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Factors Governing onset of Local Instabilities in Fire Exposed Steel Beams

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1.0 ABSTRACT

This paper presents critical factors that influence the onset of local buckling in steel beams when exposed to fire conditions. A three-dimensional nonlinear finite element model, capable of accounting for critical factors that influence local instability in fire exposed steel beams is developed. This model is applied to investigate the effect of beam-slab interaction, strength properties (Grade) of steel, and presence of fire insulation on the onset of local instability, and resulting capacity degradation in fire exposed steel beams. Results from numerical simulations are utilized to evaluate failure of beams under different limit states including flexure, shear, sectional instability and deflection. These results infer that web instability can occur at early stages of fire loading leading to faster degradation of shear capacity and pre-mature failure of steel beams before attaining flexural capacity. Also, results from the analysis indicate that the contribution of concrete slab to shear capacity can counterbalance the adverse effect of web local instability to a certain degree. Thus, neglecting the effect of web local instability and contribution of concrete slab to shear capacity can lead to unconservative design in certain scenarios, especially when beams are subjected to high shear loading and/or local instabilities.

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

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2.0 INTRODUCTION

In current practice, steel beams are to be designed to satisfy flexural, shear and serviceability limit states [1]. In ambient design, one of the key factors that need to be satisfied in achieving required moment and shear capacity is local buckling limitations. However, in current fire design provisions, only moment capacity at a given fire exposure time is utilized to evaluate failure of steel beams under fire loading without giving any due consideration to shear and sectional instability. Although deriving failure of beams based on flexural limit state is valid in most application and loading scenarios, this assumption may not be representative in certain situations where shear and instability (i.e., web local buckling) effects can be dominant in a fire exposed member [1].

Shear effects can be dominant under certain loading scenarios such as beams with concentrated loads acting at interior or end supports; as in the case of beams connecting offset columns in a building and transfer beams. In addition, temperature-induced buckling of web can be a governing factor in steel structural members with certain geometrical features i.e., beams with coped ends, deep beams and plate girders (with slender webs) [2-6]. Since webs are typically much more slender than flanges, larger surface area of web gets exposed to fire leading to rapid rise of temperature in webs [1]. The faster rise in web temperatures lead to rapid degradation in strength and stiffness properties. This can initiate local buckling in web at lower steel temperatures and accelerate failure of beams.

Temperature-induced sectional instability in fire exposed steel beams result from the built-up of internal compressive stresses due to applied loading, and also due to rapid degrading in

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strength and modulus properties of steel with temperatures. When these built-up stresses reach plastic limit, sectional instability is said to occur. Occurrence of such instability reduces effective area, which in turn decreases available flexural and shear capacity under fire conditions. In fire exposed beams subjected to high shear forces, a combination of temperature-induced strength degradation (in web) and earlier onset of local instability due to temperature-induced web local buckling can cause failure of beams in "Shear" mode before attaining flexural capacity.

The effect of local buckling on the response of fire exposed steel members was studied by few researchers [7-9]. For instance, Uy and Bradford [7] studied local buckling of cold-formed steel structural members at elevated temperatures using finite strip method. The authors reported that the degradation in properties of steel at elevated temperatures can influence local buckling in steel-concrete composite construction. Zhao and Kruppa [8] through fire tests on fire exposed steel composite beams reported that steel beams classified as compact sections (at room temperature) can undergo local buckling under high temperature exposure.

In a recent study, Kodur and Naser developed a three dimensional finite element model to study the shear response of fire exposed steel beams [1]. The authors investigated the effect of different loading patterns, web slenderness and presence of fire insulation on steel beams subjected to high shear loading and exposed to fire conditions. Based on results from numerical studies, Kodur and Naser reported that shear capacity can degrade at a much higher pace than flexural capacity, thus leading to pre-mature failure under shear limiting state. This shear failure is initiated due to local buckling in web which can occur at early stages of fire loading. The authors illustrated

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that this failure to occur under certain loading conditions, such as in beams loaded with high shear forces near end or interior supports.

The beneficial effect of composite action is generally accounted for in evaluating flexural capacity at room temperature. Yet, in current provisions (AISC) of composite beam-slab assemblies, shear capacity is evaluated based on contribution of web only without any consideration to contribution from concrete slab of composite beam-slab assemblies [10]. Similarly, Eurocode provisions state that resistance to vertical shear should be taken as the resistance of the structural steel section alone (web) "unless the value for a contribution from the reinforced concrete part of the beam has been established" [11]. Hence, any contribution of reinforced concrete (RC) slab through composite action is neglected in evaluating shear capacity.

Although current design provisions neglects the positive contribution of concrete slab, experimental evidence suggests otherwise [12-14]. For instance, Johnson and Willmington studied the shear capacity of composite beams in the negative moment regions and reported that the concrete slab contributes 20–40% of the total shear capacity [12]. Shear capacity of composite steel beams was also studied by Nie et al. [13] through tests on 16 composite beams and two individual steel beams at ambient conditions. The authors reported that steel–concrete composite beams designed with full shear transfer between steel beam and concrete slab can develop much higher shear capacity as compared to plain steel beams. Also, it was found that the contribution of the concrete slab can enhance shear resistance by 33–56%. The above studies clearly show that current code provisions underestimate the shear capacity of composite beams by neglecting the positive contribution of concrete slab.

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Unfortunately, effect of local instability, and composite action on fire response of steel beams were not considered in earlier studies. In order to bridge this knowledge gap, a numerical study is carried out using a three-dimensional nonlinear finite element model. The developed model can trace fire response of steel beams subjected to significant bending moment and shear loading. The model is applied to examine the effect of beam-slab interactions, strength properties (Grade) of steel and fire insulation on the onset of local instability in steel beams exposed to fire.

3.0 EFEFCT OF LOCAL BUCKLING ON FLEXURAL AND SHEAR CAPACITY

Adverse effect of local buckling on the response of steel beams is taken into account in evaluating flexural and shear capacity at room temperature [10, 11]. For instance, 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. This sectional slenderness ratio is usually compared against two upper limits; compact (λ_p) and non-compact (λ_r) to classify the shape of the section. These upper limits are a function of strength and stiffness properties ($\sqrt{\frac{E}{f_y}}$) of steel.

Under fire conditions, local buckling can occur once strength and stiffness properties start to degrade with rise in steel temperatures. Onset of local buckling can induce further degradation in flexural and shear capacity of beams under fire conditions. Strength and stiffness properties of steel starts to degrade at different rates after about 400 and 150˚C, respectively (see Fig. 1). Hence, local buckling can occur at earlier stages of fire, even before strength properties starts to degrade. Therefore, capacity degradation in steel beams can arise from temperature-induced web local buckling (at 150˚C) followed by degradation of strength properties (at 400˚C) [1].

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Figure 2 illustrates how width-to-thickness classification limits of Grade 50 steel (345 MPa) change with elevated temperatures. Slenderness limits generally decrease as a function of temperature. However, flange slenderness limit, used in flexural evaluation, tend to be stable and experience slight fluctuation at elevated temperature (see Fig. 2). It can be seen in the figure that web slenderness limits (for shear evaluation) vary over a smaller range ($59 \leq \lambda \leq 77$) than that for flange slenderness limits used in shear calculation ($90 \leq \lambda \leq 137$). Thus, fire exposed steel beams are more sensitive to local buckling in web than that due to local buckling in flange.

Sectional slenderness (width-to-thickness ratio (λ)) depends only on the geometrical features (dimensions) of a section. Thus, flange and web slenderness ratios (λ_{flange} and λ_{web}) of a given shape remain invariant even under fire exposure. Since width-to-thickness classification limits decrease at high temperature, the constant value of flange and web slenderness ratios (λ_{flange} and λ_{web}) can exceed that of the degraded width-to-thickness classification limits. Once flange and/or web slenderness ratios exceed corresponding classification limits, temperature-induced local buckling is said to occur. Therefore, classification of a fire exposed steel section can change from that at room temperature.

To illustrate this, a W16×31 beam section made of Grade 50 (345 MPa) steel with web slenderness (λ_{web}) of 57.8 is selected. When comparing this web slenderness with web slenderness limit (λ_{wp}) of Grade 50 (345 MPa) steel at room temperature ($\lambda_{wp} = 1.10\sqrt{\frac{k_v E}{f_y}} = 59.24$), this beam falls under "compact" section category. However, when comparing web slenderness of the same section against web slenderness limit at 500˚C ($\lambda_{wp\ at\ 500^{\circ}C} = 51.9$), web slenderness of a W16×31

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section clearly exceeds that of web slenderness limit (λ_{wp}) at 500˚C. Thus, the beam is classified as a non-compact section when steel temperatures reach 500˚C. On the other hand, flange slenderness of the same section (λ_{flange}) is 6.28. This slenderness ratio remains below flange slenderness limit (λ_{fp} = $0.38\sqrt{\frac{E}{f_y}} = 9.15$) even at temperatures of 500˚C and beyond ($\lambda_{p\ at\ 800^{\circ}C}$ = 8.03). Therefore, flanges of a W16×31 beam section remain compact even at elevated temperatures.

Current design provisions in codes and standards provides no recommendations for classification of steel sections under fire conditions, based on local buckling criterion. This implies that if a section is compact at ambient conditions, this section continues to remain compact during fire conditions. From the analysis shown above, it is clear that current provisions may not lead to same shape of steel beams under fire conditions.

4.0 FINITE ELEMENT MODEL

To study the effect of local buckling on the response of steel beams under fire conditions, a three dimensional nonlinear finite element model was developed in ANSYS 14.0 [15] to trace the realistic response of fire exposed steel beams. This model accounts for critical parameters that influence sectional instability including geometric and material nonlinearities, composite action arising from the concrete slab, temperature dependent material properties and various failure limit states.

4.1 Discretization of beam

The three dimensional finite element model, is capable of tracing thermal and structural response of fire exposed steel beams from pre-loading stage till failure of the beam. In order to simulate the realistic response of fire exposed steel beams, the developed model was discretized using various element types available in ANSYS 14.0 [15]. These element types can simulate both thermal and structural behavior associated with fire exposed steel beams.

SHELL131, SOLID70, LINK33 and SURF152 elements are used as thermal elements to simulate heat transfer between steel girder and fire source [15]. 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 is a uniaxial (bar) element with the ability to conduct heat between its nodes and a single degree of freedom, temperature. 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 effects from fire zone to steel beam. In order to simulated convection and radiation mechanism, a convection coefficient of α_c= 25 W/(m$^{2\circ}$C) and Stefan-Boltzmann radiation constant of 5.67x10^{-8} W/(m$^{2\circ}$C) was applied in the thermal analysis.

For discretizing steel beam to undertake structural analysis, SHELL181, SOLID65, LINK8, COMBIN14, COMBIN39, BEAM188, CONTA173 and TARGE170 are utilized. SHELL181, used to model steel beam, has four nodes with six degrees of freedom (three translations and three rotations) per node. This element can capture local buckling of flanges and web and also lateral torsional buckling of steel beam and therefore is well-suited for large rotation, large strain and nonlinear problems. SOLID65 is used for three-dimensional modeling of solids with or without reinforcing bars (such as concrete slab). SOLID65 is capable of accounting for concrete cracking in tension and crushing in compression.

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LINK8, used to model steel reinforcement in concrete slab, is a two node uniaxial tension-compression spar element with three degrees of freedom (translations) at each node. This element is used to model the internal (steel) reinforcement embedded in the concrete slab. COMBIN14 and COMBIN39, are spring like element and are used to simulate bond between steel reinforcement and surrounded concrete and shear studs and concrete slab, respectively. BEAM188, used to model shear 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 [15]. The shear studs (BEAM188 elements) are embedded in the concrete slab and are assumed to be connected to the surrounding concrete. Thus, the nodes of BEAM188 elements are coupled with the nodes of concrete elements (SOLID65). These elements are also fully 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).

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. The element types used in structural analysis as well as a discretized view of the developed model is shown in Fig. 3.

4.2 High temperature material and constitutive laws

For undertaking fire resistance analysis, temperature-dependent thermal and mechanical properties of steel, concrete and fire insulation are to be input to the finite element model. The

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thermal and mechanical properties of structural steel, concrete and reinforcing steel are assumed to vary with temperature as per Eurocode 3 and 2 relations [16, 17]. For fire insulation, room temperature thermal properties are used in fire resistance analysis since there is very limited information on variation of thermal properties with temperature. The room temperature thermal conductivity and specific heat of fire insulation is 0.0815 W/m.˚C and 1047 J/kg.K, respectively.

For steel, a multi-linear stress-strain relationship, with kinematic hardening plasticity model as obtained using Eurocode 3 model is used. This constitutive material model consist of multiple stress-strain curves that vary temperature. In order to define the plastic behavior of the concrete, ANSYS uses a constitutive material model formulated by William and Wranke [18]. Concrete is assumed to follow a nonlinear parabolic behavior in compression, as described in Eurocode 2, and is treated as an isotropic elastic material until it cracks. Once a concrete element cracks, the tensile softening behavior is modeled using a tri-linear response. The tensile response starts with an ascending linear-elastic regime until it reaches the ultimate tensile strength (f_t). Once the ultimate tensile strength is reached, concrete tensile strength drop to ($0.6f_t$) then softens in a descending manner gradually to zero. Further, the concrete model requires additional parameters; open and close crack shear transfer coefficients, (β_t and β_c). These shear transfer coefficients are generally used to account for shear stiffness retention in cracked concrete elements and range from zero to one; to represent smooth and rough cracks, respectively. The values of β_t and β_c used in the developed model are assumed to be 0.2 and 0.7, respectively.

In order to accurately model the interface between reinforcing steel rebars and surrounding concrete, the longitudinal bond-slip between reinforcing steel rebars and surrounding concrete is

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modeled using COMBIN14 elements. COMBIN14 elements require a value for the longitudinal stiffness (k) which is obtained from the secant of Eq. (2) as derived by Nie et al. [19];

$$k = \frac{\pi}{s_u} p d_r N_r \tau_u \left(\frac{L_1 + L_2}{2} \right) \tag{2}$$

where,

p is the horizontal distance between the tension steel reinforcement bars in (mm), d_r is the diameter of the steel bars in (mm), N_r is the number of reinforcing bars and L_1 and L_2 represent the lengths of two adjacent reinforcement elements (LINK8) in (mm).

To account for shear force-slip between shear studs and concrete slab, a nonlinear constitutive relationship suggested by Ollgaard et al. [20] is used (shown in Eq. 3). The outcome of this relationship is used to supply the required shear force and slip behavior of shear studs modeled using COMBIN39 elements.

$$Q = Q_u \left[1 - e^{-4.75S} \right] \tag{3}$$

where Q is the shear force, Q_u is the strength of the studs calculated as $Q_u = 0.43 A_s \sqrt{E_c f_c} \le 0.7 A_s f_u$; A_s, E_c, f_c and f_u are cross sectional area of the shear studs, elastic modulus of the concrete taken as $4600\sqrt{f_c}$ (in MPa), compressive strength of the concrete, and ultimate strength of the studs taken as 420 MPa, respectively. S is slip length and the maximum slip length was set to 1.27 mm [21].

4.3 Failure limit states

The failure of a beam under fire conditions can occur in different modes and for evaluating realistic failure, different end failure criteria are to be applied. In this analysis, all possible different failure limit states including flexural, shear, local buckling and deflection are considered for evaluating failure of the beam at each time step and the failure is said to be reached once any of these failure limit states is exceeded. For example, moment and shear capacity at any time step is evaluated utilizing internal bending and shear stresses generated from ANSYS analysis. These stresses, generated at individual elements, were 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 each time step. These internal moment and shear capacitates are compared against flexural and shear limiting criteria. 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.

In addition, local buckling limit state is also checked at each time step by updating slenderness limits which changes with steel temperatures. In addition, slenderness of flanges and web is updated and checked against flexural and shear slenderness limits at different steel temperatures (time steps). Once the sectional slenderness exceeds that of degraded slenderness limit (λ_p or λ_r), local buckling is said to occur and sectional capacity is adjusted to account for losses arising from local buckling. Finally, deflection limit state is also applied to evaluate failure of fire exposed beams. When the beam attains a deflection of ($L/20$) or rate of deflection reaches ($L^2/9000d$); where L and d are the span and depth of the beam, respectively, the beam is said to attain failure [22].

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5.0 MODEL VALIDATION

Since there is lack of published experimental data on steel beams subjected to high shear forces, the above finite element model was validated using data from tests on conventional steel beams. Kodur and Fike [23] have conducted a fire resistance test on a 4 m long W12×16 A992 steel beam under ASTM E119 standard fire (see Fig. 4). The beam was insulated with 50 mm thick fire insulation to achieve a 2-hr fire resistance rating. The beam was loaded up to 31% of its flexural capacity using two point loads near mid-span.

The tested beam is analyzed using the above developed model and temperatures, mid-span deflection, moment and shear capacity and failure mode generated in the analysis are compared against measured test data. For instance, Fig. 5 shows a comparison of predicted and measured temperatures in the steel beam as a function of fire exposure time. It can be seen that the average of both flanges and web temperatures in steel section rises slowly due to the presence of fire insulation. These temperatures continue to rise until failure of beam. Predicted temperatures plotted in Fig. 5 shows good agreement with measured temperatures up to the first 45 minutes of fire test (when steel reaches 350˚C). Then, predicted temperatures tend to be slightly higher than the measured data points until 100 minutes into fire test. It should be noted that such variation can be related to differences in assumed and actual thermal properties of fire insulation at elevated temperatures. Towards the end of fire test, both measured and predicted temperatures converge at temperatures around 690˚C.

A comparison of predicted and measured mid-span deflection response of the tested steel beam is shown in Fig. 5b. In the first 90 min, the beam experiences slight level of deflection

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attributed to low rise of temperature in the insulated steel beam. However, as the temperature in steel beam reaches 550˚C (at about 100 min) strength and stiffness properties of steel experience faster degradation which lead to rapid rise in deflection. After two hours of fire exposure, steel loses most of its strength and stiffness as the temperature of the beam rises to 600˚C. This significant loss of strength and stiffness properties leads to rapid rise in deflections and produces runaway failure of the beam at 122 min. Further details on this validation process can be found elsewhere [1].

For this beam, degradation of moment and shear capacity with fire exposure time at corresponding critical sections; mid-span section for moment and support section for shear is shown in Fig. 6. The moment capacity in the beam remains intact for the first 75 minutes due to lower temperatures (much below than 350˚C) in flanges of steel beam. However, shear capacity starts to degrade at 35 min due to relatively faster rise in web temperature. Then, moment and shear capacity starts to degrade when the temperature in steel section reaches 400˚C. Degradation of moment capacity of steel section continues till 130 min at which the beam fails. This beam fails because its moment capacity at mid-span falls below the moment due to applied loading. Since the beam was subjected predominantly to flexural loading, the resulting applied shear force does not fall below shear capacity near the vicinity of support section. Hence as per the analysis, failure of this beam occur due to flexural effects at 130 min while correspondingly failure of this beam in fire test occurred at 122 min. This comparison in response trends clearly indicate that the above developed model is capable of tracing the response of beams under fire conditions.

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6.0 CRITIAL FACTORS GOVERNING LOCAL BUCKLING UNDER FIRE CONDITIONS

The validated finite element model was applied to study the effects of beam-slab interaction, strength properties (Grade) of steel and fire insulation on the onset of local instability and capacity degradation in steel beams exposed to fire. Results from finite element analysis is examined to evaluate failure of beams under different failure limit states including flexure, shear, sectional and deflection criteria, but due consideration was given to isolate the effect of sectional stability.

6.1 Analysis details

The beam selected for analysis is a simply supported beam of W16×31 section fabricated from Grade 345 MPa steel and is taken from AISC design manual [10]. The flange width is of 140.6 mm and overall depth of this section is 404 mm while the corresponding flange and web thicknesses are 11.2 and 7 mm, respectively. The beam is subjected to combined bending and shear loading and exposed to ASTM E119 fire exposure. To simulate high shear loading, a uniformly distributed loading (UDL) across the whole span of the beam, and two concentrated loads close to supports are applied as shown in Fig. 7.

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 scenario the girder is exposed to. Once sectional temperatures are generated from thermal analysis, they are input to the second stage of fire analysis. Both temperature and loading is applied simultaneously to the second stage of fire analysis to carry out structural analysis. In the structural analysis, mechanical response such as mid-span deflection, stress, strain and stability states along with sectional capacity of fire exposed beam is evaluated.

As part of numerical study, the effect of three main parameters are studied, namely, the effect of beam-slab interactions, strength properties (Grade) of steel and presence of fire insulation on the onset of local buckling of steel girders exposed to fire conditions. In order to vary critical parameters, seven beams, with varying characteristics, are analyzed and the variables are shown in Table 1. All beams were exposed to standard fire exposure scenario (ASTM E119).

6.2 Effect of composite action

In beam-slab composite assemblies both steel beam and concrete slab contribute to resist the applied loading through composite action. The beneficial role of composite action on flexural response at ambient conditions is well established and is accounted for in AISC manual and Eurocode design provisions. However, despite experimental evidence, contribution of composite action is not taken into account in evaluating shear capacity and local buckling in composite assemblies [5-9, 24]. The effect of composite action arising from presence of concrete slab on the onset of local buckling in fire exposed steel beams is studied herein by analyzing a bare steel beam, "Beam 1" (without any composite action), and a composite beam "Beam 2" under fire conditions. The concrete slab considered in "Beam 2" is of width 115 mm and thickness of 1145 mm. Also, in this beam, full composite action between steel beam and concrete slab is facilitated by providing forty ½×2½ shear studs as per AISC provisions. These studs are placed symmetrically in two rows and spaced at 229 mm (center-to-center). These shear studs have an ultimate tensile strength of 420 MPa and a shear capacity of 51.6 kN.

In the analysis, "Beam 2" was subjected to the same loading used in "Beam 1" (see Fig. 7) and exposed to ASTM E119 fire exposure. Generated thermal results from this analysis is shown in Fig. 8. These results show predicted temperature progression in the web of steel beam, as well as in the concrete slab at a depth of 50 mm and at the top surface of slab. Temperatures in both steel beams (Beam 1 and Beam 2) rise rapidly after the first 8 min of fire exposure, however, temperature progression in the concrete slab increases at a much lower rate due to better thermal inertia of concrete (much lower thermal conductivity and higher heat capacity of concrete). At 30 minutes into fire exposure, temperatures in concrete slab remain below 380˚C at depth of 50 mm and 220˚C at top surface of the slab. Thus, concrete slab continues to contribute to moment and shear capacity, especially once much of steel strength and stiffness is lost due to increased temperatures.

In order to further analyze Beams 1 and 2, degradation of the flexural and shear capacity (at critical sections) with fire exposure time in these beams is plotted in Fig. 9. It can be seen from Fig. 9a that the presence of the concrete slab greatly enhances moment capacity of the composite beam (Beam 2) at ambient and fire conditions. Accounting for composite action increased the overall moment capacity at room temperature by 55%. Following flexural limit state, flexural failure of "Beam 1" occurs at an earlier time (14 min), when compared to flexural failure of "Beam 2" (28 min).

Figure 9b shows shear response of Beams 1 and 2. As mentioned earlier, AISC provisions assume the web of the W-shaped section to provide full shear resistance to the applied loading and ignores contribution of concrete (slab) to shear capacity. According to these provisions, both

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Beams 1 and 2 have similar shear capacity and are to fail at the same time. However, predictions from analysis show that failure of Beams 1 and 2 occurs at different times. In order to distinguish the effect of web local buckling on response of Beams 1 and 2, two scenarios of accounting for local buckling of web (WLB) and not considering local buckling (no WLB) are plotted in Fig. 9b. It is clear from Fig. 9b that loss in shear capacity in these beams occurs due to local buckling effects early into fire test. Then, additional losses due to strength degradation of steel arises at a later stage.

Failure of Beams 1 and 2 is initiated by web local buckling which start to occur early into fire exposure. It is clear that initiation of web local buckling occurs regardless of beam configuration (bare beam or composite beam) since local buckling is a function of steel temperature and steel section properties. When the response of Beams 1 and 2 is compared, it can be seen that response of "Beam 2" is much better than that of "Beam 1". This is due to the positive contribution of concrete slab to sectional capacity which enhances failure time of this beam. "Beam 2" fails at 16 min (when web temperature is 780˚C) as compared to failure of "Beam 1" in 11.6 min (when web temperature is 560˚C).

The progression of mid-span deflections with fire exposure time is plotted in Fig. 10. The initial deflection in the composite beam (Beam 2) is slightly lower than that of the bare steel beam (Beam 1). This can be attributed to the additional stiffness provided by the concrete slab. Then, both beams undergo a steady mid-span deflection until steel temperature starts to increase. Once steel temperature reaches 500˚C, strength and elastic modulus of steel starts to degrade rapidly and the neutral axis of Beams 1 and 2 starts shifting upwards to balance internal tensile and

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compressive stresses. At this point, "Beam 1" experiences faster and larger amount of deflection as that compared to "Beam 2" due to absence of concrete slab. This leads to early failure of "Beam 1" at about 11.6 min, when temperature in web reaches 560˚C. On the other hand, presence of cooler concrete slab, in "Beam 2", continues to contribute in resisting the applied forces and aids in delay failure of this beam. The mid-span deflection of "Beam 2" stabilizes before the beam fails at 16 min which occur after web temperature reaches 780˚C. "Beam 2" fails when applied shear force exceeds its degrading shear capacity due to local buckling and strength degradation. It should be noted that mid-span deflection of "Beam 2" is 150 mm at failure (16 min), which is significantly lower than that obtained from the non-composite steel beam "Beam 1" that reaches 200 mm at 11.6 min. It can be seen that occurrence of local buckling do not significantly affect mid-span deflection of fire exposed beams since it occurs locally in steel web. However, local buckling can lead to significant loss of shear capacity.

6.3 Effect of strength properties (Grade) of steel

In order to study the effect of mechanical properties (Grade) of steel on web local buckling in fire exposed steel beams, an additional "Beam 3" is analyzed using the above developed model. "Beam 3" has the same geometrical features, loading set-up and boundary conditions of that of "Beam 1", but is assumed to be made of Grade 36 steel (f_y = 250 MPa) instead of Grade 50 (345 MPa) used in "Beam 1".

The variation of strength (Grade) of steel does not influence the thermal properties of steel and hence temperature progression in Beams 1 and 3 is similar as can be seen in Fig. 8. However, strength analysis of these beams produces quite different response under fire conditions. Figure 11 shows degradation of moment and shear capacity of Beams 1 and 3 with fire exposure time. It is

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clear that room temperature flexural and shear capacity of "Beam 3" is lower than that of "Beam 1" due to the lower strength (Grade) of steel used. As discussed above, flexural failure of "Beam 1" occurs at 14 min, when average steel temperature reach 630˚C. Similar to "Beam 1", "Beam 3" also fails at 14 min. This can be attributed to several factors, for instance both beams were subjected to loading equivalent to 50% of their room temperature moment capacity. In addition, since temperature progression in both beams is independent of grade of steel and strength properties in both steel grades degrade at the same rate, these beams fail at similar steel temperatures and times.

Figure 11b shows shear response of Beams 1 and 3 when exposed to fire conditions. It can be seen that initial shear capacity of "Beam 3" is less than that of "Beam 1" due to lower yield strength (Grade) used in "Beam 3". It can be also seen that effect of web local buckling is less apparent in "Beam 3" than that in "Beam 1". Since web local buckling limit states is a function of ($\sqrt{E/f_y}$), the use of a lower yield strength (f_y) increases the limiting value for local buckling (see Fig. 12). To illustrate this, web sectional slenderness of Beams 1 and 3 (λ_w= 57.8) is compared against compactness limit (λ_p) of steel with yield strength of 345 MPa (λ_p = 1.10 $\sqrt{\frac{k_v E}{f_y}}$ = 59.24) and 250 MPa (λ_p = 1.10 $\sqrt{\frac{k_v E}{f_y}}$ = 69.6). It is clear that local buckling limit of steel made of yield strength of 250 MPa is 17.8% greater than that of steel made of yield strength of 345 MPa. Thus, for identical steel sections made of 250 MPa and 345 MPa steel, the section with lower yield strength can have better resistance to web local buckling under similar conditions and loading levels.

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To further illustrate this, web slenderness of Beams 1 and 3 exceeds that of compactness limit of Grade 345 and 250 MPa at 150 and 400˚C, respectively (see Fig. 12). Thus, “Beam 1” experiences web local buckling at lower steel temperatures and earlier times than that of “Beam 3”. It can be also seen from Fig. 12 that web local buckling of “Beam 3” occurs at 400˚C, which is the same temperature at which strength starts to degrade. This is opposite to the case of “Beam 1” where web local buckling occurs at 150˚C, but strength degradation occurs at 400˚C.

Figure 13 shows progression of mid-span defection of Beams 1 and 3. As expected, “Beam 3” experiences larger initial deflection than “Beam 1”. This can be attributed to lower yield strength used in “Beam 3”. Then, similar to “Beam 1”, “Beam 3” also undergoes steady deflection levels at the start of fire analysis. Then, deflection levels in both beams start to increase as steel temperature increases. Mid-span deflection of “Beam 3” increases at faster pace than mid-span deflection of “Beam 1” once local buckling and strength degradation effect take place. “Beam 3” exceeds the allowable deflection limit at about 12 min, while “Beam 1” exceeds the same limit at 13.8 min. Table 3 shows failure times and temperature in web and flanges of these beams.

6.4 Effect of fire insulation

In order to study the effect of fire insulation on shear response and local instability of fire exposed steel beams, four additional beams (Beams 4, 5, 6 and 7) were analyzed using the developed finite element model. Beams 4, 5, 6 and 7 are replicates of “Beam 1” but protected with 12.5, 19, 25 and 50 mm thick vermiculite/gypsum (VG) insulation, respectively.

A comparison of temperature progression in beams with different insulation thicknesses is plotted in Fig. 14. In the first few minutes of fire exposure, all four insulated steel beams experience

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slight rise in temperature. However, beyond that, temperature rise in web increases as a function of insulation thickness and the higher insulation thickness, the lower is the steel temperature.

The lower temperature in insulated beams leads to slower degradation in strength and stiffness properties. This leads to delay in occurrence of local buckling as well as delay in moment and shear capacity degradation (see Fig. 15a). Moment capacity starts to degrade at 20, 26 and 33 min for Beams 4, 5 and 6, respectively; a significant delay than that in "Beam 1" where degradation of moment capacity starts at 9 min. Once the moment capacity decreases to the level of bending moment due to applied loading, failure occurs. Under flexural limit state, failure of Beams 4, 5 and 6 occurs at 39, 65 and 80 min, respectively. It should be noted that moment and shear capacity of "Beam 7" remains intact during entire fire exposure duration since sectional temperature (both web and flanges) remain below 400˚C due to much thicker fire insulation.

Local instabilities and degradation in shear capacity in the insulated beams starts once steel temperature is in the range of 150-400˚C. The effect of local buckling of web on shear capacity is illustrated in Fig. 15b where the onset of web local buckling occurs at 8.2, 10, 13 and 22 min for Beams 4, 5, 6 and 7, and shear capacity starts to degrade at 5, 10, 13.4 and 16.2 min for Beams 1, 4, 5 and 6. In addition, Fig. 15b clearly shows how the onset of web local buckling increase the rate of degradation of shear capacity and accelerate failure in beams. When temperature-induced local buckling of web is neglected (following current fire provisions in codes and standards), Beams 4, 5 and 6 fail at 37, 55 and 67 min, respectively. However, when temperature-induced local buckling of web is accounted for, failure of these beams occur at earlier times, namely at 34, 39 and 55 min, respectively. In general, onset of web local buckling occurs when temperature in web is in the range of 150˚C. These temperature-induced web local buckling can lead to higher degradation in shear capacity of steel beams with slender webs.

Further, both Beams 5 and 6 achieved 1-hour fire rating, as per prescriptive criterion under flexural limit state. However, these beams do not yield 1-hour fire rating under shear limit state when local buckling is accounted for. Therefore, shear failure can significantly lower failure times which in turn may lead to unconservative fire resistance under certain scenarios.

Figure 16 compares predicted mid-span deflection in Beams 1, 4, 5, 6 and 7 to the British Standard BS-476 deflection limit state [22]. In all beams, mid-span deflections remain low at the initial stages of fire exposure. Then, mid-span deflections gradually increase with increasing fire exposure time. In general, Beams 4 and 5, with thinner fire insulation undergo larger deflections than Beams 6 and 7 throughout fire. Failure times of these beams is also detailed in Table 4. Overall, failure in this group of beams occur primarily in shear limit state (as shown in Table 4). This failure mainly results from web local buckling since the onset of web local buckling occurs when web temperature is in the range of 150-200˚C, at 8.2, 10, 13 and 22 min for Beams 4, 5, 6 and 7, and shear capacity starts to degrade at 10, 13.4 and 16.2 min.

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 effect of temperature-induced local buckling in tracing the response of steel beams exposed to fire conditions.

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2. Onset of temperature-induced local buckling in web can induce stability related failure in steel beams. This effect, in combination to temperature-induced strength loss, can accelerate shear failure before attaining flexural capacity.

3. The presence of concrete slab enhances sectional capacity and counterbalance adverse effect of temperature-induced web local buckling in fire exposed steel beams.

4. The strength (Grade) of steel determines the temperature at which local buckling occurs in fire exposed steel beams. In steel beams made of Grade 345 MPa steel, local buckling can occur when steel temperature reach 150˚C, while in steel beams made of Grade 250 MPa, local buckling occur at much higher steel temperatures (around 400˚C).

5. Use of fire insulation can delay onset of temperature-induced local buckling in flanges and web and enhance fire resistance of fire exposed steel beams.

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Table 1 Factors varied in parametric study on steel beams

Beam	Variable		
	Fire insulation	Strength properties (Grade) of steel	Composite action

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Beam 1	None	Grade 50 (345 MPa)	None
Beam 2	None	Grade 50 (345 MPa)	Yes
Beam 3	None	Grade 36 (250 MPa)	None
Beam 4	12.5 mm	Grade 50 (345 MPa)	None
Beam 5	19 mm	Grade 50 (345 MPa)	None
Beam 6	25 mm	Grade 50 (345 MPa)	None
Beam 7	50 mm	Grade 50 (345 MPa)	None

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Table 2 Failure in Beams 1 and 2

Beam	Failure time (min)				Temp. at failure (˚C)		Failure mode
	Shear (no WLB)	Shear (WLB)*	Flexure	Deflection	Web	Flanges	
Beam 1	13	**11.6**	14	13.8	560	540	Shear due to local buckling
Beam 2	18	**16**	28	-	780	690	Shear due to local buckling

* Initiation of web local buckling occurs after 3 min of fire exposure (for Beams 1 and 2)

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Table 3 Failure in beams with different properties of steel

Beam	Grade of steel MPa (ksi)	Failure time (min)				Temp. at failure (˚C)		Failure mode
		Shear (no WLB)	Shear (WLB)*	Flexure	Deflection	Web	Flanges	
Beam 1	345 (50)	13	**11.6**	14	13.8	560	540	Shear
Beam 3	250 (36)	11.6	**11**	13	12	550	500	Shear

* Initiation of web local buckling at 3 and 8 min in web of Beams 1 and 3, respectively

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Table 4 Failure in beams with different insulation thicknesses

Beam	Failure time (min)				Temp. at failure (˚C)		Failure mode
	Shear (no WLB)	Shear (WLB)*	Flexure	Deflection	Web	Flanges	
Beam 1	13	**11.6**	14	13.8	560	540	Shear
Beam 4	37	**34**	39	-	540	480	Shear
Beam 5	45	**39**	65	-	609	530	Shear
Beam 6	66	**55**	80	-	565	490	Shear
Beam 7	No failure	No failure	No failure	-	-	-	No failure

* Initiation of web local buckling occurs at 8.2, 10, 13 and 22 min for Beams 4, 5, 6 and 7, respectively

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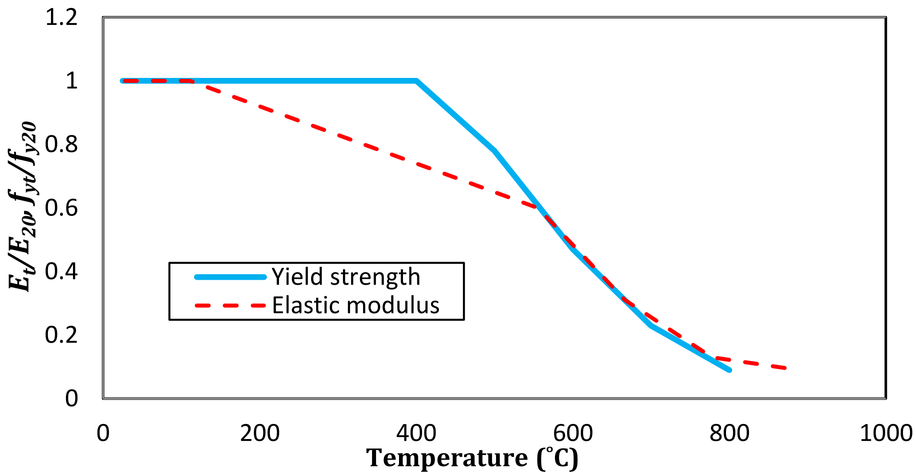

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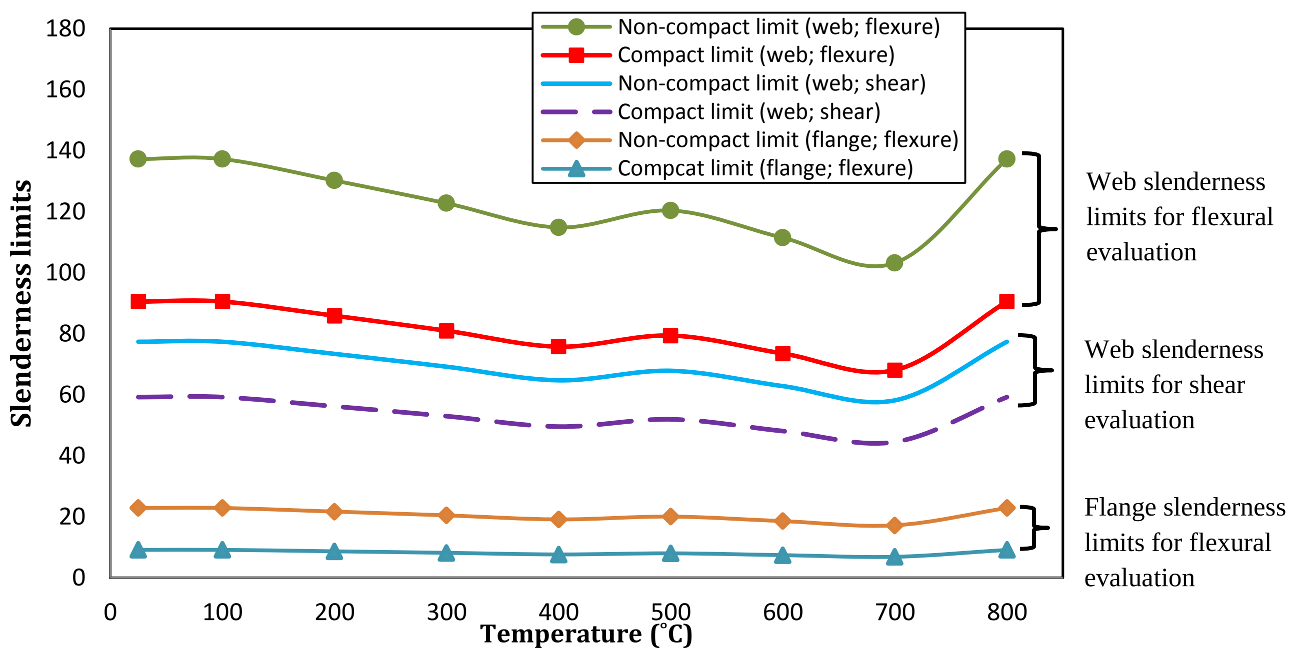

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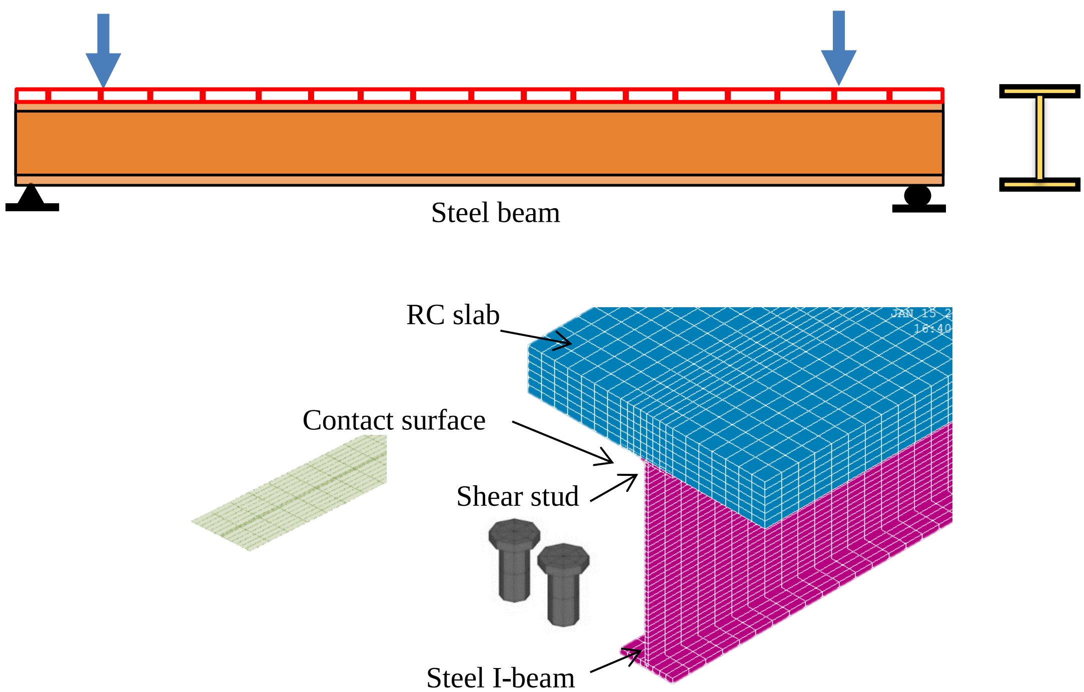

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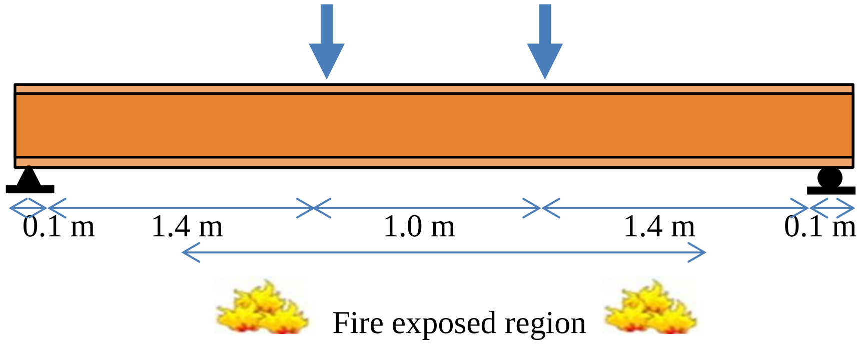

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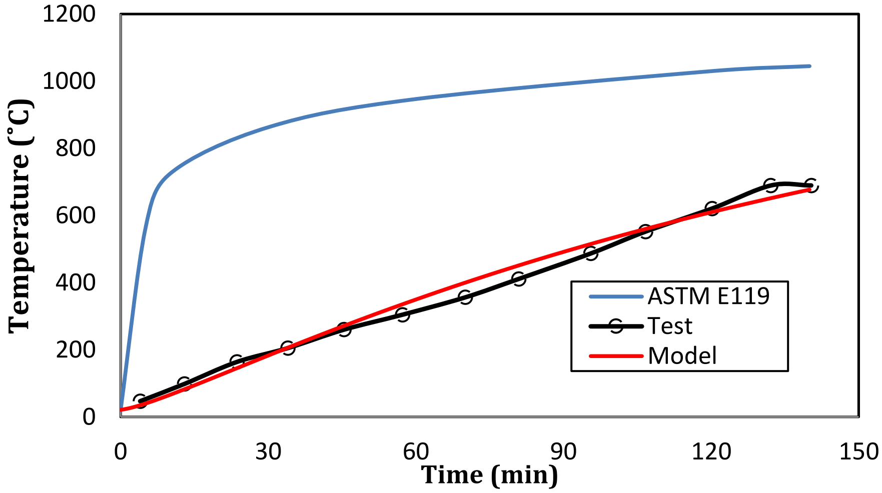
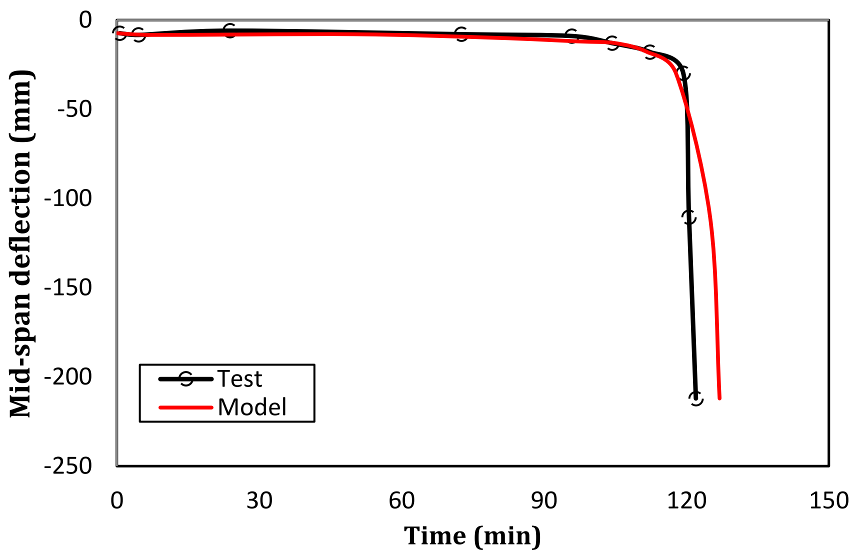


(a) Thermal response



(b) Structural response

Fig. 5. Comparison of predicted and measured temperature and deflections as a function of fire exposure time

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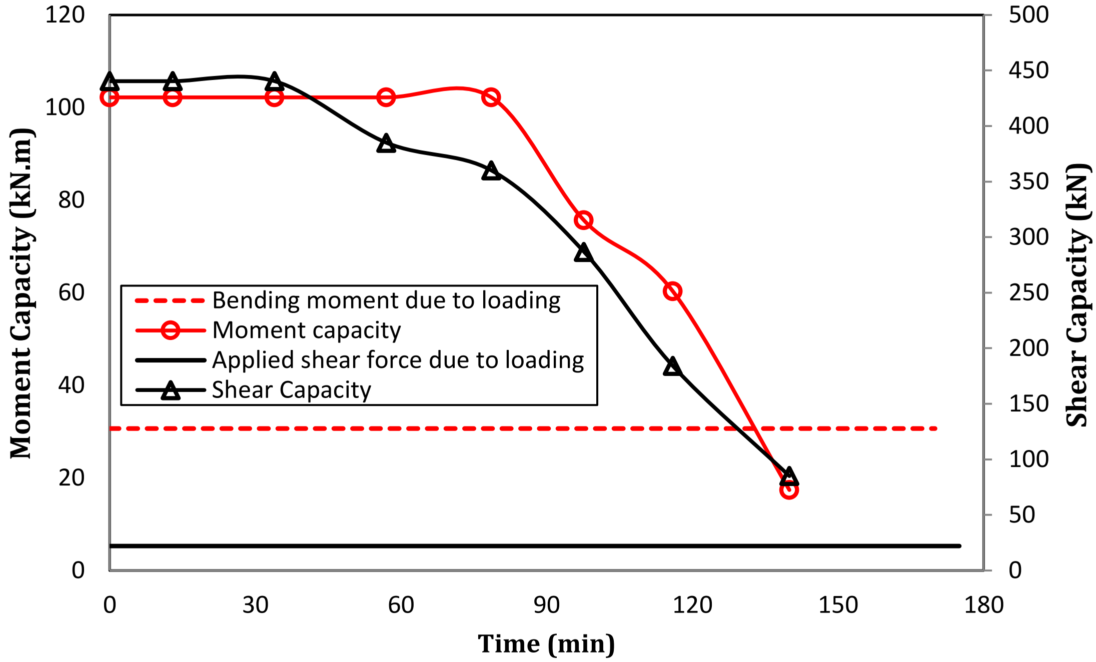

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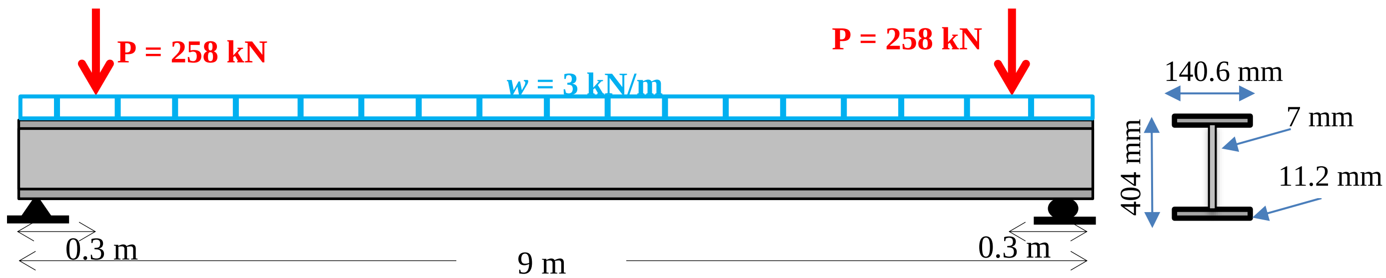

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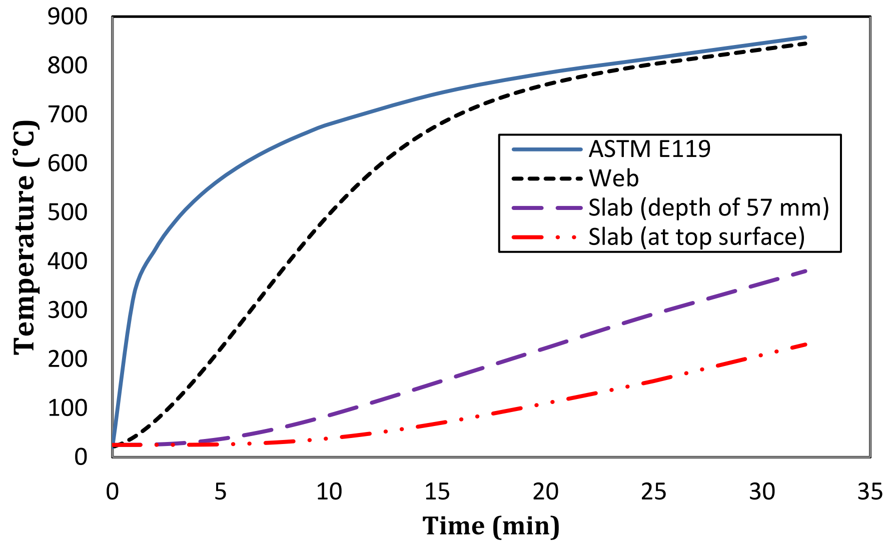

Fig. 8. Temperature in “Beam 5” as a function of fire exposure time

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Please cite this paper as:
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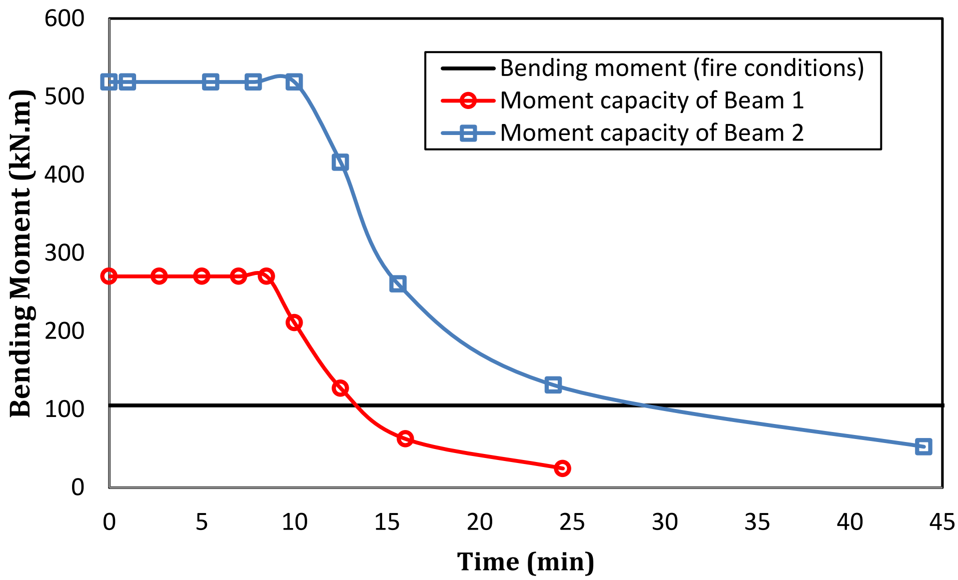
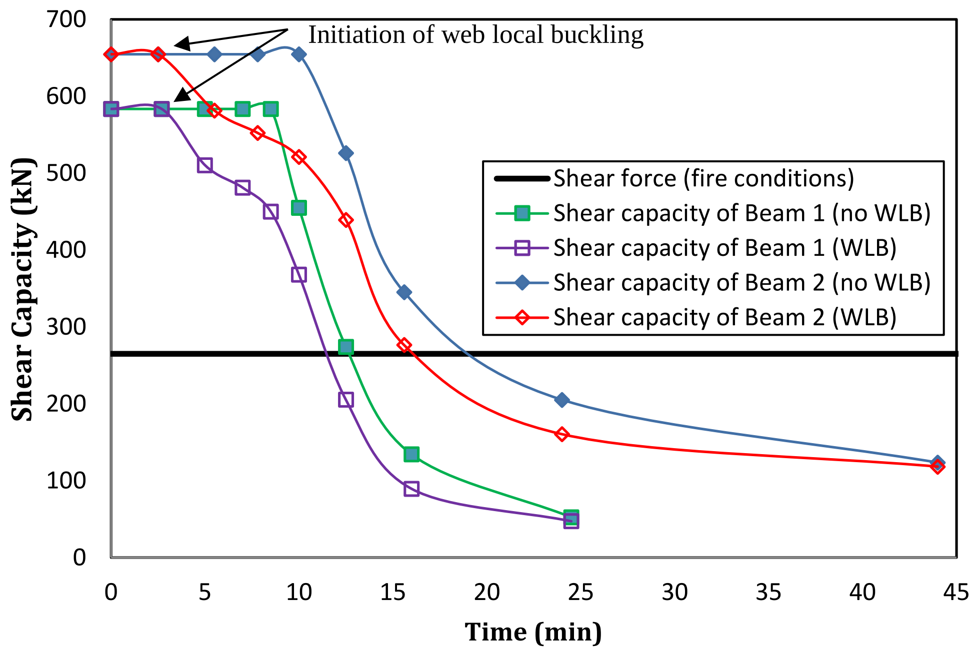


(a) Moment capacity



(b) Shear capacity

Fig. 9. Degradation of moment and shear capacity with fire exposure time in composite beam

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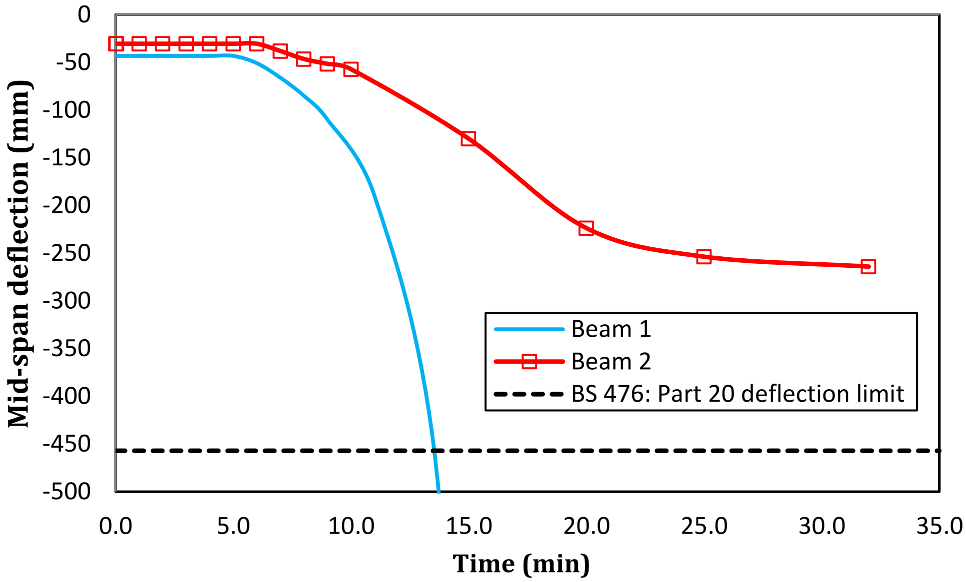

Fig. 10. Mid-span deflection in "Beam 1" and "Beam 2" with fire exposure time

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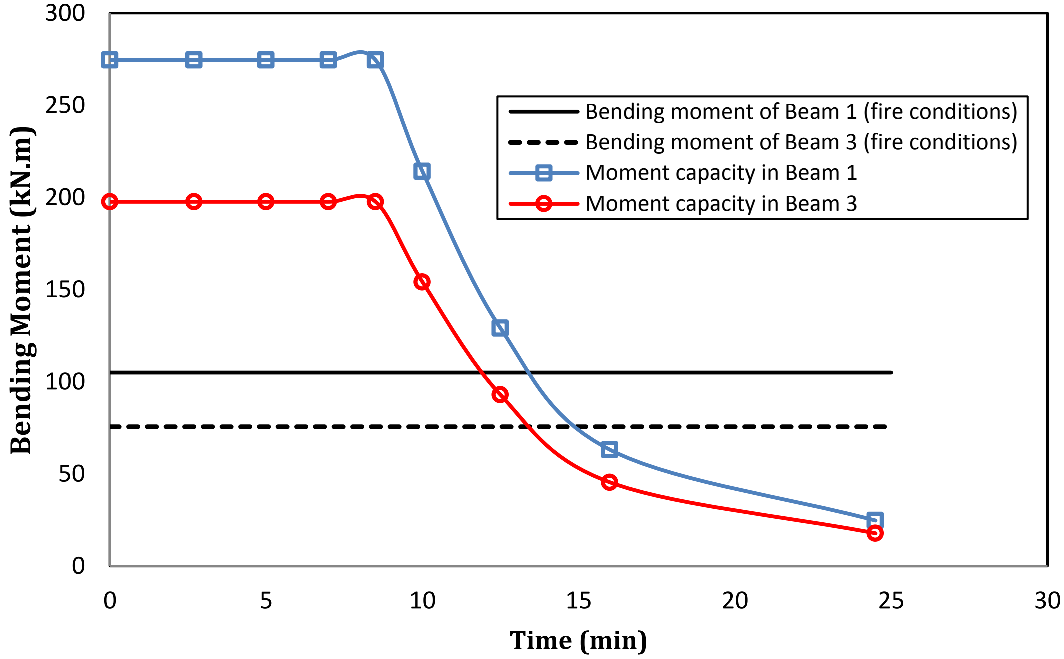
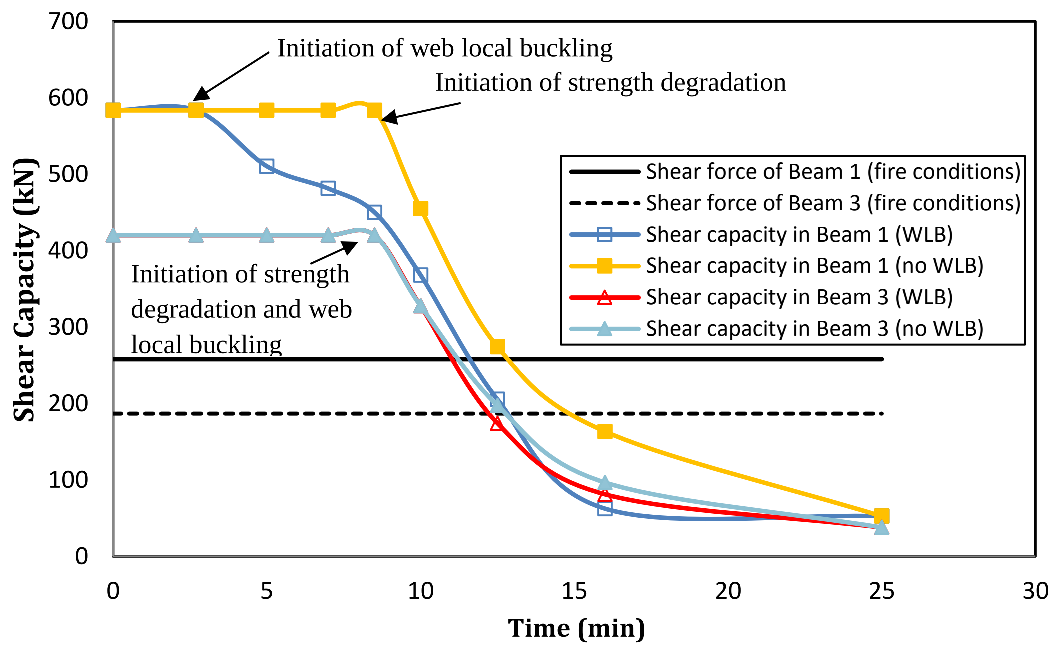


(a) Moment capacity



(b) Shear capacity

Fig. 11. Degradation of flexural and shear capacity in Beams 1 and 3 with fire exposure time

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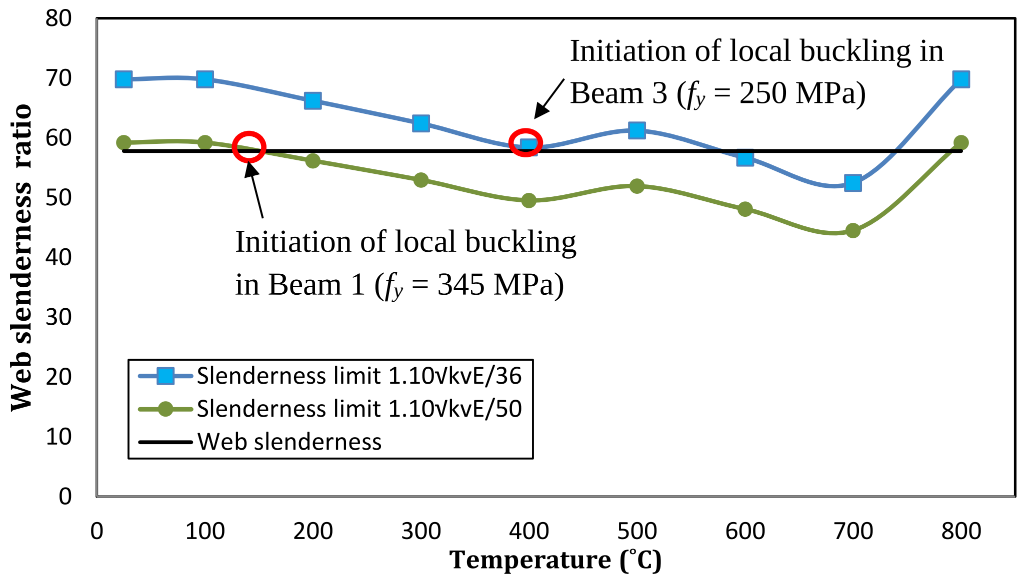

Fig. 12. Effect of properties of steel on local buckling of steel beam

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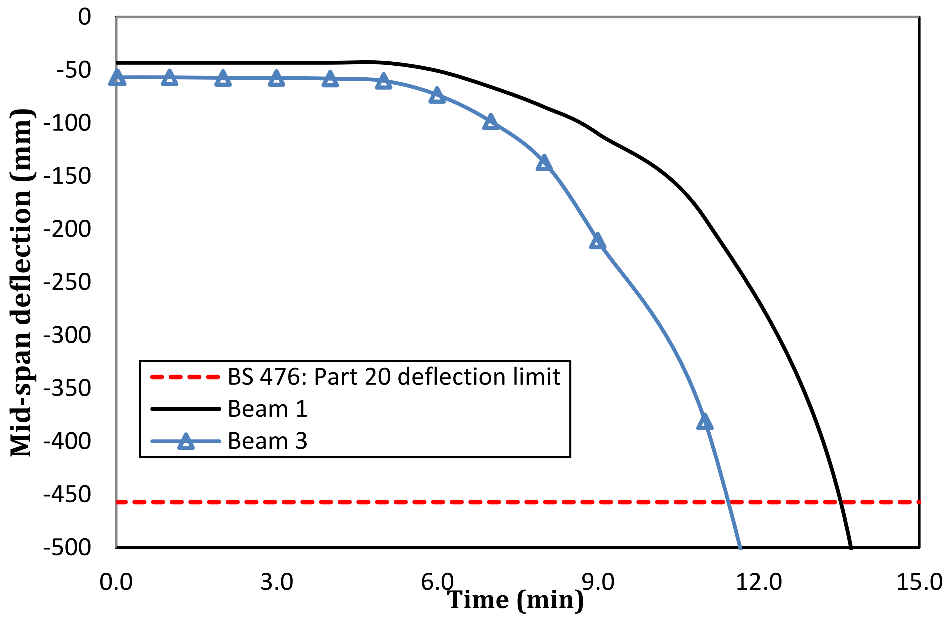

Fig. 13. Comparison between mid-span deflections in "Beam 1" and "Beam 3"

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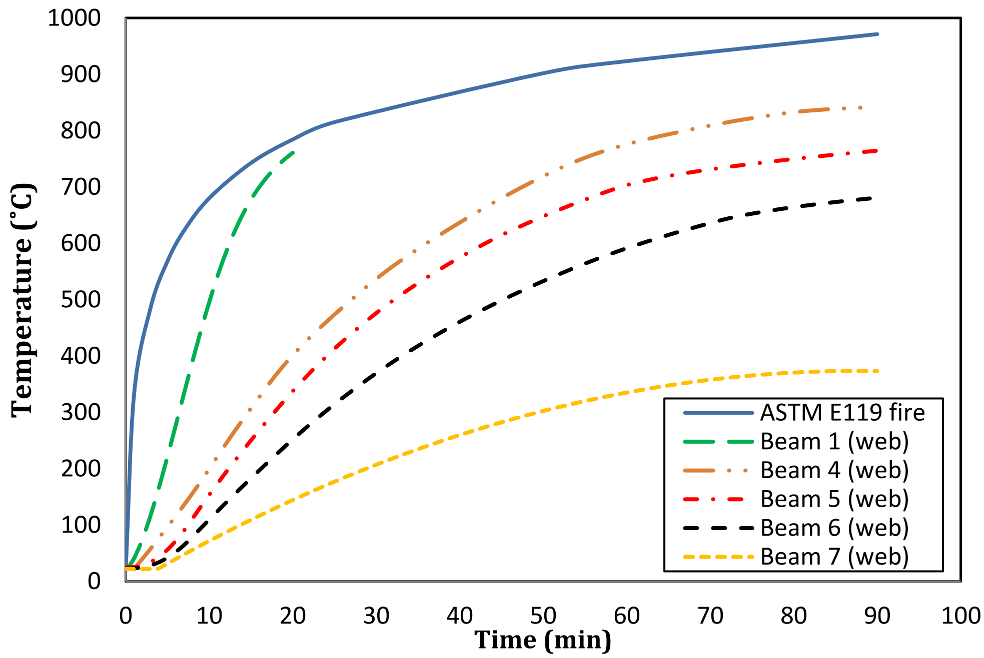

Fig. 14. Temperature propagation in web of Beams 1, 4, 5, 6 and 7

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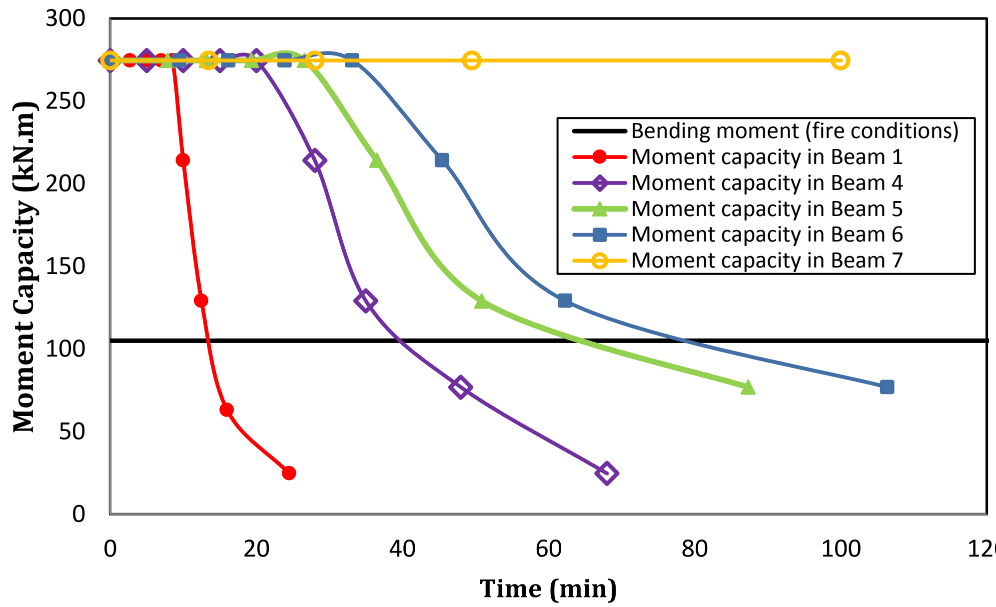
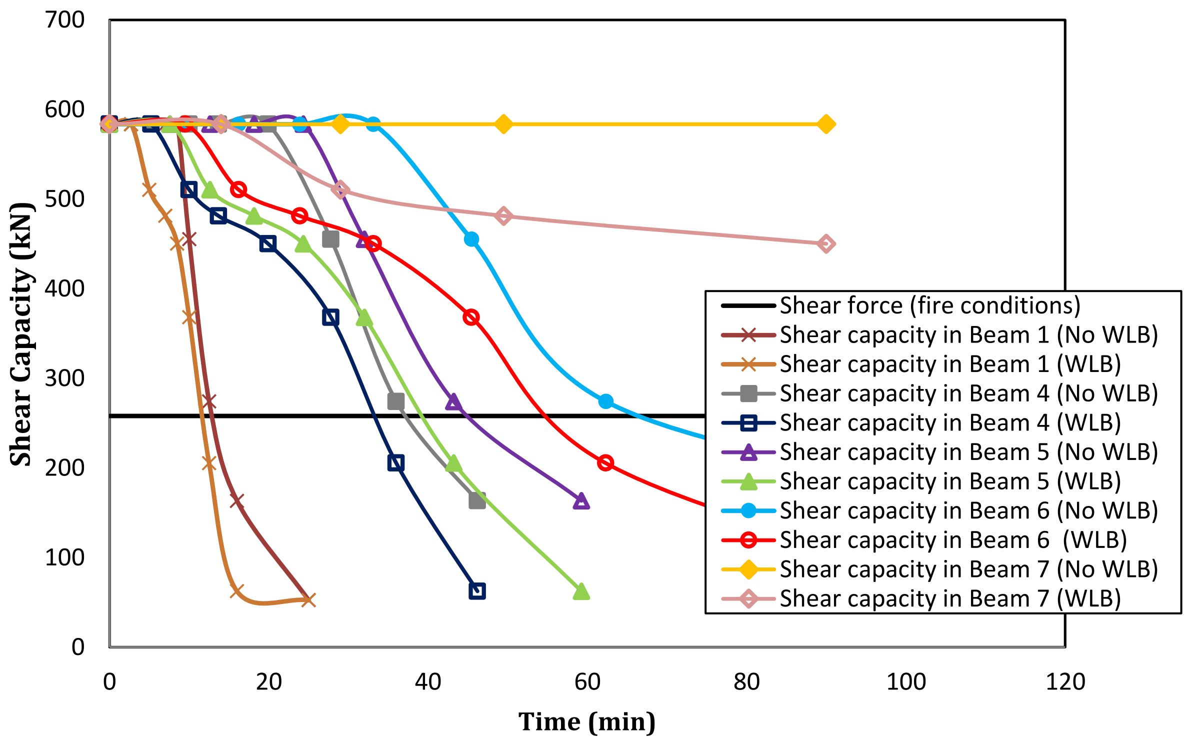


(a) Moment capacity



(b) Shear capacity

Fig. 15. Degradation of flexural and shear capacity in Beams 1, 4, 5, 6 and 7 with exposure time

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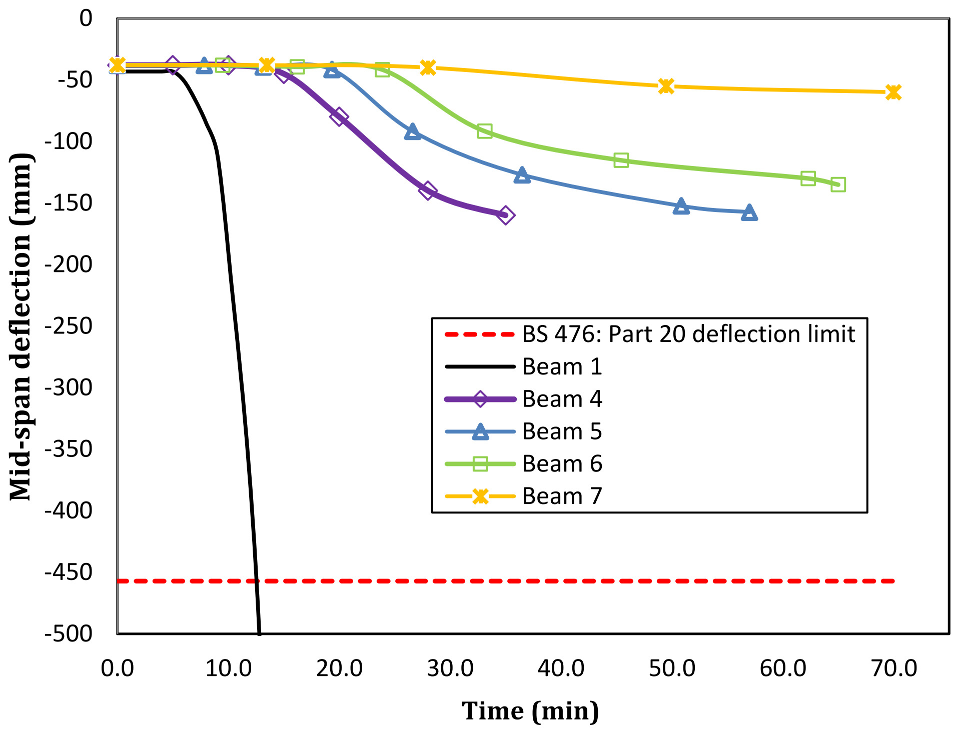

Fig. 16. Comparison between mid-span deflections of steel W-beams with different insulation thickness