 [18], the effective yield strength at a temperature θ ($f_{y,\theta}$) is based on the total strain level at 2.0%. The reduction factors for this term (i.e. $k_{y,\theta} = f_{y,\theta}/f_y$) obtained by a number of researchers (e.g. [14, 15]) are compared with the reduction factors given in the Eurocode and presented in Fig. 2. These reduction factors were achieved either through steady-state testing, whereby the specimen temperature is equilibrated at the target temperature before straining to failure at a controlled rate, or transient testing, implying that the specimen is held at a target load and then the temperature is increased at a controlled rate until failure occurs. With reference to hte figures presented in Fig. 2, it is clear that further experimental investigation into the validity of the reduction factors given in Eurocode 2 Part 1-2 for high strength steel is required; this is currently ongoing at Brunel University London. The chemical composition of the steels used in these experiments, where available, are given in Table 3.

2.5 Design guidance for HSS structural elements

The European design specification for the fire design of steel structures, Eurocode 3 Part 1-2 [18], was limited to ordinary structural steels with a yield strength of up to 460 N/mm^2 until additional rules were published for HSS in 2007 [22]. This extension supplements existing codes for mild steels to cover steel grades with a yield strength of up to 700 N/mm^2 and has been supported by an increase in research focus on the behaviour and design of HSS structures at ambient temperatures. However, despite the publication of this extension, it states that Eurocode 3 Part 1-2 is still applicable for the fire design of structural steels up to S700 although the material data presented is based on data from tests on mild steel only. There has been limited research into the applicability of these design rules for HSS, mainly owing to the significant expense associated with high temperature structural testing as well as a lack of reliable data on the material properties which are needed to develop computational design models. These factors hinder the use of HSS in structural design but are currently being studied by different research groups around the world.

2.6 Key issues for using HSS in structural applications

High strength steels are typically used in structural applications where the design is governed by strength rather than by stiffness. Widespread use in civil engineering structures has been limited to specific applications such as offshore drilling rigs, bridges and long span trusses, mainly owing to a lack of reliable design guidance as well as perceived serviceability issues, different welding procedures and misconceptions about the cost. Under fire conditions, both the strength and stiffness of the material decrease as the temperature increases, but the degree to which this occurs specifically for HSS, is still under discussion and the subject of considerable research. A key step towards greater application of HSS in structural applications is the development of appropriate and reliable design rules for high strength steel members under fire conditions, particularly when design is governed by stiffness rather than strength.

3. Finite element model

A three-dimensional finite element (FE) model has been developed using the general purpose program ABAQUS [19] to investigate the behaviour of high strength steel beams. The ABAQUS software was selected as it is commercially-available and is capable of depicting the material and geometric nonlinearities as well as the elevated temperature behaviour accurately. In this section, the development of the model is first described followed by a discussion on the validation. Thereafter, the model is used to investigate the behaviour of HSS beams at elevated temperature. The FE model represents both material nonlinearity and initial geometric imperfection (out-of-straightness) of the beam, as well as residual stress distribution in the beam cross-section.

3.1 Development of the FE model

3.1.1 FE element type and mesh

The finite element model uses three-dimensional deformable solid elements with reduced integration (C3D8R) to represent the steel section. The C3D8R is a 3-dimensional tetrahedral element which was selected because it provides excellent accuracy at relatively low computational cost, and also provides stress, strain and temperature distributions in the members. Reduced integration is used for computational efficiency. The beam is assumed to be simply supported at the ends. A suitable mesh density has been selected based on a sensitivity study in which the accuracy of the results was assessed together with practical computational times. As shown in Fig. 3, a suitable mesh size was determined to be 12 elements in the flange width and 2 through the flange thickness, 22 elements in the web depth and 2 through the web thickness and 100 elements in every 10 meters across the length.

In the model, a linear elastic buckling analysis is first conducted using the BUCKLE procedure in ABAQUS to determine the buckling mode shapes. These mode shapes are then incorporated, one as geometric imperfections into the load-displacement nonlinear analysis. This is completed using the Riks method, which accounts for both geometric and material nonlinearity and is a variation of the classic arc length method. Just one mode shape, the buckling mode shape, is incorporated into the model. The temperature is considered in the model in an instantaneous isothermal manner. Accordingly, the target temperature (e.g. 200 °C, 300 °C, etc.) is set using a predefined field as the initial temperature. Although this is not necessarily representative of a real fire scenario which would heat up in an anisothermal manner, the isothermal test conditions have been used by many researchers (e.g., [15, 23, 24] for numerical and experimental studies as it tends to be a reliable and controllable approach making validation more straight forward. The model Abaqus/Standard analysis method is used in this study.

3.1.2 Elevated temperature material modelling

In this analysis the nominal stress-strain curves were converted into true stress-plastic strain curves which were then used in the analysis. The elevated temperature material properties of the steel are characterised in accordance with the nonlinear stress-strain curve at elevated temperature for hot rolled steel (normal strength and high strength) from Eurocode 3 Part 1-2 [18], as shown in Fig. 4. This model is defined by the equations given in Table 4. In this table, $f_{p,\theta}$, $f_{y,\theta}$ and $E_{a,\theta}$ represent the proportional limit (i.e. where the stress-strain response becomes nonlinear in the elastic range), effective yield strength and slope of the linear elastic range, at a temperature θ, respectively. On the other hand, $\varepsilon_{p,\theta}$ and $\varepsilon_{y,\theta}$ correspond to the strain at the proportional limit and yield point at a temperature θ, respectively, whilst $\varepsilon_{t,\theta}$ and $\varepsilon_{u,\theta}$ are the limiting strain for yield strength and the ultimate strain at a temperature θ, respectively. In this analysis, the limiting strain values are adopted from those given in Eurocode 2 Part 1-2. However, it is noteworthy that these values for high strength steel should be adopted with due care as these materials can sometimes be less ductile compared with normal strength steel.

Reduction factors, also given in the Eurocode, are employed to depict the degradation of the proportional limit ($k_{p,\theta} = f_{p,\theta}/f_p$), yield strength ($k_{y,\theta} = f_{y,\theta}/f_y$) and Young's modulus ($k_{E,\theta} = E_{a,\theta}/E$) at various levels of elevated temperature; these reduction factors are presented in Fig. 5. It is assumed that the elastic material properties including Young's modulus (E), shear modulus (G) and Poisson's ratio (υ) are isotropic and that Poisson's ratio does not change with temperature. The temperature-dependent thermal expansion of the steel during heating is modelled using the method included in Eurocode 3 Part 1-2.

3.1.3 Consideration of imperfections and residual stresses

The influence of both global and local initial geometric imperfections is included in the model by factoring the appropriate mode shape from the eigenvalue analysis by a pre-defined amplitude. This amplitude (u) at a given location along the length of the member (z) is approximated using the expression given in Eq. (1):

$$u(z)=a\sin\left(\frac{\pi\times z}{L}\right) \tag{1}$$

where a is the maximum amplitude of the lateral imperfection and L is the overall element length. The first mode, which is the lateral torsional buckling mode, accounts for the geometric imperfections in the member.

Residual stresses, which are internal stresses that exist in a member in its unloaded state due to the manufacturing process, are also accounted for. Although the residual stress pattern in a cross-section is a continuous function, the values adopted are imported into the ABAQUS model as a linear line distribution whereby the cross-section of the beam is partitioned and each partition assigned a predefined initial residual stress value. More discussion on the depiction of geometric imperfections and residual stresses is presented later in this paper.

3.2 Validation of the FE model

The FE model described in the previous sub-section has been employed to analyse the simply supported beams tested by Vila Real *et al*. [23]. Although these experiments were conducted on beams made from normal strength steel, they are used in the current paper for validation of the model owing to a lack of experimental results for HSS beams in fire. The beams have an IPE100 cross-section as shown in Fig. 6 and three different lengths were included in the programme, namely 500 mm, 2500 mm and 4500 mm. The average cross-sectional dimensions measured during the test programme are applied in the FE model. A moment-displacement curve is obtained from ABAQUS for each analysis and the LTB resistance is considered to the be the load achieved before the response becomes unstable.

3.2.1 Loading and support conditions

The support and loading conditions are replicated in the numerical model, as shown in Fig. 7. In this figure $U_{m,1}$, $U_{m,2}$ and $U_{m,3}$ represent the lateral, vertical and longitudinal displacements permitted in the beam, respectively, at the location m, where m is either A, B or C as shown in the diagram. On the other hand, $\theta_{m,1}$, $\theta_{m,2}$ and $\theta_{m,3}$ are the rotations in the lateral, vertical and longitudinal directions at the same locations. A concentrated load of 2000N is applied at the edges of the beam cross-section at both ends. As stated before, the Riks method is applied in ABAQUS and the initial, minimum and maximum arc lengths are 0.01, 10^{-15} and 10^{36}, respectively. These edges are constrained through kinematic coupling in order to prevent possible local deformations at the loading points.

In terms of the temperature loading, this is applied to the beam model in an instantaneous isothermal manner, to replicate the test procedure. During the experiments, the temperature was measured at several locations along the length of the beam and the load was applied once the beam had stabilised at the target temperature throughout. Accordingly, the target temperature (e.g. 200 °C, 300 °C, etc.) is added to the model using a predefined field as the initial temperature. Although a detailed heat transfer model is more representative of a real fire, as stated before, this is not incorporated into the model in order to provide an accurate replica of the tests and also for computational efficiency. The numerical model is used to assess the response at six different temperatures including 20 °C (i.e. ambient temperature), 200 °C, 300 °C, 400 °C, 500 °C and 600 °C, in accordance with the test programme.

3.2.2 Material properties

The material properties are adopted in accordance with those reported [23] and therefore the yield strength and elastic modulus are taken as 320 N/mm^2 and 221000 N/mm^2, respectively. The initial out-of-straightness geometric imperfections were measured during the experiments and the maximum amplitudes are presented in Table 5. These are incorporated into the model using Eq. (1), as previously discussed.

3.2.3 Residual stresses and imperfections

The residual stresses incorporated in to the FE model are taken directly from the data measured in the test programme. The authors conducted a detailed study to determine the residual stress pattern using the drill hole method. The average axial residual stress values and their distributions on the cross-section are presented in Fig. 8(a), including the maximum tensile residual stresses σ_{rtf} and σ_{rtw} at the centre of the flanges and web, respectively, as well as the maximum compressive residual stresses σ_{rcf} and σ_{rcw} at the edges of the flanges and web, respectively. The distribution of the residual stresses on the cross-section are idealised in the model as shown in Fig. 8(b), using discrete subsections.

3.4.4 Results from the verification

The predictions from the FE model are presented together with the experimental results in Figs. 9-14 in terms of lateral-torsional buckling reduction factor values (χ_{LT} and $\chi_{LT,fi}$ at ambient and elevated temperature, respectively) for different slenderness values ($\bar{\lambda}_{LT}$ and $\bar{\lambda}_{LT,fi}$ at ambient and elevated temperature, respectively) in the tested temperature range. The LTB resistance load is defined in this analysis as the load at which an increase in the displacement value is not accompanied by a further load increase. It is noteworthy that three repetitions were conducted at each temperature level in the experimental programme (i.e. EXP1, EXP2 and EXP3), all of which are included in the figures.

Also included in the figures are the Eurocode predictions for the bucking reduction factor. At ambient temperature, the lateral-torsional buckling reduction factor (χ_{LT}) and the non-dimensional slenderness ($\bar{\lambda}_{LT}$) are determined from Eqs (2) and (3), respectively:

$$\chi_{LT}=\frac{M_{b,Rd}}{M_{c,Rd}} \tag{2}$$

$$\bar{\lambda}_{LT}=\sqrt{\frac{W_{y,pl}\times f_y}{M_{cr}}} \tag{3}$$

where $M_{b,Rd}$ is the design buckling resistance moment, $M_{c,Rd}$ is the design resistance for bending, $W_{y,pl}$ is the plastic section modulus of the cross-section and M_{cr} is the elastic critical moment for lateral-torsional buckling. For these tests, M_{cr} was determined using Eq. (4):

$$M_{cr}=\alpha_m \frac{\pi^2 EI_y}{(kL)^2}\sqrt{\left(\frac{k}{k_w}\right)^2\frac{I_w}{I_y}+\frac{(kL)^2 GI_t}{\pi^2 EI_y}} \tag{4}$$

In this expression, α_m is the moment distribution factor for different lengths as given in Fig. 15 [23]. E and G are the elastic and shear modulus, respectively, k is the effective lateral-torsional buckling length factor which is taken as 0.5 in this study because the load application points were laterally restrained and k_w is the factor which accounts for the beam end warping, taken as unity in this study. I_y, I_w and I_t are the second moment of the cross-section area for the minor axis, the warping constant and the torsional constant of the beam cross-section, respectively.

The cross-section examined in this programme is Class 1, in accordance with EN 1993-1-1, and the reduction factors for lateral torsional buckling are determined in accordance with EN1993-1-1 clause 6.3.2.3 [17]. Since the section is a rolled I-section, buckling curve ‘b’ is adopted. It is noteworthy that the Eurocode presents a total of seven buckling curves for LTB, from which the buckling capacity can be derived by applying a reduction factor to the moment capacity. The buckling curves are divided into two groups, with four curves in clause 6.3.2.2 (general case) and the remaining three presented in clause 6.3.2.3 (specifically for rolled sections and equivalent welded sections). In general, clause 6.3.2.3 is more advantageous for rolled sections [25] and is therefore used in the current work.

At elevated temperature, the reduction factor for lateral torsional buckling in the fire design situation ($\chi_{LT,fi}$) and the non-dimensional slenderness values ($\bar{\lambda}_{LT,fi}$) are determined using the following expressions:

$$\chi_{LT,fi}=\frac{M_{b,fi,t,Rd}}{M_{fi,\theta,Rd}} \tag{5}$$

$$\bar{\lambda}_{LT,fi}=\bar{\lambda}_{LT}\sqrt{\frac{k_{y,\theta}}{k_{E,\theta}}} \tag{6}$$

where $M_{b,fi,t,Rd}$ is the design lateral torsional buckling resistance moment and $M_{fi,\theta,Rd}$ is the fire design moment resistance.

The FE results are presented together with the experimental data and Eurocode predictions in Fig. 9 at ambient temperature and Figs. 10-14 at 200°C, 300°C, 400°C, 500°C and 600°C, respectively. Comparison of the FE predictions with the experimental results clearly shows that the model is capable of depicting the lateral torsional buckling behaviour with reasonable accuracy. It is noteworthy that although there is some deviation between experimental and numerical results in a small number of cases, there is also notable differences between the three repetitions of the same experiment at elevated temperature. This highlights the difficulty in observing reliably consistent behaviour during elevated temperature structural tests. In all cases, even in the higher temperature range, the FE predictions fall within a reasonable margin of the test results. Based on this, the following section uses the FE model to investigate the performance of beams using HSS under fire conditions.

4. HSS beams in elevated temperature conditions

In the previous section, the verification exercise showed that the FE model can reasonably depict the behaviour of normal strength steel beams under elevated temperature conditions. Therefore, in this section, the model is used to study the lateral torsional buckling behaviour of simply supported laterally unrestrained beams made from high strength steel, at elevated temperature. There are no available test results for HSS beams in fire.

4.1 Material properties

The beams are made from S690Q high strength structural steel and hence, the yield strength, ultimate strength and elastic modulus are taken as 690 N/mm^2, 770 N/mm^2 and 210000 N/mm^2, respectively, in accordance with [17]. At elevated temperature, the stress-strain response presented in Fig. 4 is adopted, similarly to normal strength steels (NSS), and the reduction factors presented in EN 1993-1-2 [18] are employed. As mentioned previously, the Eurocode addition for HSS [22] states that EN1993-1-2 is applicable for steels up to S700 without any additional rules or changes.

4.2 Behaviour of HSS beams

The lateral-torsional buckling behaviour of S690Q high strength steel beams with the IPE 100 cross-section (as shown in Fig. 6) are studied at ambient temperature and also at 200°C, 300°C, 400°C, 500°C, 600°C and 700°C. A range of different beam lengths are included between 500 and 4500 mm. The loading and support conditions, as well as the thermal properties such as specific heat and thermal conductivity, are identical to those employed in the NSS beam analysis previously described. It is also assumed that the geometric imperfections, residual stress values and distributions are the same as for the normal strength steel beams. It has been shown elsewhere that no bending residual stresses exist [10] and therefore only axial membrane residual stresses are likely in the section. A representative graphical image from a finite element analysis simulation is presented in Fig. 16; note that the scale is exaggerated in this image for presentation reasons. The results obtained from the ABAQUS analyses are compared hereafter with the lateral-torsional buckling design curves in Eurocode 3 Part 1-1 (ambient) and Part 1-2 (elevated temperature), and the applicability of the design rules at elevated temperatures are investigated.

4.3 Ambient temperature

The cross-section used in this analysis is Class 1, in accordance with Eurocode 3, and as discussed previously in the normal strength steel analysis. The reduction factors for lateral-torsional buckling at room temperature (χ_{LT}) obtained for different slenderness values ($\bar{\lambda}_{LT}$) are calculated according to Eurocode 3, Part 1-1 clause 6.3.2.3 [17]. Since the section is a rolled I-section, buckling curve ‘b’ is adopted.

The results obtained from ABAQUS are presented in Fig. 17, together with the Eurocode predictions. For comparison, the graph includes all seven of the Eurocode buckling curves, in order to illustrate the relative performance of the HSS beams. As presented in the figure, clause 6.3.2.3 generally gives less conservative buckling curves compared with clause 6.3.2.2. Nevertheless, in this instance, for all but one of the tested beams, current Eurocode rules (that is curve ‘b’ using clause 6.3.2.3), provide an unconservative depiction of the buckling response for this particular configuration of HSS beam. Indeed, all of the Eurocode buckling curves are shown to give unsatisfactory predictions. This implies that the numerical model predicts that the beam will fail by lateral torsional buckling at a load lower than that predicted by the Eurocode. Further experimental and numerical work is required in order to fully investigate the applicability of current Eurocode design procedures, which is currently underway, but these results indicate that current design provision are unsatisfactory.

Also included in Fig. 17 is a proposed curve for these HSS beams, which have a Class 1 cross-section and a height to breadth ratio of ≤ 2). This curve employs a plateau length, $\bar{\lambda}_{LT,0}$ (i.e. region at low slenderness when $\chi_{LT} = 1$), of 0.3 (as opposed to 0.4 for NSS in the current Eurocode) and an imperfection factor (α_{LT}) of 1.5 (as opposed to 0.34, 0.49 and 0.76 for curves ‘b’, ‘c’ and ‘d’, respectively, in the current Eurocode). Further experimental and numerical analysis is required in order to validate and verify this proposal.

4.4 Elevated temperature

The lateral torsional buckling reduction factor in fire conditions ($\chi_{LT,fi}$) has been obtained using the numerical model for a range of slenderness values ($\bar{\lambda}_{LT,fi}$) at 200°C, 300°C, 400°C, 500°C, 600°C and 700°C, respectively. The results are presented in Fig. 18, together with the unfactored Eurocode predictions which are determined in accordance with clause 4.2.3.3 of Eurocode 3 Part 1-2 [18], using Eqs (5) and (6) presented before. The reduced material properties for the high strength steel at elevated temperatures are as shown in Fig. 5.

It is clear that, in general, the Eurocode depicts the behaviour reasonably well. However, at relatively higher slendernesses ($\bar{\lambda}_{LT,fi} > 1.2$), the standard does not always provide a safe prediction. Therefore, also included in the graph is a new proposed buckling curve which, subject to further investigation, may be more appropriate for high strength steel beams in fire conditions. In the current Eurocode 3 specification, the imperfection factor α which is used to determine $\chi_{LT,fi}$ is calculated as:

$$\alpha=0.65\sqrt{235/f_y} \tag{7}$$

On the other hand, the proposed curve is calculated in accordance with Eq. (8):

$$\alpha=1.5\sqrt{235/f_y} \tag{8}$$

This proposal requires further validation for a wider range of beam profiles, as well as experimental verification but provides and essential first step towards the development of improved design rules for laterally unrestrained HSS beams in fire.

5. Conclusions

High strength steel in an increasingly common material choice in structural engineering applications owing to their favourable strength to weight ratio. However, these grades are generally under-used in load-bearing structures owing to a lack of reliable and efficient information relating to structural behaviour available in the literature and design standards. In addition, there is a general misconception that HSS is significantly more expensive than normal strength steel, which is not the case, particularly when the reduced material usage is taken in to account. One area that is particularly lacking in performance information is the response of HSS steel members under fire conditions. This is a complex subject owing to the many inter-related parameters which influence the behaviour. In this context, the current paper has presented an investigation into the lateral torsional buckling behaviour of unrestrained high strength steel beams at elevated temperature.

A geometrically and materially nonlinear finite element model was developed and described in detail. It was verified using experimental data from a series of tests on normal strength steel beams under fire conditions [23]. To date, there are no experimental data for HSS beams in fire available in the literature. The beams included in this study were between 500 and 4500 mm in length and each was subjected to temperatures between 20°C and 700°C. The results obtained from the numerical analysis were compared with the predictions of current European structural fire design specification of steel structures [18] to assess the applicability of the current design specification.

The results showed that the Eurocode buckling curves are not satisfactory in their current format for high strength steel beams, both at room and elevated temperature. Currently, there is no distinction made in the design code for lateral torsional buckling resistance between high strength and normal strength beams; the buckling curves are the same. Although HSS has a higher strength compared with normal strength steel, the elastic stiffness is the same. Therefore, HSS beams are more likely to fail by stability issues rather than attainment of their ultimate strength. The results of this study show that this may be an unsafe assumption, which requires further investigation, beyond the scope of the current study.

At elevated temperature, not only are the buckling curves identical between HSS and normal strength steel, but the elevated temperature material strength and stiffness reduction factors are also unchanged. Although the same reduction factors were used in the numerical and design analysis in the current study, it is accepted that the current design material reduction factors are inappropriate for HSS [1]. The authors are currently conducting a major experimental programme investigating the material response of HSS to elevated temperature. In addition, based on the conclusions from the current work, further investigations into the response of unrestrained HSS beams in fire are being conducted. This includes studies into the effect of beam cross-section, support conditions and loading on the susceptibility of the beam to lateral torsional buckling at elevated temperature.

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