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Effect of Local Instability on Capacity of Steel Beams Exposed to Fire

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1.0 ABSTRACT

This paper presents results from numerical studies on the behavior of fire exposed steel beams by taking into consideration temperature-induced sectional instabilities. A three-dimensional nonlinear finite element model is developed to evaluate the response of fire exposed steel beams under both flexural and shear effects. This model is applied to investigate the effect of sectional slenderness on the onset of local instability and capacity degradation in steel beams exposed to fire. Results from finite element analyses are utilized to evaluate failure of beams under different limit states including flexure, shear, sectional instability and deflection criteria. These results show that under certain loading scenarios and sectional configurations, shear capacity in steel beams can degrade at a higher pace than that of moment capacity. In addition, results from numerical studies infer that room temperature classification of steel beams based on local stability, can change with fire exposure time; a compact section at ambient conditions can transform to a non-compact/slender section under high temperature effects. This can induce temperature-induced local buckling in steel sections and lead to failure prior to attainment of failure under flexural yield and/or shear limit state.

Keywords: Local buckling, fire resistance, steel beam, shear, finite element analysis

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2.0 INTRODUCTION

Structural members, when exposed to fire, experience loss of capacity and stiffness due to temperature induced degradation in strength and modulus properties of constituent materials. When the moment capacity at the critical section of a beam drops below the applied moment due to loading, failure occurs. The time to reach this failure is referred to as fire resistance. In contrast to ambient temperature design philosophy, where a beam is generally designed to satisfy flexural limits state, and then checked for shear resistance, failure of beams under fire conditions is typically derived based on flexural limit state only [1]. In fact, provisions in current fire design standards neglect the effect of shear and sectional instabilities in evaluating failure of steel beams. Although deriving failure of beams based on flexural limit state is valid for most common scenarios, this assumption may not be representative in certain situations where shear and instability effects can be dominant in a fire exposed member [1]. Shear and instability effects can be predominant in beams when concentrated loads are placed near end supports; as in the case of beams connecting offset columns, transfer beams, coped beams (with notched ends), deep beams and plate girders [2-4].

Steel beams used in most practical applications are often of W-shaped sections which have thinner and deeper webs (more slender) than flanges. Also, in such members, a larger portion of web surface area (compared to flanges) is exposed to fire. Due to these reasons, webs in W-shaped steel sections experience faster rise of temperature as compared to flanges [5]. Such faster rise in web temperatures lead to rapid degradation in shear capacity (due to temperature-induced strength losses) as compared to deterioration of moment capacity in a beam. This

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accelerated degradation in shear capacity, when accompanied with high shear loading, can initiate sectional instability under fire conditions.

Temperature induced sectional instability occurs in steel beams when internal stress build-up approaches limiting yield strength. At this point, stiffness properties and strength of steel starts deforming which produces sectional instability (local buckling). In beams, local buckling can occur in flanges or in web and onset of local buckling reduces effective area, which in turn decrease flexural and/or shear capacity under room or fire conditions. In beams subjected to high shear forces, a combination of accelerated fire-induced strength degradation and temperature-induced instability due to web local buckling can cause pre-mature failure of beams through shear limit state.

A review of literature indicates that most of the previous studies on steel beams investigated flexural behavior of beams when exposed to fire conditions [6-9]. For instance, Newman conducted full scale fire resistance experiments on steel beams at the Cardington facility [6]. Data from these tests is utilized by various researchers for validating computer models on tracing the response of steel framed structures under fire conditions. However, effects of dominant shear loading or sectional instability (local buckling) on performance of fire exposed steel structures were not considered. A recent study carried out by Dwaikat and Kodur developed a performance based approach for assessing fire resistance of restrained beams [7]. This approach takes into consideration the influence of critical factors including fire scenario, end restraints, thermal gradients, load level, and failure criteria in evaluating fire resistance (flexural only) of restrained steel beams. The authors infer that in axially restrained steel beams,

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the influence of flange local buckling on the fire response is insignificant due to the development of tensile catenary action. However, effect of shear loading, web local buckling and instability of steel beams at elevated temperatures is not taken into consideration in evaluating fire response of steel beams [6-9].

In a recent study, Kodur and Naser applied a three dimensional finite element model to study shear response of steel beams exposed to fire [1]. The effect of different loading patterns, web slenderness and presence of fire insulation on the behavior of fire exposed steel beams subjected to high shear loading was studied. Based on these studies the authors reported that under certain scenarios, the shear capacity can degrade at a higher pace than flexural capacity, thus providing failure through shear.

The occurrence of local buckling in steel members under high temperature conditions has been studied by few researchers [10-13]. Uy and Bradford and Heidarpour and Bradford applied finite strip method to study effect of local buckling in cold formed steel structural members subjected to elevated temperatures. Results from these studies show that presence of high flexural and shear loading can significantly influence the onset of local buckling in the web. Zhao and Kruppa conducted fire tests to study the flexural behavior of fire exposed composite steel beams [12]. The authors reported that these composite beams can experience local buckling at interior supports. However, no further recommendations were discussed, possibly because tested beams were subjected to dominant moments and thus the effect of shear and fire-induced instability could not be isolated. Recently, Wang et al. [13] presented an experimental investigation on the local buckling phenomenon in flange and web of thin-walled cross sections

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subjected to elevated temperatures. Data from these tests show that buckling resistance of stub columns decrease with increase in temperature and Eurocode 3 provisions predict higher buckling load in columns than those observed in fire tests.

As discussed above, a review of literature clearly show that most previous studies mainly focused on fire behavior of steel beams subjected to bending [6-12]. The effect of shear and local buckling parameters on response of steel beams under fire conditions is not addressed. To evaluate the effect of local buckling on response of a fire exposed steel beam, a numerical study is carried out using a three-dimensional nonlinear finite element model. The developed model can trace the fire response of hot-rolled W-shaped steel beams subjected to significant bending moment and shear loading. The model is applied to examine effects of sectional slenderness on the onset of local instability and capacity degradation in steel beams exposed to fire and structural loading.

3.0 EFEFCT OF SECTIONAL INSTABILITY ON FLEXURAL AND SHEAR CAPACITY

The response of steel beams is highly influenced by the onset of local buckling in flange or web at critical sections. This fact is well recognized in codes and standards and is taken into account while evaluating flexural and shear capacity at room temperature [14, 15]. For example, AISC design manual, classifies cross-sectional shapes as compact, non-compact and slender based on sectional slenderness (width-to-thickness ratio (λ)) of flange and web [14]. This slenderness ratio is usually compared against two upper limits; compact (λ_p) and non-compact (λ_r). If the sectional width-to-thickness ratio is less than compact limit, then the section is considered compact. However, if λ lies in between compact and non-compact limits, the section

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is classified as non-compact. Finally, a slender section is that with λ exceeding the limit of non-compactness.

Table 1 presents the above discussed width-to-thickness limiting ratios under flexural and shear limit states for W-shaped steel sections at room temperature. It is clear that web slenderness can influence both flexural and shear capacities since web dimensions are accounted for in section modulus (S or Z for flexural calculation) and in web area (for shear calculation). Thus, web slenderness, as well as rate of temperature rise in web, can significantly affect flexural and shear response of steel beams.

Furthermore, it can be also seen that slenderness limits are a function of stiffness and strength properties ($\sqrt{\frac{E}{f_y}}$) of steel. In steel beams exposed to fire, increased temperature induces loss of strength and stiffness in steel (see Fig. 1). Strength and modulus of steel starts to degrade at about 400 and 150˚C, respectively, and these properties degrade at different rates. Since provisions in design codes provide no recommendations for classification of steel beams under fire conditions, if room temperature classification limits are applied to evaluate local buckling under fire conditions, these limits can vary (reduces) according to net loss of steel strength and modulus. Since stiffness properties starts to degrade earlier to strength properties (and at a faster pace), local buckling phenomenon in steel can start even when steel temperature reaches 150˚C. This can induce reduction in flexural and/or shear capacity at lower temperatures due to occurrence of buckling of flanges/web. This loss in capacity is in addition to temperature-induced strength degradation in steel.

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Figure 2 illustrates variation of width-to-thickness classification limits in a steel beam made of Grade 50 steel (345 MPa) with temperature. As shown in Fig. 2a, there is only slight variation in compactness and non-compactness limits of flange with increases temperatures. However, as can be seen in Fig. 2b, web slenderness limit used for flexural and shear capacity evaluation vary significantly with increasing temperatures. It can be also seen in Fig. 2b that web slenderness limits for shear capacity evaluation have significantly smaller range ($59 \leq \lambda \leq 77$) than that for web slenderness limits used in flexural evaluation ($90 \leq \lambda \leq 137$). Hence, local buckling of web (under fire conditions) is expected to be very sensitive to shear loading.

Flange and web (sectional) slenderness are compared against slenderness limits (given in Table 1) to classify "shape of" steel beams at ambient conditions. Sectional width-to-thickness ratio (λ) depends only on the geometrical features (dimensions) of sections. Thus, these ratios (λ_{flange} and λ_{web}) remain constant under fire exposure. As discussed above, compactness and non-compactness degrade under fire conditions, hence, sectional slenderness (λ_{flange} and λ_{web}) can (sometimes) exceed limiting values at a given temperature. For instance, for a compact steel section exposed to fire, once λ_{flange} and/or λ_{web} exceed temperature dependent compactness limit, classification of this steel section can change from that observed at room temperature.

To further illustrate this phenomenon, a compact section at room temperature can transform to a non-compact/slender section under fire conditions. For example, web slenderness limit used in shear evaluation (λ_p) of Grade 50 (345 MPa) steel is $\lambda_p = 1.10\sqrt{\frac{k_v E}{f_y}} = 59.24$ at ambient conditions and if the reduced properties of steel (E_T and f_{yT}) is taken into consideration, this slenderness limit reduces to 51.9 at 500˚C. For a W16×31 section made of the same steel

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grade, web slenderness (λ_{web}) is 57.8 which is less than plastic limit (λ_p) at ambient conditions and hence is classified to be a compact web. However, when web temperature reaches 500˚C, web slenderness exceeds that of web slenderness limit at 500˚C and hence the compact web at room temperature transforms into a non-compact web when temperature reaches 500˚C. On the other hand, flange slenderness plastic limit of Grade 50 (345 MPa) steel is; $\lambda_p = 0.38\sqrt{\frac{E}{f_y}} = 9.15$ and 8.03 at 25 and 500˚C, respectively. For a W16×31, flange slenderness (λ_{flange}) is 6.28. Hence flange slenderness do not exceed that of the flange slenderness plastic limit at 500˚C, thus flanges of this section remain compact even at 500˚C. In fact, flanges of this beam remain compact throughout the temperature range of 20-800˚C.

The above illustration indicate that temperature-induced losses in shear capacity in steel beams can result from both strength degradation, as well as occurrence of local buckling of web, when the section transform from compact to non-compact. In the case of a steel beam with W16×31 section, the reduction in shear capacity due to strength degradation is 22% (shear capacity reduces from 583.5 to 455.1 kN). When local buckling effects is taken into account, reduction in shear capacity increases to 37% (shear capacity further reduces from 455.1 to 368 kN) at 500˚C. However, reduction in moment capacity in this section results only from degradation in strength of steel (no local buckling effect) and this reduction is 22% (305 to 238 kN.m) at 500˚C. This infers that local buckling effect can lead to significant reduction in shear capacity of steel beams.

The effect of sectional instability on shear capacity of a steel beam subjected to fire loading is shown in Fig. 3. The bending moment after 2 hours of fire exposure is lower than

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moment capacity of this beam, thus the beam does not fail in flexural mode. However, shear capacity at 2 hours of fire exposure falls below that of shear forces due to loading, leading to failure of this beam in shear limit state. Moreover, when instability effects are considered, shear capacity further reduces to account for temperature-induced instability losses (web local buckling). It should be noted that instability-based reduction in capacity is generally localized at critical sections (i.e. point of high shear force), however these reductions are assumed to be uniform in Fig. 3 for illustration purposes.

4.0 FINITE ELEMENT MODEL

To study the effect of local buckling on the response of steel beams under fire conditions, a finite element model was developed using ANSYS. The three dimensional finite element model, is capable of tracing thermal and structural response of fire exposed steel beams. Critical parameters that influence instabilities including geometric and material nonlinearities, temperature dependent material properties and different modes of failure states are accounted for in the model to trace the realistic response of fire exposed steel beams. The analysis was carried out at various time steps, by incrementing time, from pre-loading stage till failure of the beam. Further details on model development, discretization, modeling techniques, and contact definition are provided in subsequent sections.

4.1 Discretization of beam

To simulate the response of a steel beam subjected to thermal and structural loading, the beam is discretized with two sets of elements available in ANSYS 14.0. SHELL131, SOLID70, LINK33 and SURF152 elements are used as thermal elements to simulate heat transfer between fire source and steel girder [16]. SHELL131 is a 3-D layered shell element having in-plane and through-thickness thermal conduction capability. SOLID70 is an eight-noded (cubic) thermal element with conduction capability. LINK33 which has a single degree of freedom, namely temperature at each node, is a uniaxial element with the ability to conduct heat between its nodes. SURF152 is a four-noded (surface) thermal element capable of simulating heat conduction, convection and radiation. SURF152 is overlaid on top of SHELL131 and SOLID70 elements to simulate convection and radiation heat transfer from fire zone to structural member (beam). It should be noted that both convection and radiation are primary heat transfer mechanisms from fire zone to the exposed surface of the beam; while conduction is the main heat transfer mechanism within the cross-section of the beam. A convection coefficient of α_c= 25 W/(m^2°C) and Stefan-Boltzmann radiation constant of 5.67x10^{-8} W/(m^2°C) was applied in the thermal analysis.

For undertaking structural analysis, the steel beam is discretized into SHELL181, SOLID65, LINK8, COMBIN39, BEAM188, CONTA173 and TARGE170 elements. SHELL181 element, used to model steel beam, has four nodes with six degrees of freedom per node, three translations in x, y, and z directions and three rotations about x, y, and z-axes. This element can capture local buckling of flanges and web and also lateral torsional buckling of the beam and therefore is well-suited for large rotation, large strain and nonlinear problems. SOLID65 is used for three-dimensional modeling of solid concrete slab with or without reinforcing bars. SOLID65 is capable of accounting for cracking in tension and crushing in compression of concrete.

LINK8, used to model steel reinforcement in concrete slab, is a 3D two node uniaxial tension-compression spar element with three degrees of freedom at each node, translations in the nodal x, y, and z directions. This element is used to model the internal (steel) reinforcement embedded in concrete slab. BEAM188, used to model steel studs, is a three dimensional two-noded beam element that has six degrees of freedom at each node; translations in the principle axes and rotations about the principle axes [16]. The shear studs (BEAM188 elements) are embedded in the concrete slab and are assumed to be fully connected to the surrounding concrete. Thus, the nodes of BEAM188 elements are coupled with the nodes of concrete elements (SOLID65). BEAM188 elements are also connected to top flange of steel beam. At the interface of shear studs and steel beam, the coinciding nodes; of BEAM188 and SOLID65 elements, were connected using spring elements (COMBIN39). COMBIN39 is a uni-axial compression-tension element with nonlinear generalized force-deflection capability [16].

In addition, the contact behavior at the interface of concrete slab and top flange of the steel beam was modeled using CONTA174 and TARGE170 elements. The contact interface is defined as surface-to-surface area that only allow sliding of the two adjunct faces. The amount of sliding is governed by Coulomb's frictional law; a coefficient of friction of 0.35 was assumed. Figure 4 shows the element types used in structural analysis as well as a discretized view of the developed model.

4.2 High temperature material properties and constitutive laws

For undertaking fire resistance analysis, temperature-dependent thermal and mechanical properties of steel, concrete, steel reinforcement and fire insulation are to be input to the finite element model. The thermal and mechanical properties of structural steel, concrete and reinforcing steel are assumed to vary with temperature as per Eurocode 3 and 2 provisions [15, 17-18]. Variation of mechanical properties of steel are accounted for through multi-linear stress-

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strain relationship, with kinematic hardening plasticity model as obtained using Eurocode 3 steel material model. The constitutive material model for concrete consist of multiple stress-strain curves that vary with temperature and take into account cracking and crushing of concrete as per Eurocode 2 recommendations. For fire insulation, room temperature thermal properties are used in fire resistance analysis since there is very limited reliable information on variation of thermal properties with temperature. The thermal conductivity, specific heat and density of fire insulation is taken to be 0.0815 W/m.˚C, 1047 J/kg.K and 240 kg/m^3, respectively.

4.3 Failure Criteria

The failure of a beam under fire conditions can occur in different modes and for evaluating realistic failure, different limiting criteria are to be applied. In this analysis, different failure limit states including flexure, shear, local buckling and deflection are considered in checking the failure state of the beam at the end of each time step. Moment and shear capacity at each time step is evaluated utilizing internal bending and shear stresses generated from ANSYS analysis. These stresses, generated at individual elements, are integrated across the depth of the section. The integration process requires a supplementary sub-routine to extract generated internal stresses to arrive at moment and shear capacities (at any time step). These internal moment and shear capacitates are compared against bending moment and shear force resulting from loading on beam. Failure is said to occur under flexural or shear limit state once bending moment (or shear force) due to applied loading exceed the moment (or shear) capacity at a critical section.

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In addition to flexure and shear, local buckling is also checked at each time step. Slenderness of flanges and web is checked against flexural and shear slenderness limit at different temperatures (time steps). Once the sectional slenderness exceeds that slenderness limit (λ_p or λ_r), the section is said to have transformed to a non-compact or slender section. This imposes instability-based reduction to sectional capacity through buckling of web/flanges. Finally, deflection limit state is also applied to evaluate failure utilizing provisions in BS 476 [19]. Accordingly, when the beam attains a deflection of $L/20$ or rate of deflection reaches a limiting rate of $L^2/9000d$; where L and d are the span and depth of the beam, respectively, the beam is said to attain failure.

5.0 MODEL VALIDATION

The above finite element model was validated by comparing predicted response parameters against measured data in fire tests. Since there is lack of published experimental data on steel beams subjected to high shear forces, data from tests on conventional steel beams was utilized [20]. Kodur and Fike reported data from a fire resistance test on a 4 m long W12×16 A992 steel beam under ASTM E119 standard fire and the layout of the tested beam is shown in Fig. 5. This beam was insulated with 50 mm thick spray applied vermiculite based fire insulation to achieve a 2-hr fire resistance rating. The beam was loaded with two point loads (close to mid-span) and the resulting moment and shear force represented 31% of its flexural capacity and 5% of its shear capacity at room temperature.

The tested beam is analyzed using the above developed model and various output parameters generated in the analysis, namely temperatures, mid-span deflection and failure mode are compared against measured data from tests. Figure 6 shows a comparison of predicted and measured temperatures in the steel beam as a function of fire exposure time. In general, temperature in steel section increases slowly due to the presence of fire insulation. The plotted temperature in the steel section is the average of top and bottom flange and web temperature. It can be seen that there is a good agreement between predicted and measured temperatures up to about first 45 minutes (till 350˚C) in steel. Then, the predicted temperatures tend to be slightly higher than the measured ones; such variation can be attributed to differences in assumed and actual thermal properties of fire insulation at elevated temperatures. However, both measured and predicted temperatures converge towards the end of fire exposure.

A comparison of predicted and measured mid-span deflection response of the tested steel beam is shown in Fig. 6b. Initially the beam undergoes only a slight deflection and this remains constant for about 90 min. This can be attributed to slower temperature rise of the steel beam facilitated by the presence of fire insulation. Steel does not experience significant degradation in strength in temperature range of 20-400˚C, but experiences moderate loss (of about 30% at 400˚C) of elastic modulus and hence deflection remain moderate in this temperature range. However, as the temperature in steel beam reaches about 550˚C, at about 100 min, strength and stiffness properties of steel start to degrade at a faster pace leading to rapid rise in deflection. Finally, at about 122 min into fire exposure, steel loses most of its strength and stiffness as the temperature of the beam rises to 600˚C. This leads to rapid rise in mid-span deflection and produces runaway failure in the beam at 122 min. Further details on validation of additional response parameters can be found elsewhere [1].

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6.0 NUMERICL STUDIES

The validated finite element model was applied to study the effects of instability on the response of fire exposed beams. Results from finite element analysis is examined to evaluate failure of beams under different failure limit states including flexure, shear, sectional instability and deflection criteria and to illustrate the effect of local instabilities on fire resistance of beams.

6.1 Selection of beams for analysis

To study the effect of flange and web slenderness on fire behavior of steel beams, three beams, namely “Beam 1”, “Beam 2” and “Beam 3” were analyzed. “Beam 1” is a W16×31 section (from AISC design manual [14] and made of Grade 345 MPa) and “Beam 2” and “Beam 3” are replicates of “Beam 1” but with thinner (slender) webs. All three beams were subjected to a combined uniformly distributed loading (UDL) and concentrated loading as illustrated in Fig. 7. “Beam 1” is subjected to UDL of 3 kN/m together with two concentrated loads of 258 kN applied close to the supports. Beams 2 and 3 are also subjected to the same level of UDL of that of “Beam 1” but the two concentrated loads applied near supports are 178 and 55 kN, respectively. Table 2 tabulates sectional slenderness together with limiting slenderness ratios for shape classification of sections based on flexural and shear limit states as per AISC classifications [14].

From flexural limit state consideration, all three beams have compact flanges as at room temperature as per AISC classification. In addition, Beams 1 and 2 have compact web and are expected to develop their full plastic moment capacity at room temperature. However, “Beam 3” has only compact flanges and non-compact web. Hence, “Beam 3” will have reduction in moment capacity resulting from compression flange yielding (CFY) that occurs when the non-

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compact web buckles. Local buckling of non-compact web adds additional stress to compression flange which will further reduce moment capacity. From shear limit consideration, Beams 1, 2 and 3 are compact, non-compact and slender web, respectively.

6.2 Analysis details

The beam selected for analysis is a simply supported beam of W16×31 section (taken from AISC design manual [14]) and the layout of the beam is shown in Fig. 7. This section is made of Grade 345 (MPa) steel and is assumed to have continuous lateral support along its 9.14 m span length. In order to study the effect of local buckling of flanges and web on structural response, the beam is subjected to combined bending and shear loading and exposed to ASTM E119 fire exposure. The loading was applied as a uniformly distributed loading (UDL) across the whole span of the beam, and two concentrated loads close to supports. Further details on applied loading is provided in subsequent section.

In order to simulate the response of fire exposed steel girders, two stages of analysis are to be carried out at each time step. The first stage simulates heat transfer between fire source and beam-slab assembly. In this stage, temperature profiles and thermal gradients are generated based on fire exposure. Once sectional temperatures are generated from thermal analysis, they are input to the second stage of fire analysis. Both temperature and gravity loading are applied simultaneously to the second stage of fire analysis to carry out structural analysis. In the structural analysis, mechanical behavior i.e., mid-span deflection, stress, strain and stability state along with sectional capacity of fire exposed beam is evaluated.

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6.3 Effect of slenderness on response parameters

6.3.1 Temperature progression

For heat transfer analysis, temperature progression in Beams 1, 2 and 3 are plotted as a function of fire exposure time in Fig. 8. It can be seen that all three beams experience similar temperature rise due to same fire exposure and material properties. As expected, temperature in the slender (thinner) web of "Beam 2" and "Beam 3"rise at a faster rate than that of web in "Beam 1". Thus, faster degradation of strength and stiffness properties occurs in the web of "Beam 2" and "Beam 3" as compared to that in web of "Beam 1". This variation in temperature rise in these beams will significantly affect temperature induced sectional instability.

In order to study the effect of sectional slenderness on response of beams under fire conditions, the variation of flange and web slenderness limits with temperature under flexural and shear limit states is evaluated by applying temperature induced reduction in yield strength and elastic modulus to room temperature equations. Figure 9 shows temperature-dependent variation in flange and web slenderness limits, under flexure and shear limiting states, along with flange and web slenderness ratio of Beams 1, 2 and 3. As can be seen in Fig. 9a that these beams maintain compact flange status throughout fire exposure duration.

Figure 9b shows web slenderness of Beams 1, 2 and 3 plotted against temperature-dependent variation of web slenderness limits under flexural limiting state. Web slenderness of "Beam 1" remains within compactness limit at elevated temperatures. Web of "Beam 2" (compact at room temperature) becomes non-compact in the temperature range of 600-730˚C, and then transforms back to compact web beyond temperature of 730˚C. Finally, web slenderness of "Beam 3" remains non-compact throughout the temperature range of 20-800˚C.

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Figure 9c shows temperature-dependent variation of different web slenderness limits and web slenderness ratios used for shear limit state. It can be seen that Beams 1, 2 and 3 have compact, non-compact and slender webs at room temperature. Web slenderness of "Beam 1" exceeds that of non-compactness limit at 150˚C, thus transforming into a non-compact web. At temperatures in the range of 620-710˚C, the non-compact web of "Beam 1" transforms into a slender web. Then, at temperatures of 710-780˚C and beyond 800˚C, this web transforms a non-compact and compact web, respectively (refer to Fig. 9c). "Beam 2" has a non-compact web at room temperature which changes to a slender web once its temperature exceeds 180˚C. However, this web transforms back to a non-compact web beyond 760˚C. "Beam 3" has a slender web and remains slender throughout the temperature range.

6.3.2 Flexural and shear capacity degradation

Figure 10 shows degradation of moment and shear capacity with fire exposure time at critical sections (mid-span and near end supports) of Beams 1, 2 and 3. Moment capacity is governed by yield strength of steel and plastic section modulus (Z). Although all three beams have similar flange width and thickness as well as height dimensions, the moment capacity in Beams 2 and 3 is slightly lower (by 5.6 and 14.8%) than that in "Beam 1" due to reduced web thickness and thus lower plastic modulus (see Fig.10a). Once exposed to fire, temperature in steel section increase and the moment capacity in Beams 1, 2 and 3 starts to degrade after about 9, 7 and 5 min, respectively, as can be seen in Fig. 10a. The rate of degradation in moment capacity increases after steel temperatures rise beyond 400˚C.

Figure 10b illustrates degradation of shear capacity in Beams 1, 2 and 3 with fire exposure time. Since shear capacity is a function of yield strength and web area, shear capacity

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in Beams 2 and 3 is significantly lower (12.8 and 45.5%) than that in "Beam 1" due to reduced web thickness. Once exposed to fire, shear capacity degrades with rise in steel temperature. Degradation of shear capacity is also influenced by the onset of local buckling (instability) in web as can be seen in Fig. 10b where variation in shear capacity is plotted under two scenarios of accounting for local buckling of web (WLB) and not considering local buckling (no WLB). The rate of degradation in shear capacity is different in each of these beams due to variation in temperature rise resulting from different web thicknesses. Figure 11 illustrates typical failure of Beams 1 and 2 as a result of web local buckling.

To further understand the fire response of Beams 1, 2 and 3, time history of reserve moment and shear capacity of these beams is also studied. At the beginning of fire exposure, Beams 1, 2 and 3 have reserve moment and shear capacity of 2.6 and 2.26 of that of the applied loading, as shown in Fig. 12. It is clear that room temperature reserve shear capacity is lesser than reserve moment capacity, thus these beams will fail in a shear failure mode at critical sections with high shear point loading. At the start of fire, reserve sectional capacity (moment and shear) starts to decrease due to temperature-induced strength and instability losses until failure occurs in the beams. In Beams 1, 2 and 3, reserve shear capacity always exceeds shear force due to applied loading ($\frac{reserve\ capacity}{applied\ loading} = 1$) prior to exceeding reserve moment capacity. This agrees well with presented results in Fig. 10.

6.3.3 Temperature-induced local buckling

In order to quantify and isolate the effect of temperature-induced web local buckling on shear capacity, further analysis has been carried out on Beams 1 and 2. Figure 13 shows the extent of degradation of shear capacity from the effect of temperature-induced strength

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degradation and local buckling effects, respectively. In "Beam 1" effect of web local buckling on the shear capacity is apparent from initial stage of fire exposure. This is due to the fact that web slenderness of "Beam 1" (λ_{web} = 57.8) is close to limiting compactness slenderness (λ_p = 59.24) at the start of fire exposure. Once steel temperature rises above 150˚C, at 4 min into fire, elastic modulus of steel starts to decrease. Hence, λ_p reduces until it decreases below λ_{web}, and temperature-induced web local buckling occurs. When steel temperature reaches 400-550˚C range, at 8 to 10 min, temperature-induced strength degradation occurs in steel. It can be seen that the 22% reduction in shear capacity, when steel temperature is in the range of 150-550˚C (corresponding time in 4-10 min) is only due to temperature-induced instability (see Fig. 13a). At temperatures above 550˚C, most of losses in shear capacity of "Beam 1" is due to temperature-induced strength degradation and effect of local instability tend to slightly decrease. Effect of local instability decreases following temperature-based variation in web slenderness limits discussed in Fig. 9c. Finally, this beam fails at 11.6 min when the web temperature is 560˚C. At this point, reduction in shear capacity of "Beam 1" due to temperature induced strength and instability effects is 40% and 12%, respectively.

In "Beam 2", effect of strength and local instability on shear capacity is quite different than that of "Beam 1" (see Fig. 13b). This is because "Beam 2" is a non-compact section with web slenderness (λ_{web} = 67.0) lying fairly in between limiting compactness slenderness (λ_p = 59.24) and limiting non-compactness slenderness (λ_r = 73.78). Although elastic modulus of steel starts to degrade at 150˚C, effect of web local buckling is not apparent until web temperature reaches 300˚C (at 5.5 min). At this temperature, web slenderness (λ_{web}) exceeds non-compactness limit and temperature-induced instability occurs. Shortly after that, steel temperate

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reaches 400˚C (at 7 min) and additional loss in shear capacity is observed due to degradation in strength properties of steel. Finally, "Beam 2" fails at 10 min when web temperature is 540˚C. At this point, reduction in shear capacity of "Beam 2" due to temperature induced strength and instability effects is 38% and 5%, respectively.

It can be seen that the effect of local buckling is very significant when a compact section transforms into a non-compact section (can cause up to 22% reduction in shear capacity). In order to minimize instability in compact sections under fire conditions, web slenderness in these sections can be limited to a certain degree. This limit can be quantified as the lowest compactness limit that can occur under fire conditions (λ_{lc}). As shown in Fig. 9c, this value equals 44 and usually occurs at about 700˚C for steel Grade 345 MPa. For example, a compact W-shape steel beam with web slenderness of 44 and lower does not experience fire-induced instability since web slenderness cannot exceed that of compactness limit under fire conditions, and as a result the web remains compact.

However, since most available W-shaped steel beams are of web slenderness of 50 and more, webs of steel beam are more likely to experience some level of local buckling once exposed to fire. Such beams are usually compact at room temperatures, but can transform to non-compact/slender sections under fire conditions. Following the above discussion and to limit adverse effect of instability of fire exposed compact sections, a more practical limit is proposed. This proposed limit equals the lowest non-compactness limit that can occur under fire conditions (λ_{lc}). The lowest non-compactness limit of Grade 345 MPa steel occurs around 700˚C and equals 55. Thus, using steel beams with web slenderness not exceeding 55 ensure that these beams

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remain non-compact under fire conditions. This minimizes reduction in shear capacity arising from web local buckling (from transforming into slender sections). Since λ_{lc} and λ_{ln} are a function of strength and stiffness properties of steel, this limit is a constant value and can be used for any loading/fire scenario/duration. Similar values of λ_{lc} and λ_{ln} for other steel grades can be arrived at using similar charts as that of Fig. 9c.

6.3.4 Mid-span deflections

The variation of mid-span deflection for Beams 1, 2 and 3 as a function of fire exposure time is plotted in Fig. 14. Mid-span deflection in these beams are compared against deflection limits recommended in British Standard BS-476 [19]. In all three beams, mid-span deflections are quite small at the initial stage of fire, then increases steadily with fire exposure time. At later stages (up to failure of beams), deflections rapidly increase due to rise in steel temperature. In general, Beams (2 and 3), with larger web slenderness than "Beam 1", experience larger deflections throughout fire exposure duration. Thus, web slenderness also has some level of influence on deflection in beams under fire conditions.

6.4 Failure modes

Table 3 summarizes failure modes in Beams 1, 2 and 3 analyzed with different web slendernesses. When local buckling in web is neglected, Beams 1, 2 and 3 fail when temperature of web is 610, 583 and 340˚C, which corresponds to 13, 11 and 7 min, respectively. However, when local buckling effect of web is accounted for, Beams 1, 2 and 3 fail through shear limit state at web temperature of 560, 540 and 420˚C which corresponds to 11.6, 10 and 7 min, respectively. This failure occurs when web temperature is relatively higher than flanges temperature as shown in Table 3. These results infer that the failure in these beams result from

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local buckling in web rather than flexural effects. In addition, these results show that beams fail at later times (and corresponding higher temperatures) when shear and local buckling effects are neglected. Thus, current philosophy of evaluating time to failure based on flexural effects only may not be fully conservative in cases of slender webs and when beams are subjected to significant shear loading. It should be noted that there is only slight difference in failure times of these uninsulated steel beams under different limit states. However, effect of different failure modes and corresponding failure times can be much more apparent in fire insulated beams [1].

7.0 CONCLUSIONS

Based on the results of the analysis presented herein, the following conclusion can be drawn:

1. The proposed finite element model accounts for effects of local buckling in tracing the response of steel beams exposed to fire conditions.

2. The degradation in flexural and shear capacity in fire exposed steel beam result from the effects of temperature-induced strength loss and temperature-induced sectional instability.

3. Onset of local buckling in web can lead to additional degradation of shear capacity in a beam and lead to its failure under shear limiting capacity.

4. Limiting web slenderness of a compact steel beam to (λ_{lc} = 44) prevent occurrence of web local buckling under fire conditions (transforming into a non-compact section). Similarly, limiting web slenderness of compact and non-compact steel beams to (λ_{ln} = 55) prevent transformation of such beams into slender sections once exposed to fire conditions.

8.0 ACKNOWLEDGMENTS

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LIST OF TABLES:

Table 1 Width-to-thickness limiting ratios for flexural and shear strength evaluation of W-shaped sections

Element	λ	Flexural capacity		Shear capacity	
		λ_p	λ_r	λ_p^*	λ_r^{**}
Flange	$b_f/2t_f$	$0.38\sqrt{\frac{E}{f_y}}$	$1.0\sqrt{\frac{E}{f_y}}$	-	-
Web	h/t_w	$3.76\sqrt{\frac{E}{f_y}}$	$5.70\sqrt{\frac{E}{f_y}}$	$1.10\sqrt{\frac{k_v E}{f_y}}$	$1.37\sqrt{\frac{k_v E}{f_y}}$

b_f is flange width; t_f is the flange thickness; h is the depth of section and t_w is the web thickness.

**limit of inelastic web buckling, ** limit of elastic web buckling, kv is 5 for unstiffened webs with $\lambda \leq 260$*

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Table 2 Beams with varying slenderness and relevant slenderness limits for flexural and shear design

Beam	Flange slenderness (λ_f)	Web slenderness (λ_w)	Flexural limit state				Shear limit state	
			Flange slenderness limits		Web slenderness limits		Web slenderness limits	
			λ_p=0.38 $\sqrt{\frac{E}{f_y}}$	λ_r=1.0 $\sqrt{\frac{E}{f_L}}$	λ_p=3.76 $\sqrt{\frac{E}{f_y}}$	λ_r=5.70 $\sqrt{\frac{k_v E}{f_y}}$	λ_p=1.10 $\sqrt{\frac{k_v E}{f_y}}$	λ_r=1.37 $\sqrt{\frac{k_v E}{f_y}}$
Beam 1	6.28	57.82	9.15	22.8	90.6	137.3	59.24	73.78
Beam 2	6.28	67.0						
Beam 3	6.28	100.13						

$f_L = 0.7 f_y$ for plate beams with slender webs

Table 3 Failure of beams with different web slenderness

Beam	Web slenderness	Flange slenderness	Failure time (min)				Temp. at failure (˚C)*		Failure mode
			Shear (no WLB)	Shear (WLB)	Flexure	Deflection	Web	Flanges	
Beam 1	57.8	6.28	13	**11.6**	14	13.8	560	540	Shear
Beam 2	67.0	6.28	11	**10**	12.5	11.7	540	507	Shear
Beam 3	100.1	6.28	7	**7**	11.5	9.3	420	346	Shear

*Corresponds to shear failure due to web local buckling

LIST OF FIGURES:

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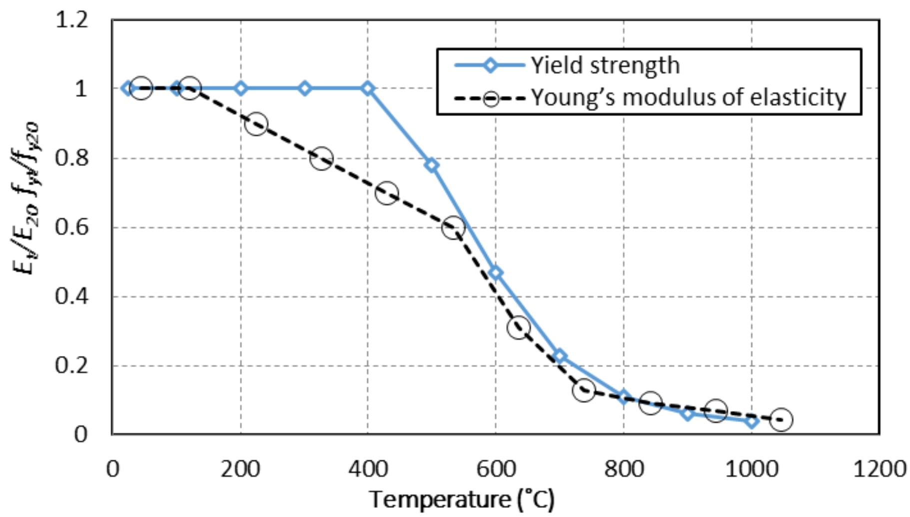
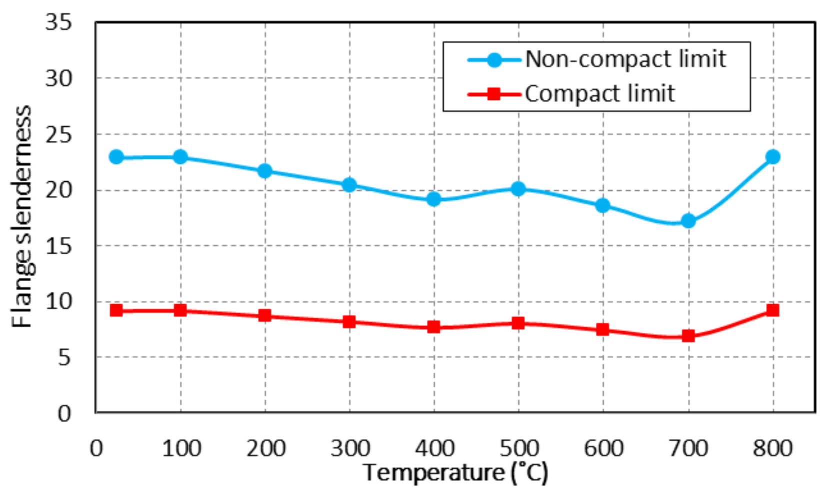


Fig. 1. Degradation of strength and stiffness properties of steel at elevated temperatures



(a) flange slenderness limits for flexural calculation

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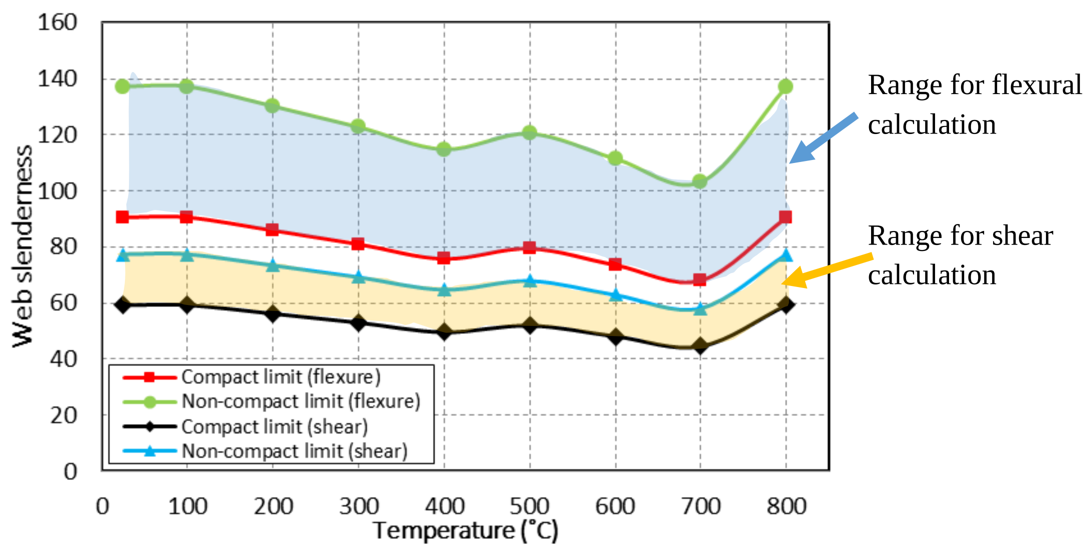

(b) web slenderness limits for flexural and shear calculations

Fig. 2. Variation of slenderness limits adopted in flexural and shear design at elevated temperatures

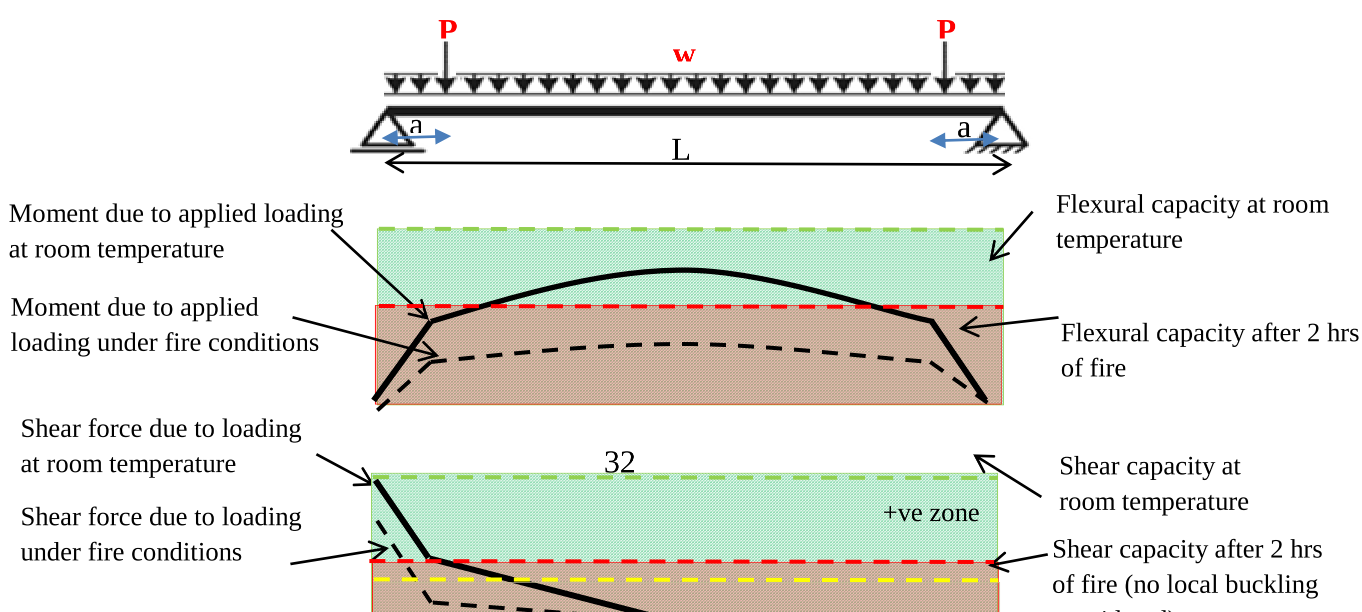

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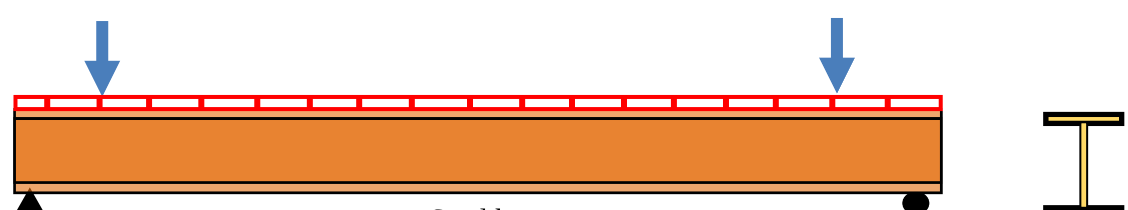
Fig. 3. Variation of bending moment and shear force in beams subjected to high shear forces

(Capacity at room temperature, Capacity under fire conditions (without local buckling), Capacity under fire conditions (with local buckling))

(Shear/moment loading at room temperature, Shear/moment loading under fire conditions)



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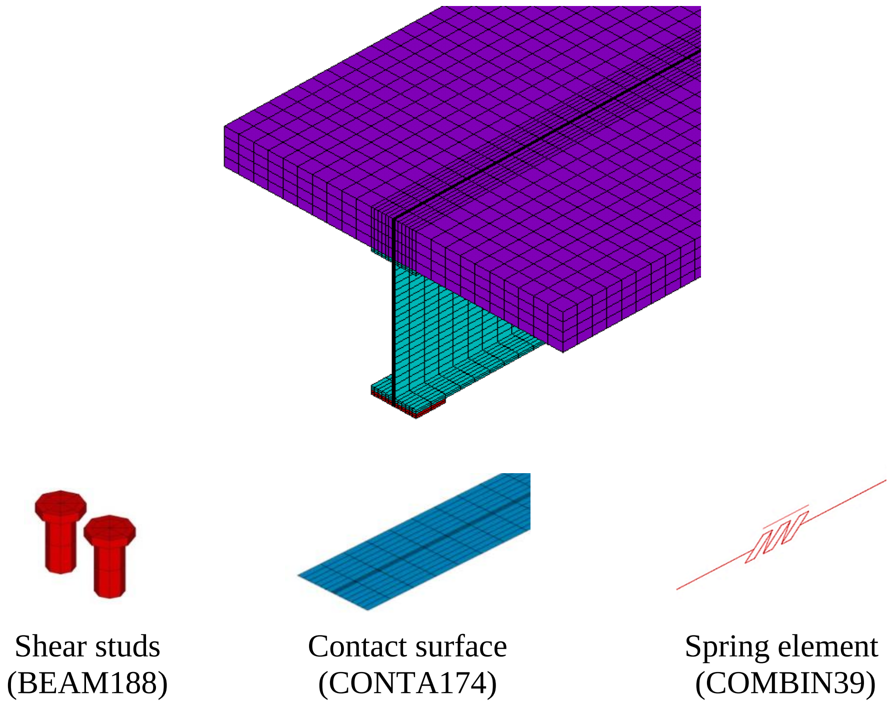


Fig. 4. Details of the developed finite element model for fire resistance analysis of beam-slab assembly

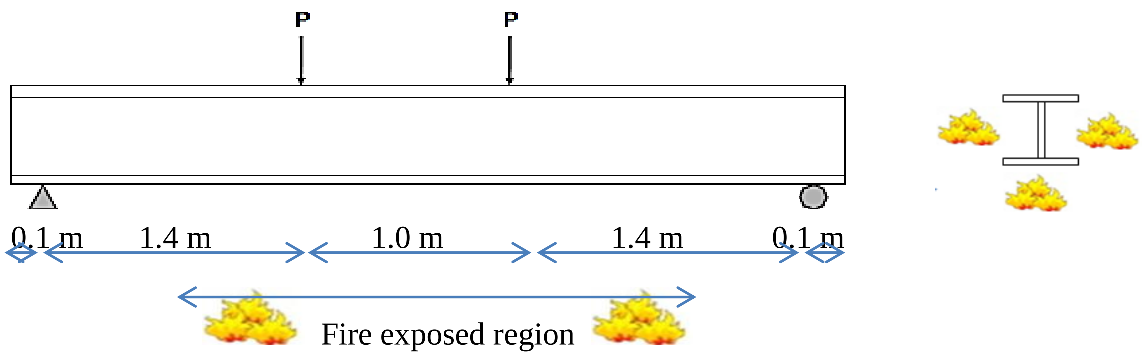
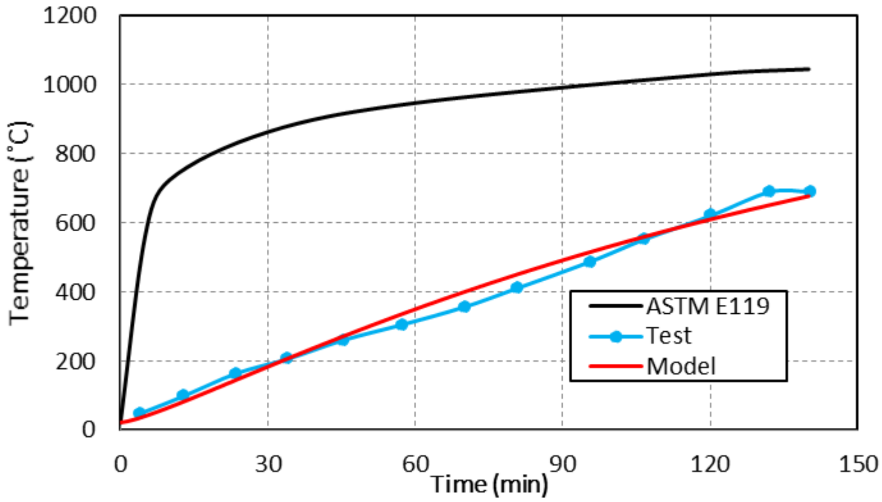
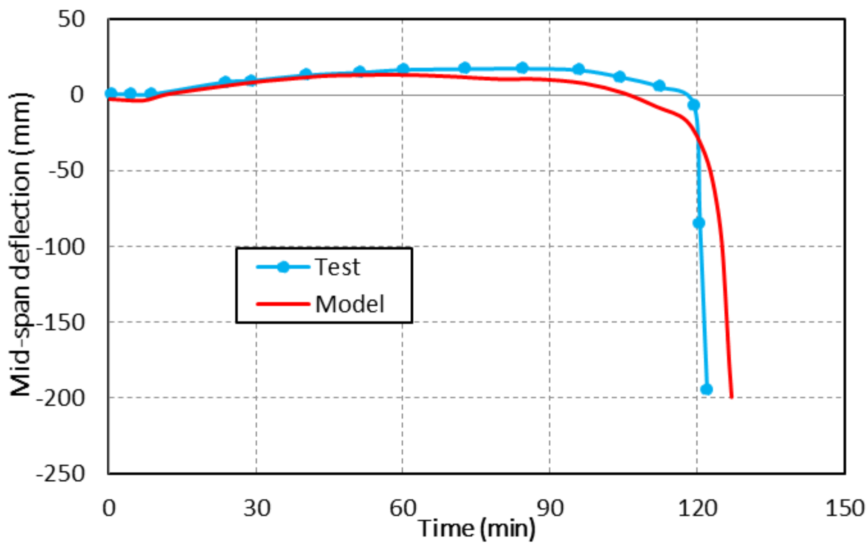


Fig. 5. Tested beam used in validating the developed finite element model



(a) Thermal response



(b) Structural response

Fig. 6. Comparison of predicted and measured temperature and deflections as a function of fire exposure time

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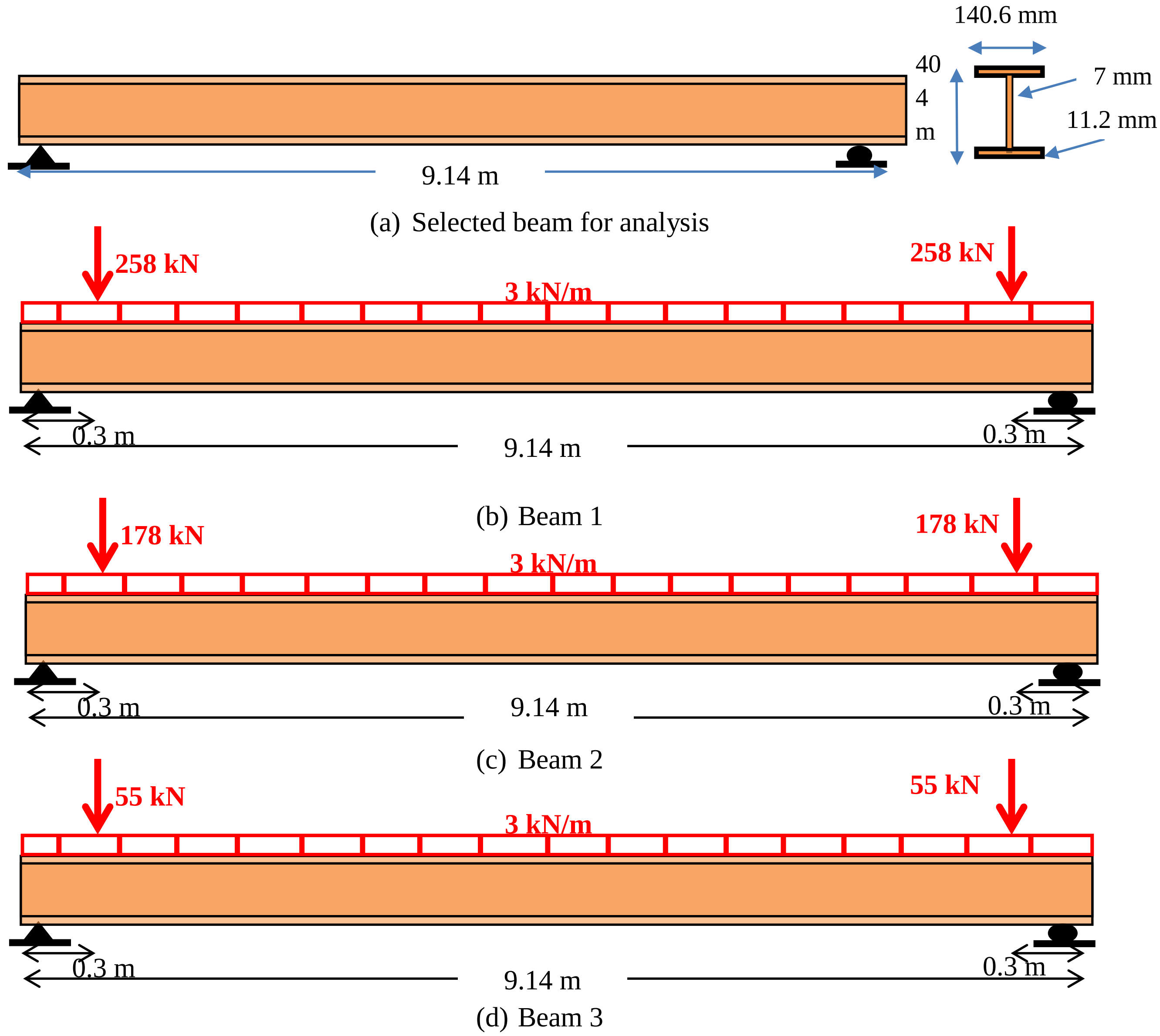

Fig. 7. Load configuration of Beams 1, 2 and 3

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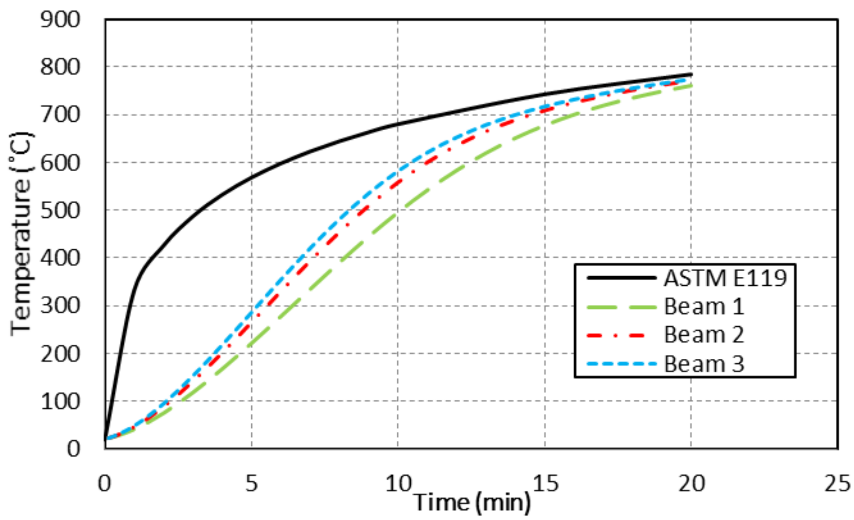
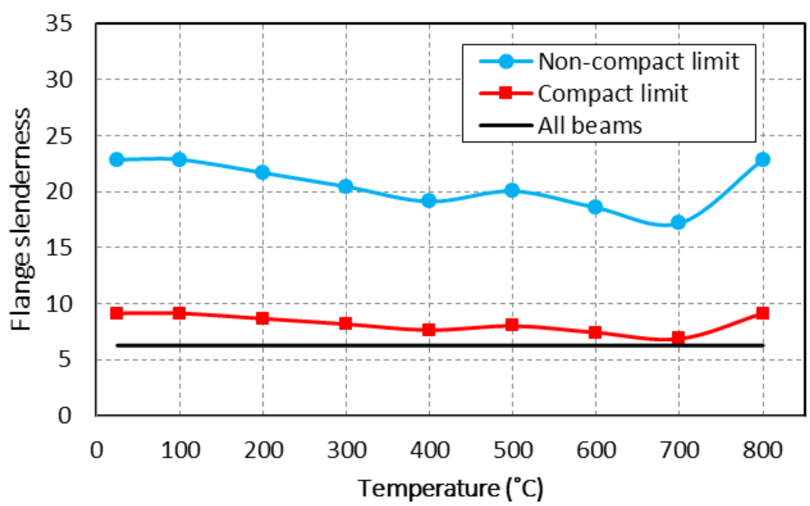


Fig. 8. Variation in web temperature of Beams 1, 2 and 3 with fire exposure time



(a) flange slenderness limits under flexural limit state

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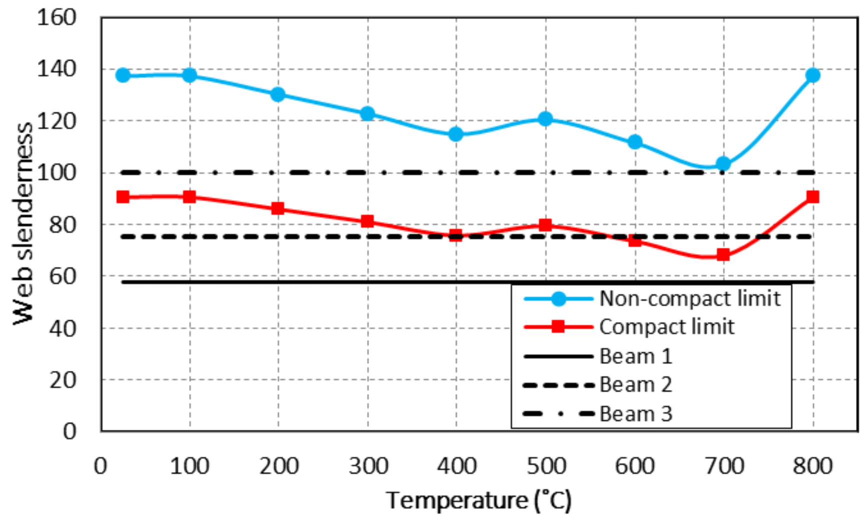
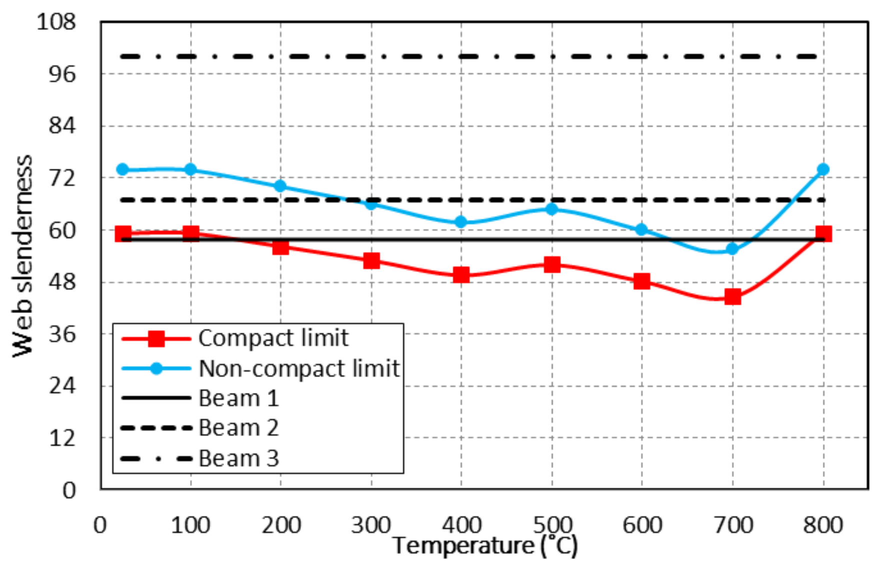


(b) web slenderness limits under flexural limit state



(c) web slenderness limits under shear limit state

Fig. 9. Variation of flange and web sectional slenderness limits under flexural and shear limit states at elevated temperatures

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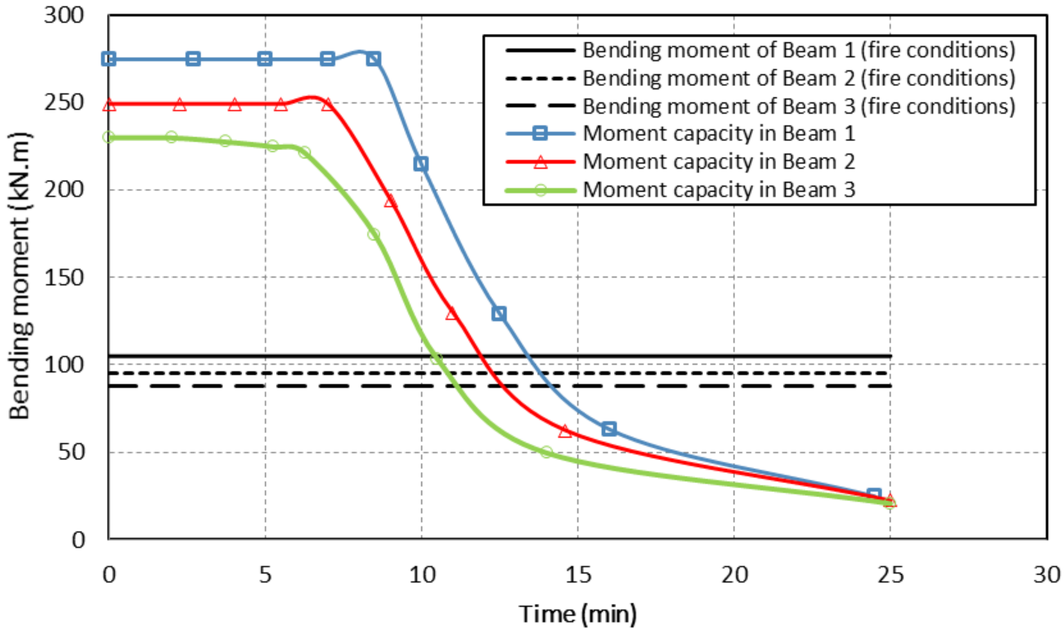
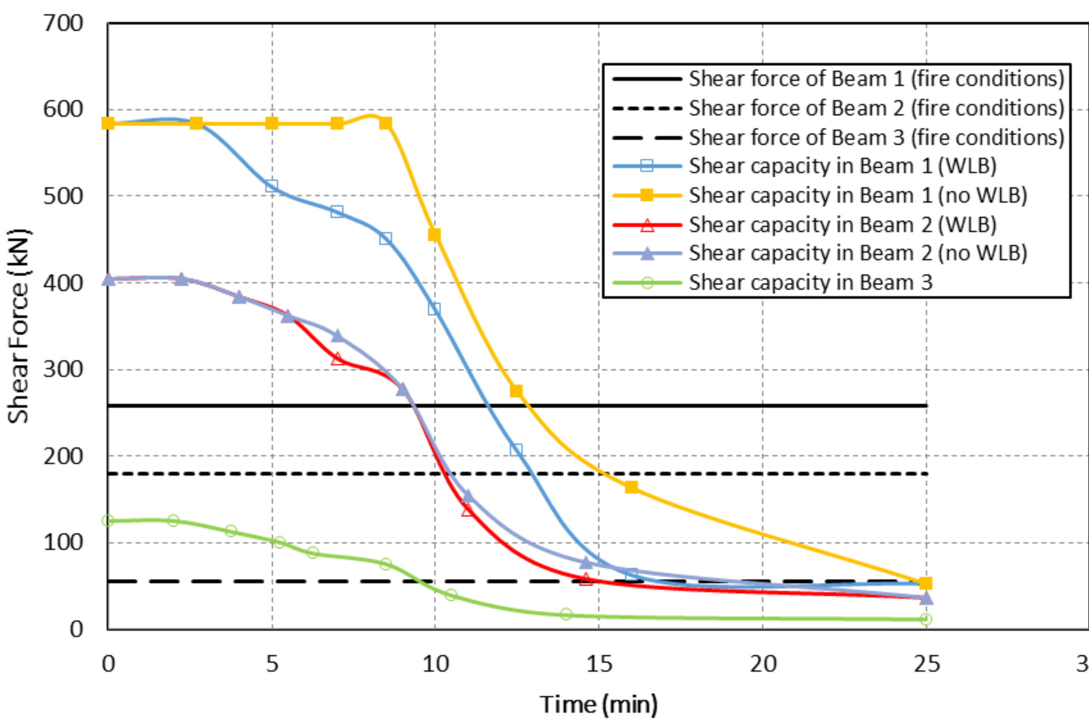


(a) Moment capacity (at mid-span)



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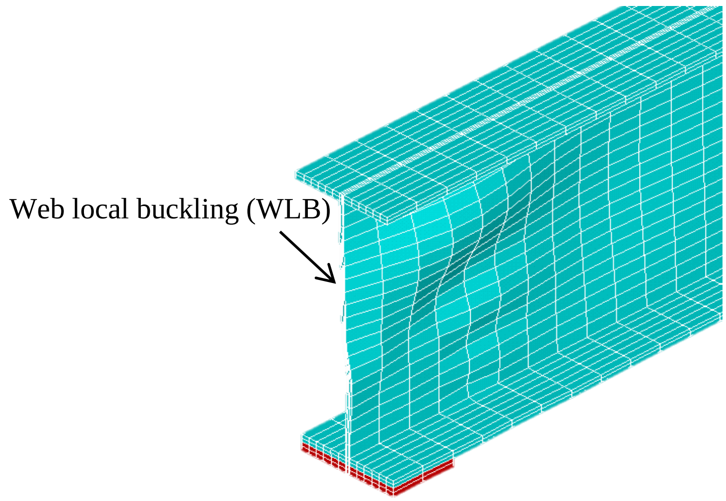
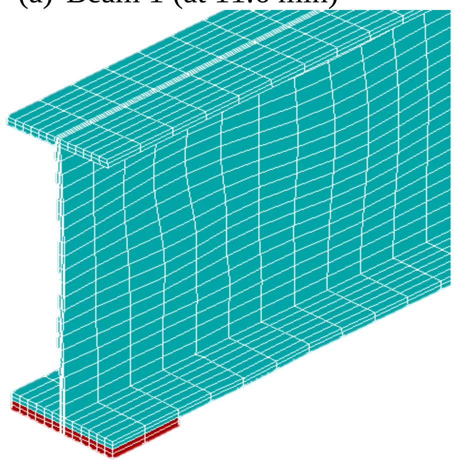
(b) Shear capacity (near end supports)

Fig. 10. Degradation of moment and shear capacity with fire exposure time in beams with varying slenderness ratio



(a) Beam 1 (at 11.6 min)



(b) Beam 2 (at 10 min)

Fig. 11. Web local buckling of Beams 1 and 2 at failure

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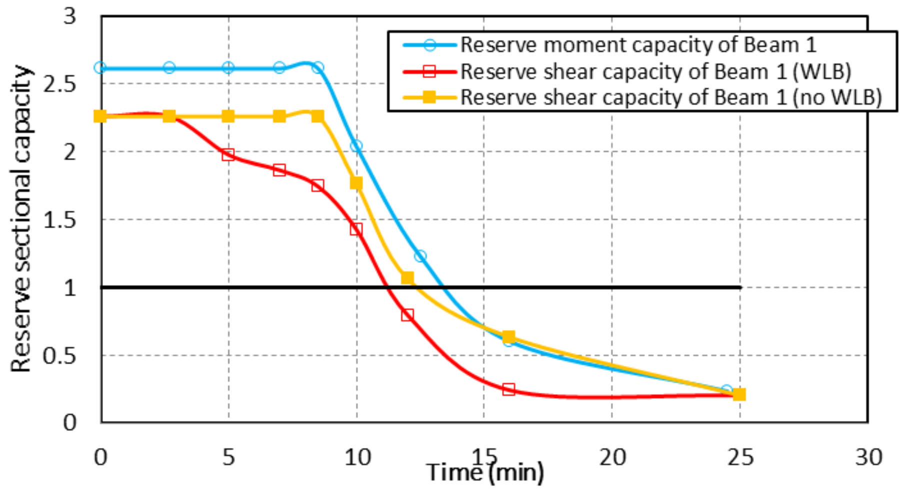
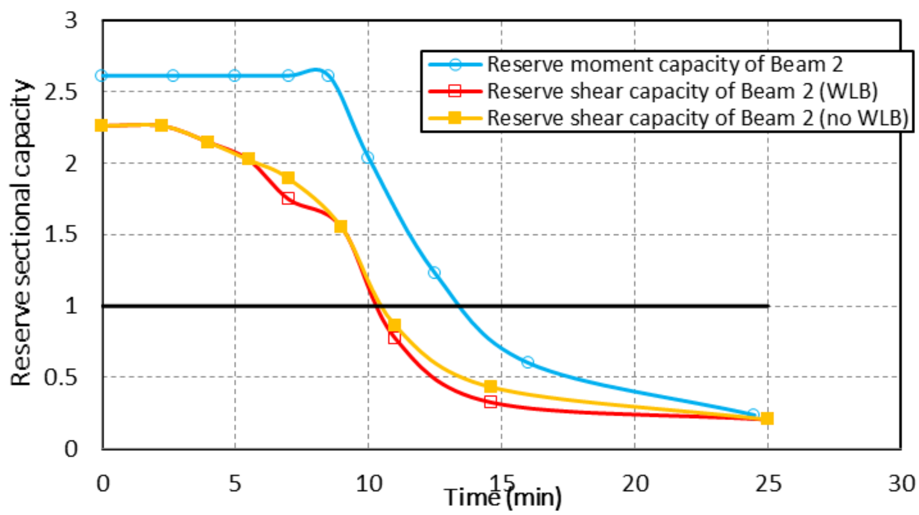


(a) Beam 1



(b) Beam 2

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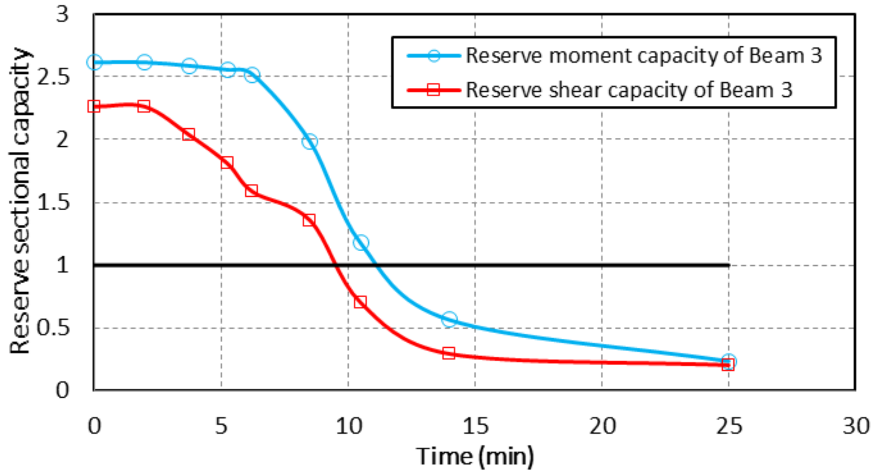
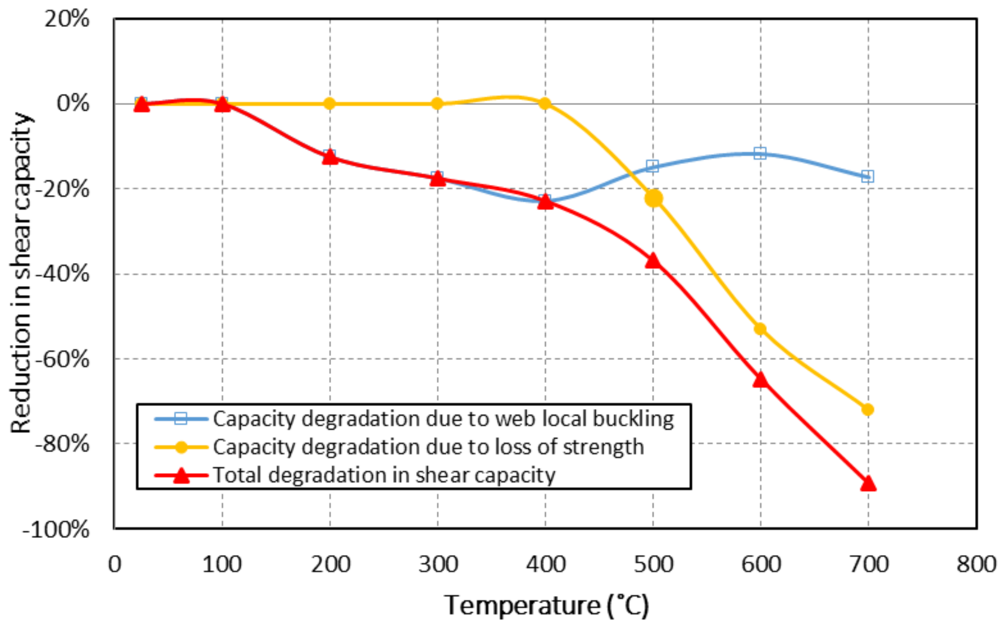
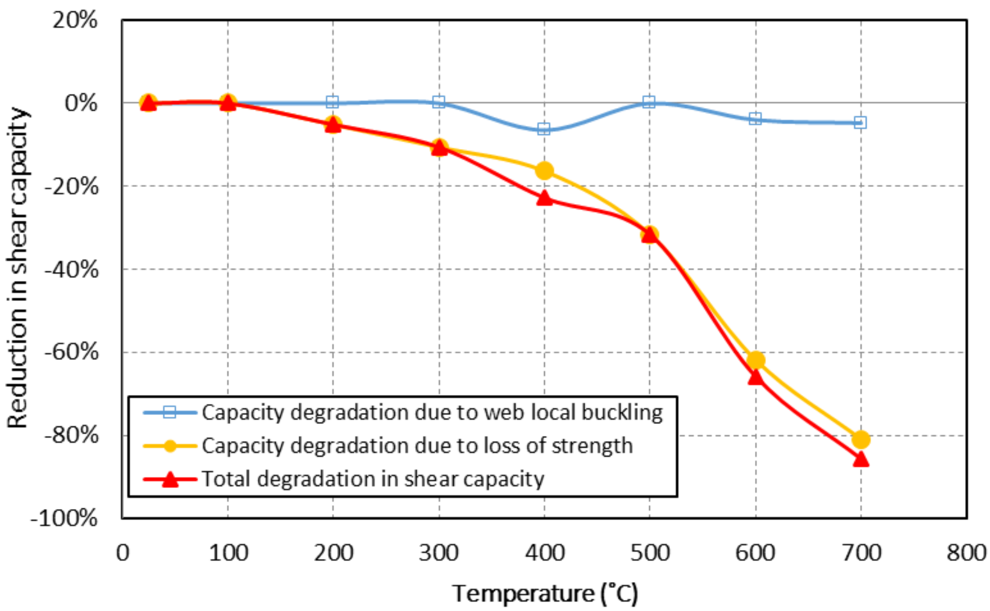


(c) Beam 3

Fig. 12. Degradation of moment and shear capacity in Beams 1, 2 and 3 with fire exposure time



(a) Beam 1



(b) Beam 2

Fig. 13. Effect of temperature-induced web local buckling on shear capacity of Beams 1 and 2

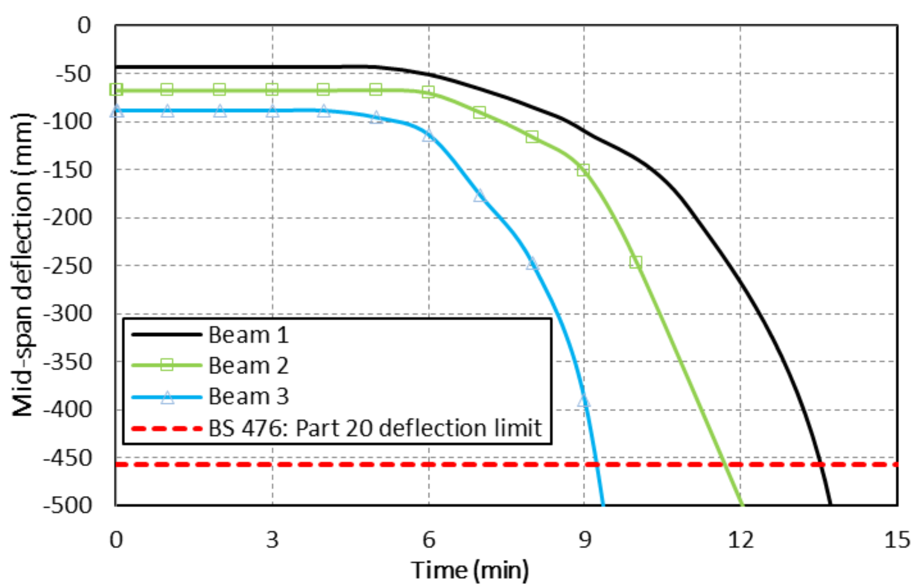


Fig. 14. Comparison between mid-span deflections of steel W-beams with different web slenderness ratios