This is a preprint draft. The published article can be found at: https://doi.org/10.1016/j.jcsr.2017.11.011

Please cite this paper as:
Kodur V.K.R., Naser M.Z. (2018). "An Approach for Evaluating Shear Capacity of Steel and Composite Beams Exposed to Fire Conditions." Journal of Constructional Steel Research. Vol. 141, pp. 91–103. (https://doi.org/10.1016/j.jcsr.2017.11.011)

Approach for Shear Capacity Evaluation of Fire Exposed Steel and Composite Beams

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1.0 ABSTRACT

A simplified approach for evaluating degradation of shear capacity in fire exposed steel and composite beams is presented. This approach takes into account temperature-induced strength degradation, and sectional instability effects, as well as level of composite action developed at the beam-slab interface. The validity of the approach in evaluating shear capacity of fire exposed steel and composite beams is established by comparing predictions from the proposed approach with results obtained from finite element analysis and fire tests. Results generated from numerical studies and illustrative examples infer that the proposed approach can evaluate degradation in shear capacity of fire exposed steel and composite beams under wide range of loading scenarios.

Keywords: *S*hear, fire resistance; steel and composite beams; design approach; instability effects; composite action.

2.0 INTRODUCTION

Steel structures experience steep rise in temperature when subjected to fire conditions due to high thermal conductivity and low specific heat of steel. This rise in temperature can lead to rapid degradation of strength and elastic modulus of steel. Hence, steel structural members can lose much of their load carrying capacity within the first 20-25 minutes of typical fire exposure

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conditions [1-6]. Therefore, steel structures are to be provided with fire insulation to achieve required fire resistance (1-3 hours) in building applications.

In most building applications, steel beams are built in situ with concrete slab, connected through welded shear connectors, thus forming composite beam-slab assemblies. In such an arrangement, steel beam is designed to resist tensile forces and concrete slab is designed to resist much of the compressive forces, and both elements act as one unit through composite interaction developing between them. Since concrete possess better fire resistance properties than steel, such as low thermal conductivity, high specific heat, and slow rate of strength loss, fire resistance of a composite beam is much higher than that of an isolated steel beam under most fire conditions [7-9]. Hence, if contribution of concrete slab is properly accounted for, then it is possible in some scenarios to achieve required fire resistance of beam-slab assemblies without the need of external fire insulation [6, 8].

Fire resistance of steel and composite beams has been investigated in numerous experimental and numerical studies [6-10]. In majority of these studies, failure of a steel beam was assumed to occur when mid-span deflection exceeds a certain limiting criterion or when moment capacity drops below the level of bending moment (arising from applied loading). However, recent studies have shown that shear limit state can govern failure of beams especially under certain loading and fire conditions [11-15]. Based on fire resistance analysis, it has been shown that beams made of hot-rolled steel sections can experience significant temperature-induced strength degradation and instability effects due to temperature rise and beams can fail in shear mode before attaining failure under deflection (as deflection is minor due to shear loading) or flexural limit state (due to occurrence of temperature-induced instability and rapid degradation in shear capacity).

Despite this phenomenon, shear limit state under fire conditions is still not incorporated in many of current fire design guidelines, where flexural limit state continue to be the main governing failure criterion.

To be more specific, current fire design requirements for steel structures in standards such as AISC [16], Eurocode 3 and 4 [17, 18] and AS 2327.1 [19] refer designers to specified requirements for design of steel structural members at room temperature which may not be representative under certain loading, such as high shear and fire loading. These guidelines do not explicitly specify to check for shear failure under fire conditions. Further, temperature-induced local buckling effects in steel beams is not taken into account and it is assumed that steel beams can maintain their room temperature stability classification (i.e. compact, non-compact and slender) even under fire conditions. Recent studies have shown such assumptions to be unconservative under certain fire loading and sectional configurations [6, 8, 11, 12].

Current fire design requirements also require composite beams to be designed such that applied shear force is resisted by web alone, hence neglecting any contribution of concrete slab to shear capacity. Not only that, this approach contradicts flexural design guidelines, which accounts for contribution of concrete slab to moment capacity. Test results generated in recent studies have shown that concrete slab can enhance shear capacity in composite beams, under ambient and fire conditions, by 10-40% [11-15]. Therefore, current design guidelines may not lead to realistic assessment of behavior of steel and composite beams, specifically when subjected to high shear and fire loading.

To overcome some of the above stated limitations, an approach for evaluating shear capacity of steel and composite beams as a function of fire exposure is presented in this paper. The

proposed approach accounts for temperature-dependent property degradation of constituent materials, temperature-induced sectional instability and level of composite action developing between steel beam and concrete slab, in evaluating shear capacity of steel and composite beams. The validity of the proposed approach is established and its applicability to design situations is illustrated through numerical examples.

3.0 LIMIT STATES GOVERNING FAILURE UNDER FIRE CONDITIONS

Till 1980's, fire resistance ratings for steel beams was assessed through prescriptive based tabulated listings and empirical methods [20]. These listings and empirical methods are derived from data generated from standard fire tests, in which failure of fire exposed steel beam is said to occur when it reaches a critical temperature corresponding to 50% strength loss in steel. However, in the last three decades, there has been growing realization that the current prescriptive approaches have numerous drawbacks, including limited applicability, as these approaches do not account for critical design parameters such as realistic fire exposure, load level, restraint conditions and all possible failure limit states in evaluating fire resistance.

Hence, strength limit state was introduced in deriving failure of steel and composite members. As a result, failure of fire exposed steel and composite beams is said to occur when degrading flexural capacity exceeds the applied bending moment during fire exposure, without due consideration to degrading shear capacity [16]. This is in contrast to ambient temperature design, where a beam is to be designed to satisfy flexural limits state and then checked for shear resistance. Deriving failure in fire exposed beams based on flexural limit state, although valid for most common scenarios, may not be representative in certain situations where shear forces are dominant or shear capacity degrades at a rapid pace with fire exposure time.

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Shear forces can be dominant in beams subjected to certain loading configurations such as high concentrated (point) loads acting on beams, as in the case of transfer girders [21]. Also, shear effects can be significant in short beams, beams with copped ends and beams with slender webs, as in the case of deep beams and plate girders. Further, webs in such deep beams as well as plate girders are usually much more slender (thinner) than flanges. These webs can experience rapid rise in temperature since they are exposed to fire from two sides (i.e. have larger surface area). Hence, strength properties of steel in the web can degrade at a rapid rate than that in flanges. Thus, under high temperature exposure, shear capacity of steel beams can degrade at a much higher rate than flexural capacity. Therefore, in these scenarios, shear failure can be governing limit state and need to be accounted for.

4.0 APPROACH FOR EVALUATING SHEAR CAPACITY UNDER AMBIENT AND FIRE CONDITIONS

From above discussion, it is clear that current provisions of evaluating fire resistance of steel and composite beams based on flexural limit state only may not be representative under certain scenarios. In order to consider shear limit state, an approach for evaluating degradation in shear capacity as a function of fire exposure in steel and composite beams is derived. This approach accounts for temperature-induced property degradation, sectional instability effects, as well as level of composite action that develops at the beam-slab interface.

4.1 Shear capacity of steel beams under ambient conditions

The basis for evaluating shear capacity of steel beams at room temperature is derived from principles of von Mises yield criterion (Huber-von Mises-Hencky [22]). In this theory, the uniaxial yield stress (in terms of the three principal stresses) is given by;

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$$\sigma_y^2 = \frac{1}{2}[(\sigma_1 - \sigma_2)^2 - (\sigma_2 - \sigma_3)^2 - (\sigma_1 - \sigma_3)^2] \quad (1)$$

where, σ_1, σ_2 and σ_3 is the tensile and compressive stresses in the three principle directions that act in the three mutually perpendicular planes of zero shear, and σ_y is the yield stress that to be compared against uniaxial yield stress, f_y.

Huber [22] states that in typical civil engineering applications, one of the three principal stresses is either zero or small enough to be neglected. Hence, Eq. 1 reduces to;

$$\sigma_y^2 = \sigma_1^2 + \sigma_2^2 - \sigma_1\sigma_2 \quad (2)$$

Since pure shear occurs on 45° plane to the principle planes, i.e. when $\sigma_2 = -\sigma_1$, then corresponding shear stress equals principle stress ($\tau = \sigma_1$). Substitution of $\sigma_2 = -\sigma_1$ into Eq. 2 gives;

$$\sigma_y^2 = \sigma_1^2 + \sigma_1^2 - \sigma_1(-\sigma_1) = 3\sigma_1^2 \quad (3)$$

$$\sigma_1 = \tau = \frac{\sigma_y}{\sqrt{3}}$$

which indicates that the yield state for shear stress acting alone is;

$$\tau = \frac{\sigma_y}{\sqrt{3}} = 0.58 f_y \approx 0.6 f_y$$

(4)

Thus, nominal shear capacity (V_n) of a steel beam with a web area, A_w (where $A_w = d \times t_w$) at room temperature is evaluated as:

$$V_n = \tau_y \, d t_w \, C_v \quad (5)$$

where, τ_y is the shear yield strength of steel web ($= 0.6 f_y$), t_w is the thickness of the web, d is the overall depth for hot-rolled beams, C_v is the web shear coefficient to account for instability effects in web and depends on slenderness of web.

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Since shear stress is taken to be equal to $0.6f_y$ (from Eq. 4), nominal shear capacity is taken as listed in Eq. G2-1 in AISC design manual:

$$V_n = 0.6f_y\, A_w\, C_v \tag{6}$$

It can be seen that shear capacity in steel beams is a function of yield strength of steel and area of web which comprises of depth and thickness of web. These two parameters (d and t_w) are also used to classify web slenderness through a slenderness ratio (λ, where $\lambda = d/t_w$).

The effect of web slenderness on shear capacity of a steel beam is illustrated in Fig. 1a where shear capacity decreases with increased web slenderness. This figure also shows that there are three distinct zones in variation; i.e., shear yielding (plastic), inelastic buckling, and elastic buckling. The plastic zone corresponds to the case of a web not susceptible to any local buckling and this state represent a "compact" section phenomenon. In other words, a web with slenderness less than of that of the inelastic slenderness limit ($d/t_w < 1.10\sqrt{\frac{k_v E}{f_y}}$) will achieve its full shear capacity.

On the other hand, inelastic buckling zone corresponds to a web with slenderness ratio between $1.10\sqrt{\frac{k_v E}{f_y}}$ and $1.37\sqrt{\frac{k_v E}{f_y}}$. In this range, web buckling will occur after some part of web has yielded due to development of compressive stresses and activation of residual stresses (that can exist from fabrication/rolling process). A web in this zone will lose some of its shear strength due to development of inelastic buckling. Finally, elastic buckling zone corresponds to a web with sectional slenderness greater than that of $1.37\sqrt{\frac{k_v E}{f_y}}$, and in this case, web is considered to be a slender where failure of web can occur prematurely (in an elastic manner).

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4.2 Shear capacity of steel beams under fire conditions

The above principles can also be applied to evaluate shear capacity under fire conditions, if the effect of temperature-induced degradation in yield strength of steel, as well slenderness effects in web are to be dully account for. The degradation of yield strength of steel can be accounted for through temperature-dependent strength degradation factors which are specified in current codes and standards [16, 17].

Slenderness of web ($\lambda = d/t_w$) is a geometric property that is independent of material strength (grade), type of applied loading, boundary conditions etc. and remains "constant" even under fire conditions. At ambient conditions, web slenderness is to be compared against slenderness limits (i.e. $1.10\sqrt{\frac{k_v E}{f_y}}$ and $1.37\sqrt{\frac{k_v E}{f_y}}$) to classify web as compact (Class 1 and 2), non-compact (Class 3) or slender (Class 4). Under fire conditions, these limits vary (degrade) due to temperature-induced degradation in elastic modulus and strength properties. As a result, the three distinct zones, shown in Fig. 1a under ambient conditions, can shift depending on net loss in properties of steel. Figure 1b shows this shift as a function of varying cases of web slenderness at web temperatures of 25, 200, 400, and 600°C. It can be seen from this figure that web stability is influenced not only by web slenderness, but also with temperature rise in web.

In general, increase in web temperature lowers web stiffness to a certain degree (i.e. web has more flexibility to move laterally or buckle) due to faster degradation of elastic modulus in steel as compared to yield strength degradation. For example, Fig. 2 shows that a "compact" web, of slenderness of 57, at room temperature can transform into "non-compact" once web temperature exceeds 200°C. This transformation can lead to additional reduction in shear capacity (in addition

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to decrease in yield strength alone) due to temperature-induced instability effects. Instability effects can produce 10-20% reduction to shear capacity which occur through a different mechanism to that of temperature-dependent strength degradation [6]. Thus, total loss in shear capacity arises from two components i.e. due to strength degradation and due to instability effects.

To account for such temperature-induced web instability, an iterative procedure to evaluate slenderness of web as a function of fire exposure is proposed. This iterative procedure can track "stability" status of web during fire exposure to account for temperature-induced transformation effects. In this procedure, temperature-dependent slenderness limits ($1.10\sqrt{\frac{k_v E_T}{f_{y,T}}}$ and $1.37\sqrt{\frac{k_v E_T}{f_{y,T}}}$) are evaluated at various temperatures, ex: 100, 200, 300, …, 800°C and relative zones are plotted in Fig. 2. At each temperature, slenderness of web is compared against that of temperature-dependent slenderness limits. If (at a given target temperature) web slenderness does not exceed temperature-dependent slenderness limit, then the web is assumed to maintain its classification. For example, a compact web of slenderness of 46 remains compact (within limits of green-colored zone) in temperature range of 25-800°C and may not undergo temperature-induced instability. On the other hand, if web slenderness exceeds the compactness or non-compactness limits, appropriate reduction is applied to shear capacity to account for web buckling. This reduction is applied through a temperature-dependent web shear coefficient ($C_{v,T}$). This coefficient can be grouped under three ranges, which follow that of web slenderness limits of room temperature:

(i) $C_{v,T} = 1.0$, when $\frac{h}{t_w} \leq 1.10\sqrt{\frac{k_v E_T}{f_{y,T}}}$

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(ii) $C_{v,T} = \dfrac{1.10\sqrt{\frac{k_v E_T}{f_{y,T}}}}{h/t_w}$, when $1.10\sqrt{\frac{k_v E_T}{f_{y,T}}} \leq \frac{h}{t_w} \leq 1.37\sqrt{\frac{k_v E_T}{f_{y,T}}}$

(iii) $C_{v,T} = \dfrac{1.37\sqrt{\frac{k_v E_T}{f_{y,T}}}}{h/t_w}$, when $1.37\sqrt{\frac{k_v E_T}{f_{y,T}}} \leq \frac{h}{t_w}$

where,

k_v = is shear buckling coefficient (k_v = 5, for webs without transverse stiffeners such as those in standard hot-rolled sections).

Once $C_{v,T}$ is quantified[‡], shear capacity of a steel beam under fire conditions ($V_{n,T}$) can be evaluated as:

$$V_{n,T} = 0.6\, f_{y,T} A_w C_{v,T} \tag{7}$$

where,

$f_{y,T}$ = is yield strength of steel at temperature, T.

In order to control transformation of web and associated reduction of shear capacity, an upper boundary limit is proposed. This boundary limit is dictated through a slenderness parameter (β) which is defined as the critical slenderness ratio at which a compact (or non-compact) web transforms into a slender web. Thus, adoption of this parameter facilities conservative estimation of shear capacity under fire conditions.

To further illustrate the role of slenderness parameter (β) in tracking temperature-induced instability of steel web, compactness and non-compactness slenderness limits for a typical steel

[‡] It should be noted that the principles and rationale behind this analytical procedure are discussed in details elsewhere [11].

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grade is plotted in Fig 2. It can be seen from Fig. 2 that the lowest web slenderness at which a compact or non-compact web can transform into a slender web is 57[§]. For example, a web of slenderness of 57 is classified to be "compact" at room temperature, however, when temperature in this web reaches 700°C, the slenderness of this web ($\lambda = d/t_w = 57$) exceeds the slenderness limit ($1.37\sqrt{\frac{k_v E_{700°C}}{f_{y,700°C}}}$) which transforms it into a "slender"[**] web. In this case, the critical slenderness parameter, β, is equal to 57.

This slenderness parameter is applied into Eq. 7 to arrive at a newly derived Eq. 8 as follows:

$\beta = d/t_w \rightarrow d = \beta \times t_w$ and since $A_w = d \times t_w$, this leads to $A_w = (\beta\, t_w)\, t_w$

Upon substitution of $A_w = (\beta\, t_w)\, t_w$, the simplified expression to calculate shear capacity turns out to be:

$$V_{n,T} = \begin{cases} 0.6 f_y\, A_w\, C_v, & T < 150°C \\ 0.6 f_{y,T}\, \beta t_w{}^2 C_{v,T}, & T \geq 150°C \end{cases} \quad (8)$$

The above derived equations are applicable when temperature in steel beam is in the range of 25-800°C which is the most critical range for capacity degradation in beams under fire conditions. Since yield strength and modulus of elasticity of steel do not degrade at temperatures between 25-150°C, no temperature-induced instability occurs till 150°C. Thus, in this temperature range, shear capacity of a steel beam can be obtained from AISC design provisions/published tables. However, in cases where temperature of steel beam exceeds 150°C, shear capacity can be

[§] For illustrative purposes, $\beta = 57$ is applicable for steel grade of 345 MPa. Same procedure can be used to arrive at boundary factor β of other steel grades.
[**] Same procedure can be used to limit shear capacity of web so that it cannot exceed that of compactness limit.

evaluated through the proposed approach. This shear capacity is compared against shear force due to applied loading (V_f). Thus, failure due to shear effects occurs once shear capacity ($V_{n,T}$) falls below level of applied loading (V_f).

Figure 3 presents a flow chart that outlines various steps needed to evaluate shear capacity of steel beams taking into account the effect of temperature-dependent strength properties as well as temperature-induced web instability. These calculation steps can be implemented into a simple spreadsheet to be performed at target temperature (or time-step). It should be noted that for comprehensive analysis of failure in a given beam, other failure limit states such as flexural and deflection limit (along with shear) need to be checked as well.

4.3 Shear capacity of composite beams under ambient conditions

The composite interaction between steel beam and concrete slab has numerous advantages, and much of this is taken into account in flexural calculations at ambient and fire conditions. For instance, contribution of concrete slab to flexural capacity has been thoroughly investigated in various studies [7-9] and its effect is accounted for in strength equations given in current design codes [16-19]. It is worth mentioning that although a number of recent studies [12-15] have pointed out that composite action developed at beam-slab interface can increase available shear capacity of a composite beam (see Fig. 4), this contribution of concrete slab to shear capacity (at ambient and fire conditions) continues to be neglected in current design provisions.

The positive contribution of concrete slab to shear capacity at ambient conditions has been well documented in various experimental studies [13-15]. For example, Vasdravellis and Uy [13], Nie et al. [14] and Liang et al. [15] tested a large number of composite beams under combined effects of flexural and shear loading. Results from these tests indicate that composite beams have

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a much higher shear capacity (of about 10-40%) than that of isolated steel beams. Further, these researchers have shown that total shear capacity of a composite beam is equal to the algebraic sum of shear capacity contributed by steel beam and concrete slab (see Table 1). Despite these findings, current design guidelines still consider contribution of concrete slab to shear capacity to be insignificant and evaluates shear capacity of a composite beam as the shear capacity of steel beam alone.

4.4 Shear capacity of composite beams under fire conditions

Under fire conditions, concrete slab acts as a heat sink due to the high specific heat and low thermal conductivity of concrete. This effect attracts much of heat from top portion of steel beam, hence lowering temperature rise in steel. As a result, rate of degradation in strength and elastic modulus properties of steel beam slows down which delays failure of composite beam under fire conditions [23, 24]. Further, temperature in concrete remains low for a longer fire exposure and thus much of concrete area retaining its room temperature strength and stiffness properties. Thus, these two effects along with positive contribution from slab to shear capacity as discussed in Sec. 4.3, need to be taken into account in order to accurately evaluate shear capacity in a composite beam. In this study, the overall shear capacity of a composite beam (V_n) is taken as the contribution of shear capacity of steel beam (V_{beam}) and contribution shear capacity of concrete slab (V_{slab});

$$V_n = V_{beam} + V_{slab} \tag{9}$$

Previous studies, carried out at ambient and fire conditions [11-15], infer that contribution of concrete slab to total shear capacity is a function of tensile strength of concrete, effective area of concrete subjected to shear (which is a function of slab thickness (D_{slab})), level of composite

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action (C), and stiffness of shear studs (K). Thus, from the analysis on reported tests and numerical analyses, this contribution of concrete slab to shear capacity (V_{slab}) is given by:

$$V_{slab} = 0.16\sqrt{f_c'}\, b_f + 0.95 D_{slab} + 89C - 4.7K - 179 \tag{10}$$

where, $\sqrt{f_c'}$ is a measure of tensile strength of concrete [27] and b_f is the width of the top steel flange.

This equation was arrived at through genetic algorithm regression procedure which has been shown to be superior to conventional regression analysis as documented by recent studies [11, 25, 26]. The proposed equation (Eq. 10) has a coefficient of determination (R^2) of 0.85 as shown in Fig. 5. This equation accounts for level of composite action and stiffness of shear studs and is also applicable to both ambient and fire conditions; unlike other equations listed in Table 1. Figures 5a and 5b compare predictions from the proposed Eq. 10 with other previous researchers. The trend in the figures clearly show that the proposed equation predict contribution of concrete slab to shear capacity with good accuracy.

In order to evaluate shear capacity of a composite beam at a given temperature in steel beams ($V_{n,T}$), contribution of concrete slab (V_{slab}) is to be added to contribution of steel beam (V_{beam}) such that:

$$V_{n,T} = V_{beam} + V_{slab}$$

$$V_{n,T} = \begin{cases} 0.6 f_y\, A_w\, C_v + 0.16\sqrt{f_c'}\, b_f + 0.95\, D_{slab} + 89C - 4.7K - 179, & T < 150°\text{C} \\ 0.6 f_{y,T}\, \beta t_w{}^2 C_{v,T} + 0.16\sqrt{f_{c,T}'}\, b_f + 0.95\, D_{slab} + 89C - 4.7K - 179, & T \geq 150°\text{C} \end{cases} \tag{11}$$

Similar to Eq. 8, Eq. 11 is also applicable when temperature in composite beam is in the range of 25-800°C. Up to 150°C, shear capacity of composite beams is assumed to be equal to the sum of shear capacity of steel beam (obtained from AISC design provisions/published tables) and

shear capacity of concrete slab (calculated using Eq. 10). In cases where temperature of composite beams rises to more than 150°C, shear capacity of composite beam is evaluated as the sum of shear capacity contribution of steel beam (calculated using proposed Eq. 8) and concrete slab (calculated using Eq. 10). Thus, failure due to shear effects occurs once shear capacity ($V_{n,T}$) falls below level of applied loading (V_f).

A flow chart outlining various steps needed to evaluate shear capacity of composite beams taking into account effect of temperature-induced strength degradation and web instability effects, together with contribution of concrete slab is illustrated in Fig. 6. Thus, for a comprehensive analysis of analyzed beam, other failure limit states such as flexural and deflection limits (along with shear) need to be checked as well.

5.0 VALIDATION OF THE PROPOSED APPROACH

The above proposed approach for evaluating shear capacity under fire conditions was validated against predictions from finite element analysis and AISC ambient design equations (extended to fire design). In the case of composite beams, the proposed approach was also validated against predictions obtained from derived expressions by other researchers [13-15].

A finite element based numerical model was developed in ANSYS 14.0 to generate results for validation [11]. 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, end boundary conditions, temperature-induced thermal expansion and various failure limit states. For undertaking fire resistance analysis, thermal and mechanical properties of structural steel, and concrete are assumed to vary with temperature as per Eurocode 3 and 2 relations [17, 28].

In order to simulate fire response of steel and composite beams, two stages of analysis are to be carried out. In the first stage, selected beams are subjected to the ASTM E119 standard fire scenario and thermal analysis is carried out at incremental time steps to generate sectional temperatures (and thermal gradients). In the second stage of analysis, both temperature and loading are applied, simultaneously, as part of structural analysis. Full details on the development of this finite element model (i.e. discretization, failure limit states etc.) are not presented herein for brevity but can be found elsewhere [2, 11].

In addition to comparing predictions from the proposed approach with that obtained from finite element analysis, predictions from the proposed approach were also compared against those evaluated form AISC modified design equation. AISC room temperature equations (specifically Eq. 6) was extended to fire conditions by introducing temperature-dependent yield strength reduction into the equation. The degradation of yield strength of steel can be accounted for through temperature-dependent strength degradation factors specified in current codes and standards [16, 17] as shown in Sec. 4.2. In order to ensure consistent comparison throughout this study, temperature rise in steel beam and concrete slab are obtained through finite element analysis[††].

It should be noted that the beams selected for validation assessment are of Grade 345 (MPa) steel and are subjected to high level of shear loading applied near interior supports (to maximum shear effect and minimize bending moment) while being simultaneously exposed to ASTM E119 standard fire (see Fig. 7). This shear loading is equivalent to 40% of shear capacity and 5% of

[††] Temperature rise in steel beams can be also evaluated using simplified methods such as those proposed by Dwaikat and Kodur [29]

moment capacity at room temperature [11, 30]. As a result, failure of these beams occurred in shear mode, i.e. when shear capacity at a certain temperature when shear capacity drops below applied shear loading. It should be noted that mode details on model development and finite element analysis procedure are not provided here for brevity but can be found elsewhere [31, 32].

5.1 Steel beams

To validate the proposed approach, four steel beams made of hot-rolled sections, namely W16×26, W18×40, W24×55, and W40×167 were selected for analysis (see Table 2). As discussed earlier, in derivation of Eq. 8, shear capacity of steel beams at room temperature (and up to 150°C) is assumed to be similar to that obtained from AISC design equations. Hence, the proposed approach is validated for steel beams subjected to temperatures higher than 150°C.

Figure 8 shows variation of shear capacity with web temperature of selected beams as obtained from the proposed approach, finite element analysis and AISC extended design equations. Analysis of these plots infer that predictions from proposed approach is in close agreement with those obtained from finite element analysis and are more conservative than predictions from AISC design equations. AISC extended design equations only accounts for temperature-related strength degradation, but not temperature-induced instability effects. It should be noted that the proposed approach as well as finite element model can trace effect of temperature-dependent strength degradation and temperature-induced instability. From response plots shown in Fig. 8, it is can be inferred that temperature-induced instability effects can lead to 10-20% reduction in shear capacity and is apparent especially at temperatures higher than 150°C.

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The temperature at which failure occur in beams (where failure occurs when shear capacity drops below an assumed level of loading of 40% of shear capacity) is shown in Table 2 and the temperature at failure is significantly different when predictions from the proposed approach (T_{PA}) and finite element analysis (T_{FE}) are compared against that of AISC (T_{AISC}). Since tested beams are made of same steel grade and are exposed to same loading and fire conditions, temperature in these beams rises at the same rate. Thus, degradation in strength properties of steel occur at the same pace in all tested beams i.e. when AISC rationale is applied, shear capacity in all beams drops below the assumed level of loading at the same temperature which corresponds to 640°C[‡‡]. However, actual failure of tested beams occur at temperature lower than that predicted by AISC extended equation due to combined effect of temperature-induced instability along with strength degradation as captured by the proposed approach (and finite element analysis). Since temperature-induced instability can initiate at early stages of fire (around 150°C), instability effects can cause early loss in shear capacity.

5.2 Composite beams

In order to validate the proposed approach in fire design of composite beams, four composite beams of various steel sections of W16×26, W18×40, W24×55, and W40×167 and concrete slab configurations are analyzed as a function of fire exposure time (ASTM E119). The applicability of the proposed approach in evaluating shear capacity of composite beams is also validated and compared against predictions from AISC extended design equations, finite element analysis, as well as predictions from expressions derived by others [13-15]. Further, to study a

[‡‡] Please note that the time to reach this temperature (of 640°C) will vary in each beams based on the mass and geometric dimensions of the steel section.

wide range of geometric and material parameters, selected composite beams were tested by varying concrete compressive strength, thickness of slab, stiffness of shear studs and level of composite action as shown in Table 3.

It should be noted that derived equations in [13-15] are based on room temperature tests and to maintain applicability of these expressions, an assumption that average temperature rise in concrete slab to remain relatively moderate under fire conditions is specified. This assumption has been verified by a number of experimental and numerical studies which indicate that temperature rise in much of concrete slab can be in the range of 150-350°C (till about 1 to 2 hours of fire exposure), thus the concrete slab can be safely assumed to maintain its room temperature design strength (under similar fire conditions)[§§] [8, 9, 11, 30].

A comparison of shear capacity-web temperature variation in composite beams obtained from the proposed approach, finite element analysis as well as design expressions developed by Vasdravellis and Uy [13], Nie et al. [14] and Liang et al. [15] is plotted in Fig. 9. It can be seen from this figure that predicted shear capacity from proposed approach (T_{PA}), finite element analysis (T_{FE}) and expressions derived by other researchers ($T_{Nie\ et\ al.}$, $T_{Liang\ et\ al.}$ and $T_{Vasdravellis\ and\ Uy}$) is higher than that predicted following AISC design provisions. This is because AISC design provisions do not account for contribution of concrete slab and assume shear capacity of a composite beam to be only carried by the web of steel beam.

Figure 9 also shows that in some cases, predictions from derived expressions by Vasdravellis and Uy [13], Nie et al. [14] and Liang et al. [15] can be much higher than that

[§§] For extreme fires, appropriate reduction to concrete strength need to be appropriately accounted for as shown in derived Eq. 10.

predicted by the proposed approach. This is mainly due to the fact that these derived expressions by other researchers do not account for variation in level of composite action, shear stud stiffness or instability factors which were varied as shown in Table 3. In all cases, it is clear that presence of concrete slab can counterbalance, to some extent, the adverse effects of local buckling which usually occurs at temperatures between 150-200°C. In this temperature range, reduction to shear capacity occurs due to buckling of web, however, strength contribution of concrete slab can, in most cases, provide sufficient strength to enhance shear capacity in a composite beam (as compared to an isolated steel beam).

Moreover, analysis of response plots shown in Fig. 9 infer that concrete slab can increase total shear capacity in tested steel beams by 10-20% at ambient conditions which agrees well with findings of other researchers [13-15]. Under fire conditions, beyond steel temperature range of 150-400°C, concrete slab can enhance shear capacity of isolated steel beams by 20-40%. This enhanced contribution to shear capacity from concrete slab is mainly due to slower degradation in strength (tensile strength) of concrete. These results infer that presence of concrete slab can be much more beneficial under fire conditions than that at room temperature.

6.0 DESIGN APPLICATIONS

The proposed approach can be applied to evaluate shear capacity of steel and composite beams under any given fire scenario. The applicability of the proposed approach in evaluating shear capacity of steel and composite beams is illustrated for a typical steel beam and a composite beam subjected to combined shear loading and ASTM E119 fire. These beams are assumed to have sufficient flexural capacity and failure under fire conditions in these beams occur through shear mode. It should be noted that temperature rise in these beams are obtained from the finite

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element analysis described above. Alternatively, temperature rise in steel can be evaluated from simplified temperature prediction equations available in literature [29]. Full details on analysis and calculation procedure are shown in Appendix A.

6.1 Isolated Steel Beam

The proposed approach is applied to evaluate shear capacity of a steel beam of W18×40 section and made of A992 steel at ambient and fire conditions. The configuration of the beam and loading set-up is similar to that shown in Fig. 7. The shear capacity of this beam, at ambient conditions, can be obtained using AISC provisions and was found to be 752 kN. The shear capacity of this beam can also be evaluated under fire conditions (at 600°C) using AISC rationale (when extended to fire conditions) as well as proposed approach as shown in Sec. 4.1. The predicted shear capacity at 600°C using AISC extended design equations and proposed approach is 353.5 and 301.4 kN, respectively.

Figure 10 illustrates the variation in predicted shear capacity from the AISC extended design equations as well as proposed approach. This figure clearly shows that unlike predictions from proposed approach, AISC extended equation does not account for occurrence of temperature-induced instability effects and associated reduction in shear capacity (at 150-200°C). Hence, if an assumed level of loading (of 50% of shear capacity) is to be applied to the beam, this beam fails at temperature of 574°C (as compared to 590°C from predictions of AISC provisions).

6.2 Composite Steel Beam

To further illustrate the applicability of the proposed approach for analysis of composite beams, a composite beam comprising of a W-shape (W18×40) steel section made of A992 steel

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and in situ with a 140 mm thick reinforced concrete slab is selected for the analysis. This beam develops full composite action between steel beam and concrete slab, since sufficient number of stiff shear studs are provided.

Following AISC provisions, which neglects contribution of concrete slab, shear capacity of this composite beam, at ambient conditions is similar to that of an isolated steel beam which equals to 752 kN (as shown in Sec. 6.1). However, the proposed approach estimates shear capacity to be 924 kN, which includes the contribution of concrete slab, and is 23% higher than that predicted by current AISC provisions.

Shear capacity of this composite beam is also evaluated at various temperatures and plotted in Fig. 11. This figure shows that when steel temperature reaches 600°C (in web), shear capacity evaluated using AISC extended design equations equals to that shown in Sec. 6.1 (V_n = 353.5 kN). The shear capacity of this composite beam can be also obtained by applying proposed approach in Eq. 11 and is evaluated as 473.4 kN (34% larger than that predicted using AISC provisions). Although steel beam undergoes some level of local buckling, which explains the slight drop in shear capacity around 150°C (shown in Fig. 11), the proposed approach still predicts higher shear capacity (due to the contribution of concrete slab). Thus, if an assumed level of loading (of 50% of shear capacity) is to be applied to this assembly, the composite beam fails at temperature of 685°C (as compared to 595°C, from AISC predictions). It is clear that contribution of concrete slab to shear capacity can be very beneficial under fire conditions.

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7.0 LIMITATIONS OF PROPOSED APPROACH

Due to the large variability in available cross-sectional configurations of steel sections, the proposed approach is specifically derived for hot-rolled standard steel beams with W-shaped sections. However, the same methodology can be applied for M, S and built-up sections (plate girders) as well as non-standard (built-up) sections if special attention is applied to the derivation of the slenderness parameter (β) and temperature-dependent web shear buckling ($C_{v,T}$). Although the proposed approach provides a comprehensive and standardized methodology for evaluating shear capacity, the following are the range of applicability under which the proposed approach may not be fully applicable:

- The validity of the proposed approach is verified for steel temperatures between 25-800°C.
- The proposed approach is validated for composite beams with normal strength concrete. Further validation is needed to ensure the applicability of this approach for composite beams made of special concrete mixes including fiber-reinforced concrete (FRC), high strength concrete (HSC) and ultra-high performance concrete (UHPC).
- The proposed approach does not account for uncertainty factors such as fire-induced spalling of concrete.

8.0 CONCLUSIONS

Based on the information presented in this paper, the following key conclusions can be drawn:

1. Current design philosophy of evaluating failure of fire exposed steel and composite beams based on flexural strength limit state alone may not be conservative in certain situations where steel and composite beams with slender webs, are subjected to high

shear forces. In these scenarios, shear parameters can dominate fire response and failure can occur under shear limit state, prior to attaining flexural limit state.

2. The degradation of shear capacity in steel and composite beams under fire conditions results not only from the effects of temperature-induced strength loss in steel, but also from temperature-induced sectional instability in web.
3. Onset of local buckling in web can produce about 10-20% degradation in shear capacity in steel beam, and this can lead to early failure under shear limiting capacity, prior to failure under flexural limit state. The effect of local buckling can be accounted for through two new parameters proposed in this study, namely temperature-dependent web shear coefficient and slenderness parameter.
4. Composite action arising from concrete slab, not only enhances sectional (shear) capacity, but can also minimize temperature-induced web local buckling on shear capacity (sectional instability) and thus enhance shear capacity of composite beams under fire conditions.
5. Presence of concrete slab can significantly enhance shear capacity of composite beams under fire conditions more than that at room temperature. Contribution of slab to shear capacity at ambient and fire conditions can be in the range of 10-20% and 20-40%, respectively.

9.0 ACKNOWLEDGMENT

This material is based upon the work supported by the National Science Foundation under Grant number CMMI-1068621 and Michigan State University. Any opinions, findings, and

conclusions or recommendations expressed in this paper are those of the authors and do not necessarily reflect the views of the sponsors.

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This is a preprint draft. The published article can be found at: https://doi.org/10.1016/j.jcsr.2017.11.011

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APPENDIX

The application of the proposed approach for analysis of steel and composite beams is illustrated in two numerical examples of a typical isolated steel beam and a composite beam. These beams are assumed to be subjected to high shear loading and ASTM E119 standard fire exposure.

A.1 Design Example – I (Isolated Steel Beam)

The proposed approach is applied to evaluate shear capacity of a steel beam made of A992 steel and of W18×40 section at ambient and fire conditions (at an elevated temperature of 600°C).

The properties of steel beam are;

$d=454.6$ mm, $t_w = 8$ mm, $f_y = 345$ MPa, $f_{y,600°C} = 162$ MPa

At ambient conditions

The shear capacity at room temperature can be evaluated using the AISC provisions (Equation G2-1)

$$V_n = 0.6 f_y A_w C_v$$

and since,

$$\lambda = 50.9 < 2.24\sqrt{\frac{E}{f_y}} = 53.9$$

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then,

$$C_v = 1.0$$

hence,

$$V_n = 0.6 f_y \, A_w C_v$$

$$V_n = 0.6 \times 345 \times 454.6 \times 8 \times 1.0 = 752 \, kN$$

Under fire conditions (600°C)

The shear capacity of this composite beam can be evaluated under fire conditions, following flowchart presented in Fig. 3, using the AISC extended equation (when extended to fire conditions) as well as the proposed approach. Predictions from both approaches are compared and discussed below.

<u>AISC provisions (when extended to elevated temperatures)</u>

$$V_{n,AISC} = 0.6 f_{y,600°C} \, A_w C_v$$

$$V_{n,AISC} = 0.6 \times 162 \times 454.6 \times 8 \times 1.0 = 353.5 \, kN$$

<u>Proposed approach</u>

The shear capacity of this composite beam can be evaluated using Eq. 8,

since,

$\lambda = 50.9$, slenderness ratio will exceed compactness limit at 600°C (see Fig. 2)

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then,

$$\beta = 57 \; and \; C_{v,T} = 0.85$$

thus,

$$V_{n,proposed\ approach} = 0.6 f_{y,600°C}\, \beta\, t_w{}^2 C_{v,T}$$

$$V_{n,proposed\ approach} = 0.6 \times 162 \times 57 \times 8^2 \times 0.85 = 301.4\ kN$$

A.2 Design Example – II (Composite Steel Beam)

The proposed approach is illustrated to evaluate shear capacity of the composite beam at ambient conditions and fire conditions.

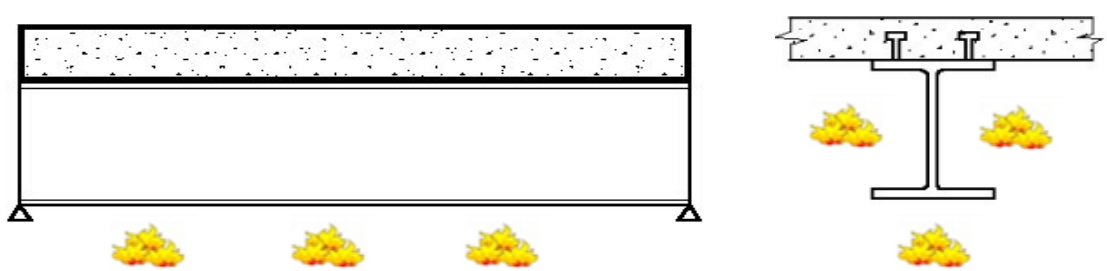
The properties of steel beam and concrete slab are;

Steel beam:

b_f = 153 mm, d=454.6 mm, t_w = 8 mm, f_y = 345 MPa, $f_{y,600°C}$ = 162 MPa

Concrete slab:

f_c' = 30 MPa, D_{slab} = 140 mm, Stiff shear studs; K = 1, full composite action; C = 100%, b = 850 mm



At ambient conditions

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AISC provisions

The shear capacity of this composite beam can be evaluated using the AISC Eq. G2-1;

$$V_n = 0.6 f_y\ A_w C_v$$

$$V_n = 0.6\ \times 345 \times 454.6 \times 8 \times 1.0 = 752\ kN$$

Proposed approach

The shear capacity of this composite beam can be evaluated at room using Eq. 11,

$$V_n = 0.6 f_y\ A_w C_v + \left\{0.16\sqrt{f'_{c,20°C}}\ b_f + 0.95\ D_{slab}\ +\ 89C - 4.7K - 179\right\}$$

$$V_n = 752 + \left\{0.16\sqrt{30}\ (153) + 0.95(140)\ +\ 89(1) - 4.7(1) - 179\right\}$$

$$V_n = 752 + 172 = 924\ kN$$

Under fire conditions (600°C)

The shear capacity of this composite beam can be evaluated under fire conditions, following flowchart presented in Fig. 6, using the AISC extended equation (when extended to fire conditions) as well as the proposed approach. The temperature in the steel beam was assumed to be 600°C.

AISC provisions (when extended to elevated temperatures)

$$V_{n,AISC} = 0.6 f_{y,600°C}\ A_w C_v$$

$$V_{n,AISC} = 0.6\ \times 162 \times 454.6 \times 8 \times 1.0 = 353.5\ kN$$

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Proposed approach

For fire calculation, results from experimental and numerical studies indicate that temperature rise in concrete slab can remain low for typical fire conditions (in the range of 150-300°C) [8, 9, 11, 30], thus the strength contribution of concrete slab can be safely assumed to remain the same even under fire conditions. Hence, the shear capacity of this composite beam under fire conditions (at 600°C) is:

$$V_n = 0.6 f_{y,600°C}\, \beta\, t_w{}^2 C_{v,T} + \left\{0.16\sqrt{f'_{c,20°C}}\, b_f + 0.95\, D_{slab} + 89C - 4.7K - 179\right\}$$

$$V_n = 0.6 \times 162 \times 57 \times 8^2 \times 0.85 + \left\{0.16\sqrt{30}\,(153) + 0.95(140) + 89(1) - 4.7(1) - 179\right\}$$

$$V_n = 301.4 + 172 = 473.4\ kN$$

This is a preprint draft. The published article can be found at: https://doi.org/10.1016/j.jcsr.2017.11.011

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Table 1 Shear capacity of composite beams as derived by Vasdravellis and Uy [13], Nie et al. [14] and Liang et al. [15]

Derived expression	V_n	V_{slab}
Vasdravellis and Uy [13]	$V_n = V_{beam} + V_{slab}$	$V_{slab} = \varphi_s f(\lambda_{sd})(b_f D_{slab})^{0.7}\sqrt{f_c'}$ $f(\lambda_{sd}) = 110\lambda_{sd} + 13$ where, φ_s = a safety factor (= 0.8) λ_{sd}= slab depth slenderness ratio (Depth of slab/Depth of beam)
Nie et al. [14]	$V_n = V_{beam} + V_{slab}$	$V_{slab} = 0.8\sqrt{f_c'}b_f h_f/\lambda$ where, λ = shear span aspect ratio of composite beam h_f= thickness of concrete flange
In Liang et al. [15]	$V_n = V_{beam} + V_{slab}$	$V_{slab} = 1.16\,(f_c')^{1/3}A_{ec}$ where, A_{ec} = effective shear area of concrete ($A_{ec} = (b_f + D_{slab})D_{slab}$)

where, $V_{beam} = V_{AISC}$

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Table 2 Failure temperature in steel beams from different approaches

Steel section	d/t_w	T_{PA} (°C)	T_{AISC} (°C)	T_{FE} (°C)
W16×26	56.8	592	640	595
W18×40	50.9	613	640	617
W24×55	54.9	600	640	604
W40×167	52.6	603	640	610

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Table 3 Geometric and material properties of composite beams used in the validation of the proposed approach

Steel section	d/t_w	D_{slab} *(mm)*	f'_c *(MPa)*	*K*	*C (%)*	T_{PA} *(°C)*	T_{AISC} *(°C)*	T_{FE} *(°C)*	$T_{Nie\ et\ al.}$ *(°C)*	$T_{Liang\ et\ al.}$ *(°C)*	$T_{Vasdravellis\ and\ Uy}$ *(°C)*
W16×26	56.8	140	25	0.5	75	760	640	790	720	-	800
W18×40	50.9	125	40	1	50	710	640	725	680	-	700
W24×55	54.9	150	45	0.75	100	715	640	720	645	-	670
W40×167	52.6	250	60	1	30	680	640	700	620	-	665

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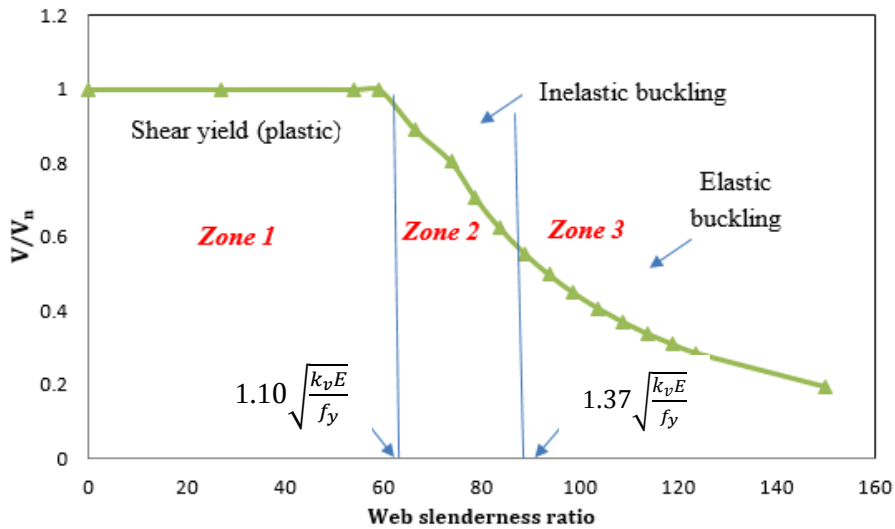
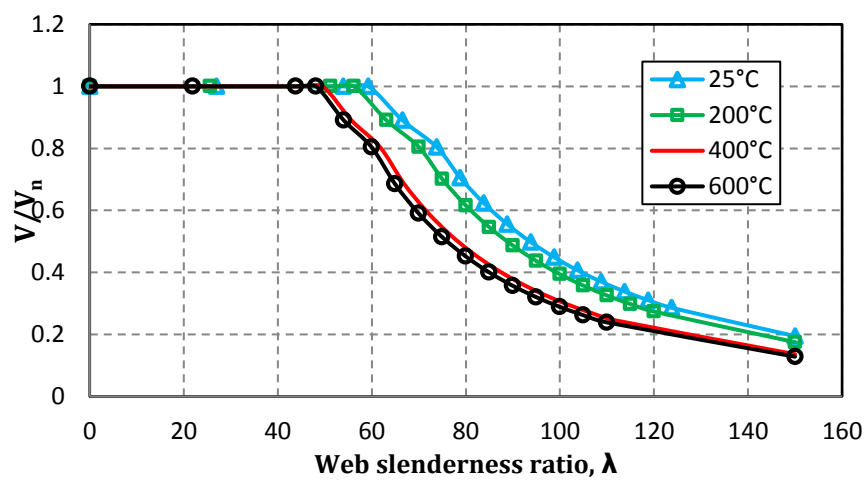


(a) Room temperature



(b) under fire conditions

Fig. 1 Classification of web in fire-exposed beams subjected to shear loading

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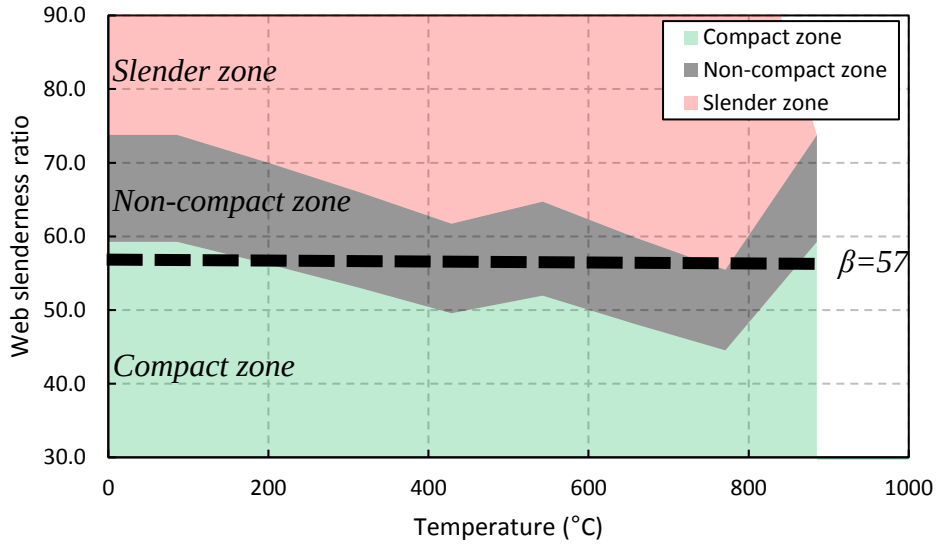

Fig. 2 Illustration of variation of slenderness parameter (β) with temperature

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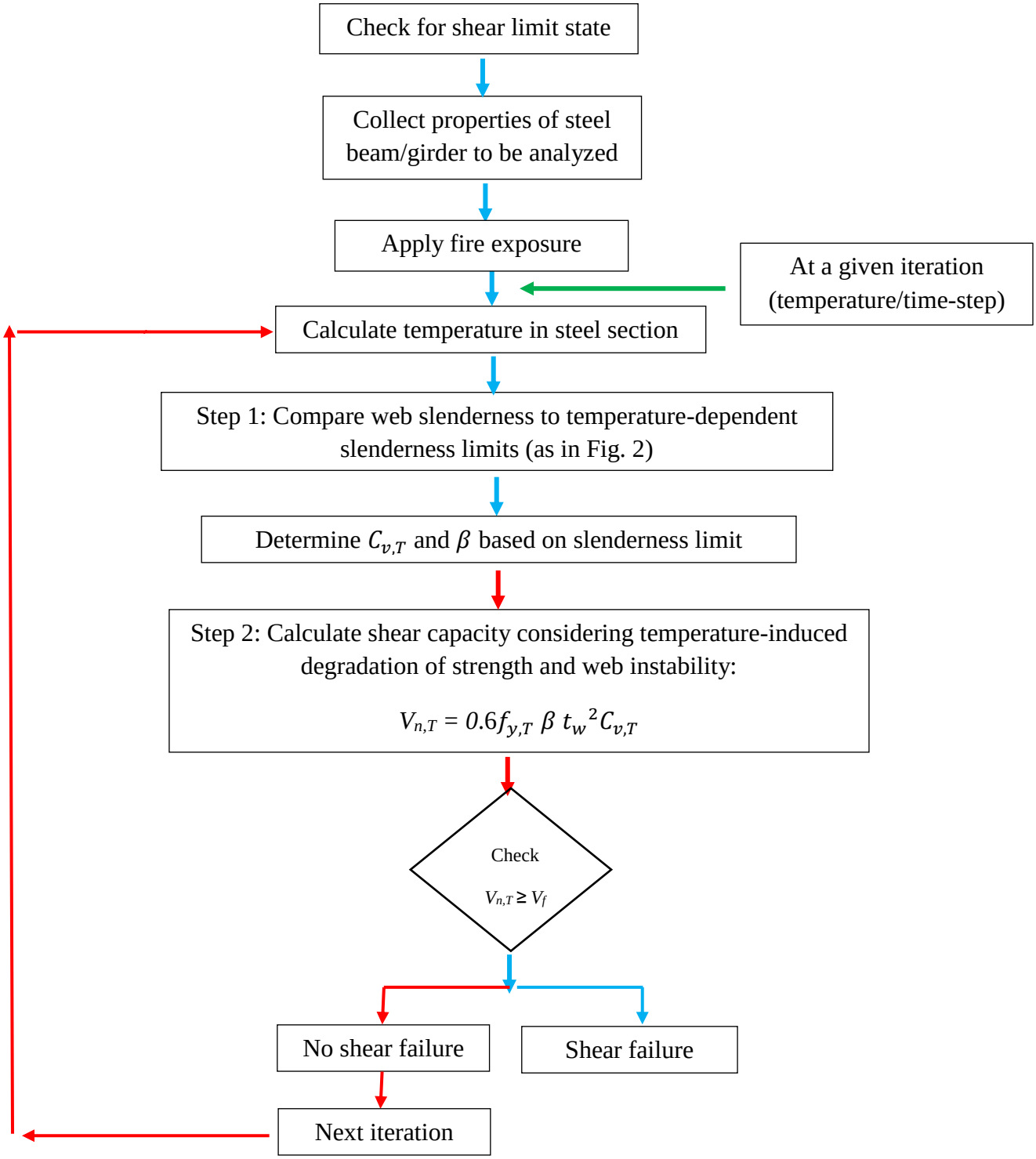

Fig. 3 Flow chart illustrating steps for evaluating shear capacity and failure of steel beam under fire conditions

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(i) Strain (ii) Shear stress

(a) No interaction

(i) Strain (ii) Shear stress

(b) Partial interaction

(i) Strain (ii) Shear stress

(c) Full interaction

Fig. 4 Strain and shear stress distibutionin of a composite beam

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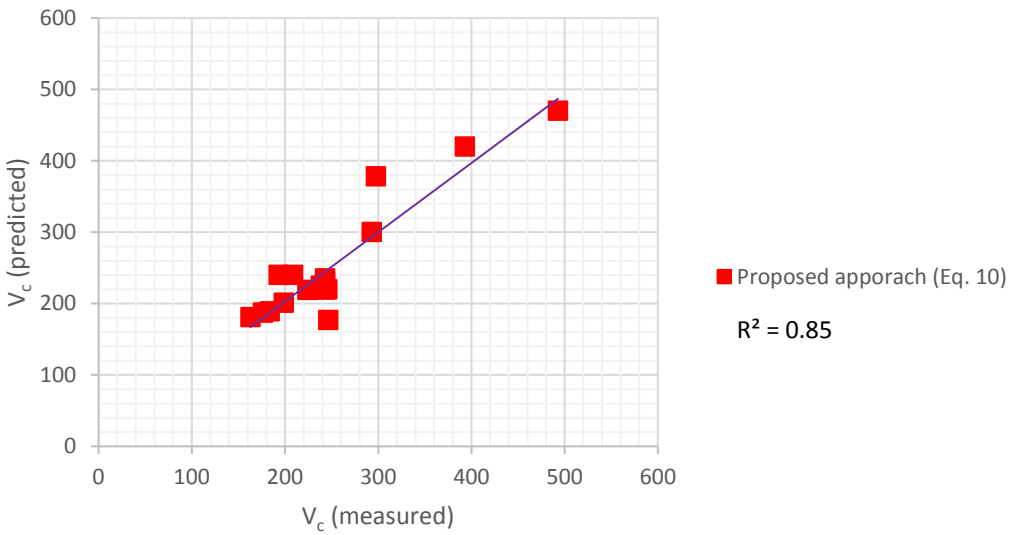
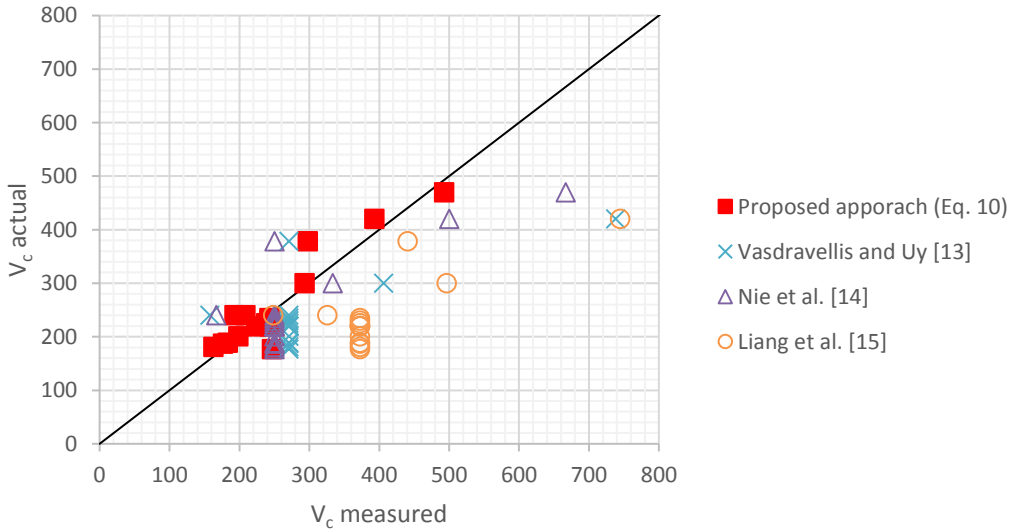


(a) Fitness of Eq. 10



(b) Comparison with other equations

Fig. 5 Validity of proposed equation in estimating shear capacity

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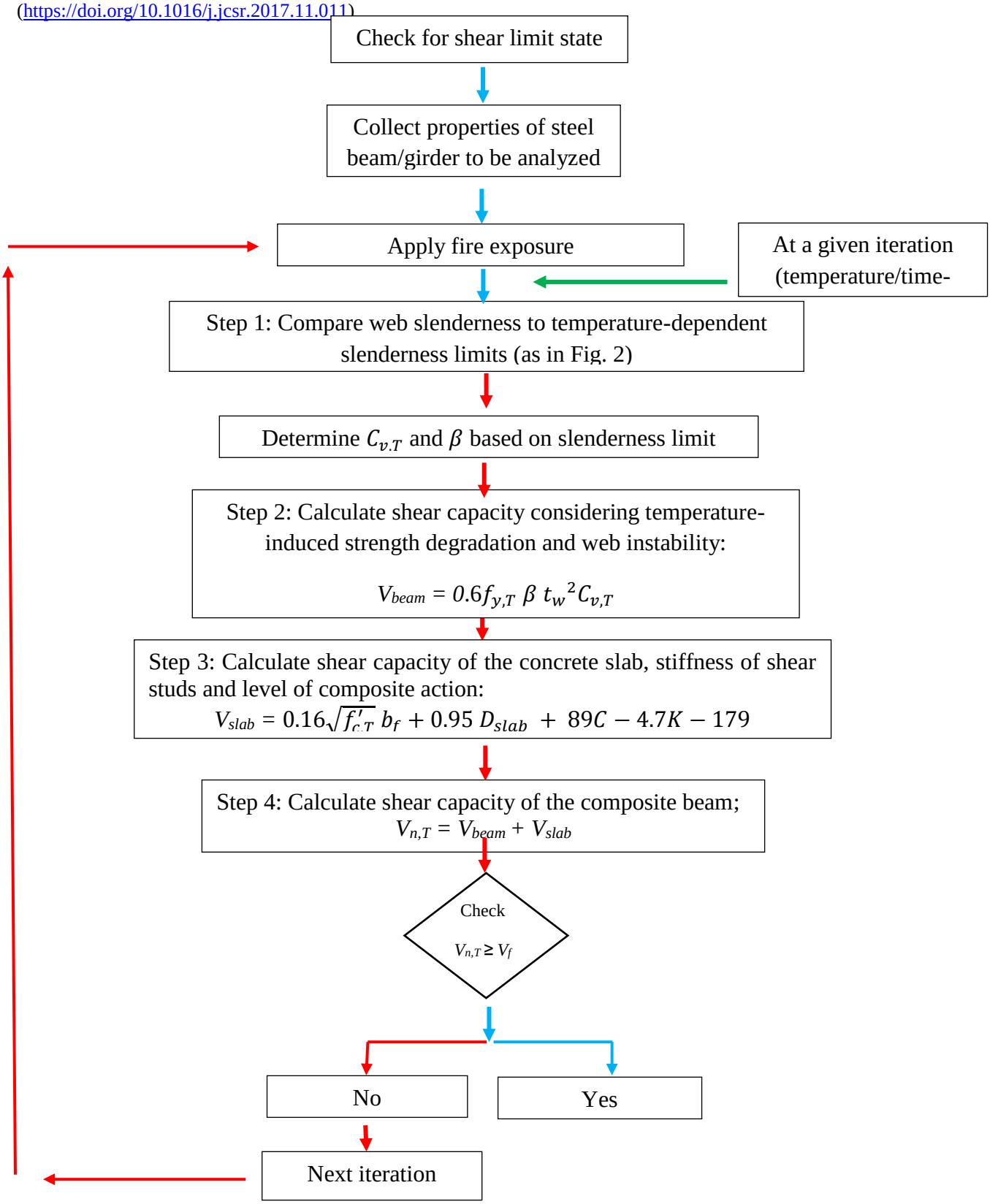

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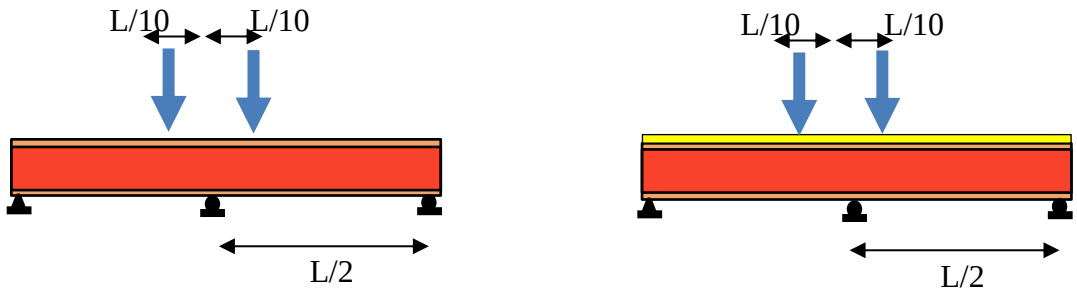
Fig. 6 Flow chart for proposed methodology of calculating shear capacity and failure of composite beam under fire conditions



(a) Steel beam (b) Composite beam

Fig. 7 Loading set-up and boundary conditions used in validation of proposed approach

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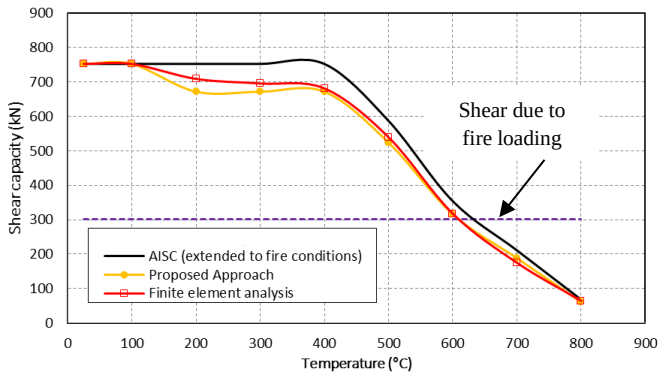
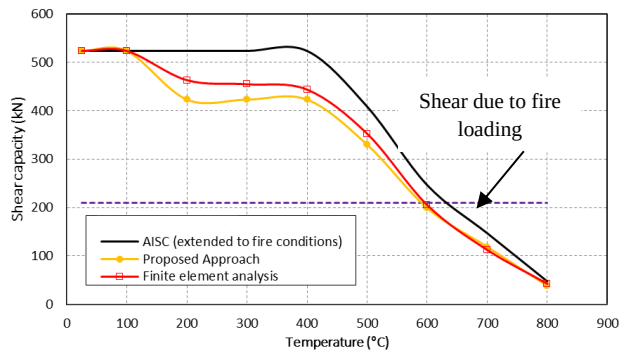
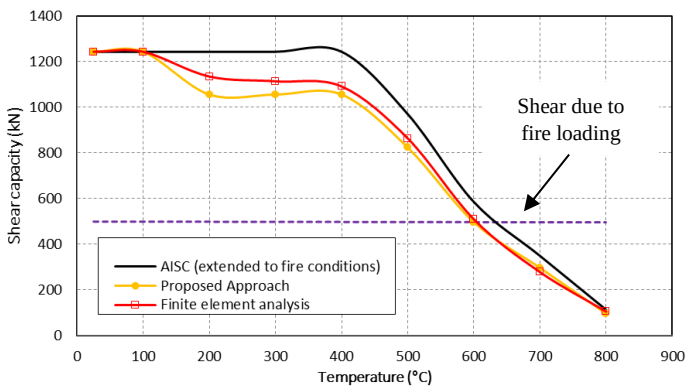


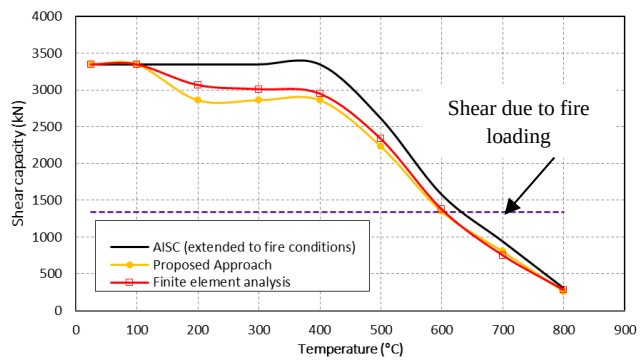
(a) W16×26



(b) W18×40



(c) W24×55



(d) W40×167

Fig. 8 Comparison between predicted shear capacity of hot-rolled sections using proposed and AISC approaches

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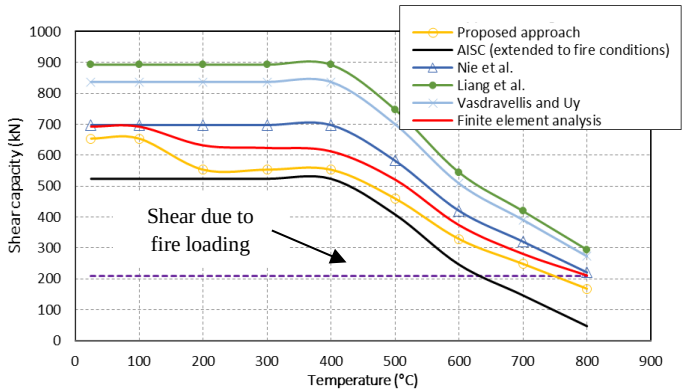
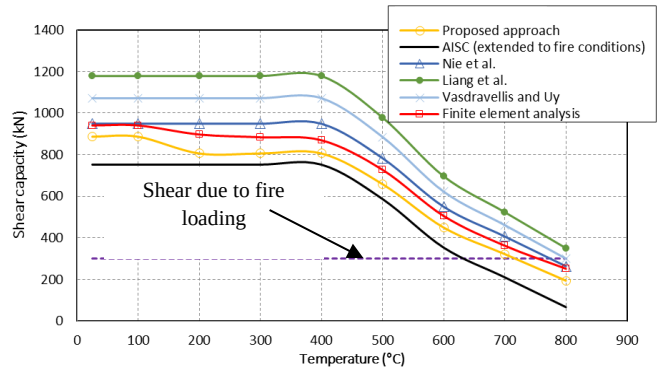
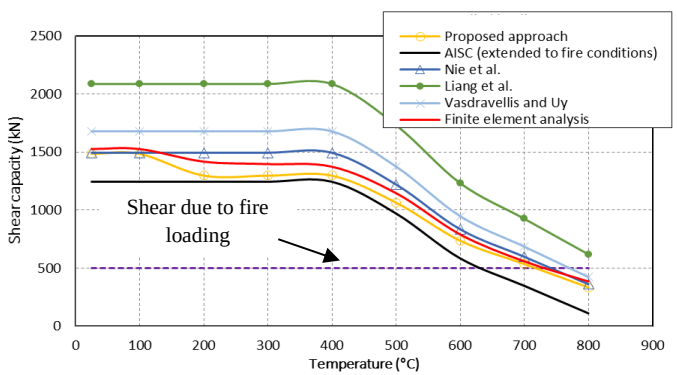


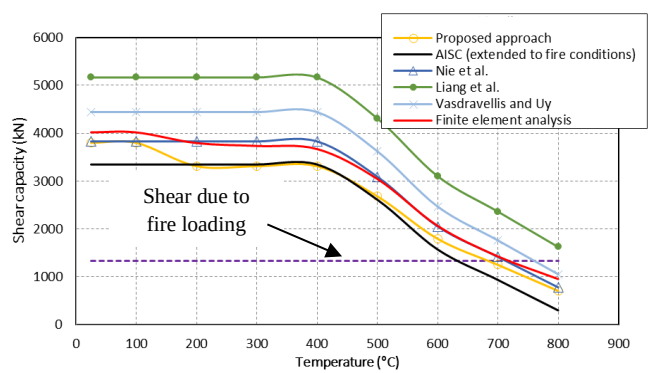
(a) W16×26



(b) W18×40



(c) W24×55



(d) W40×167

Fig. 9 Comparison between proposed approach and current design provisions

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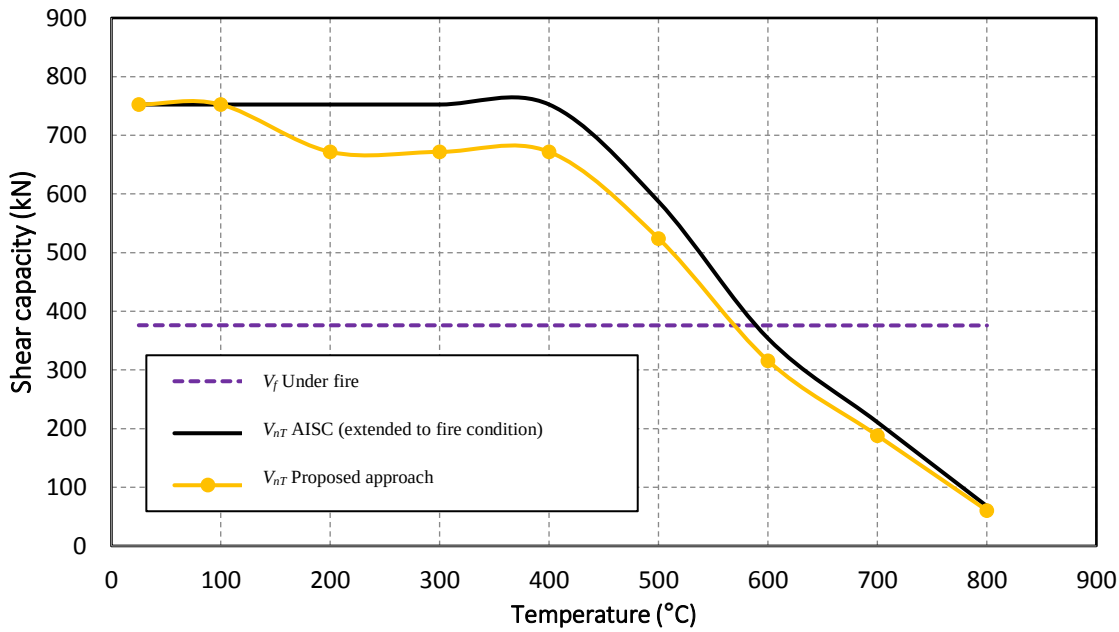

Fig. 10 Degradation of shear capacity of composite beam under fire exposure

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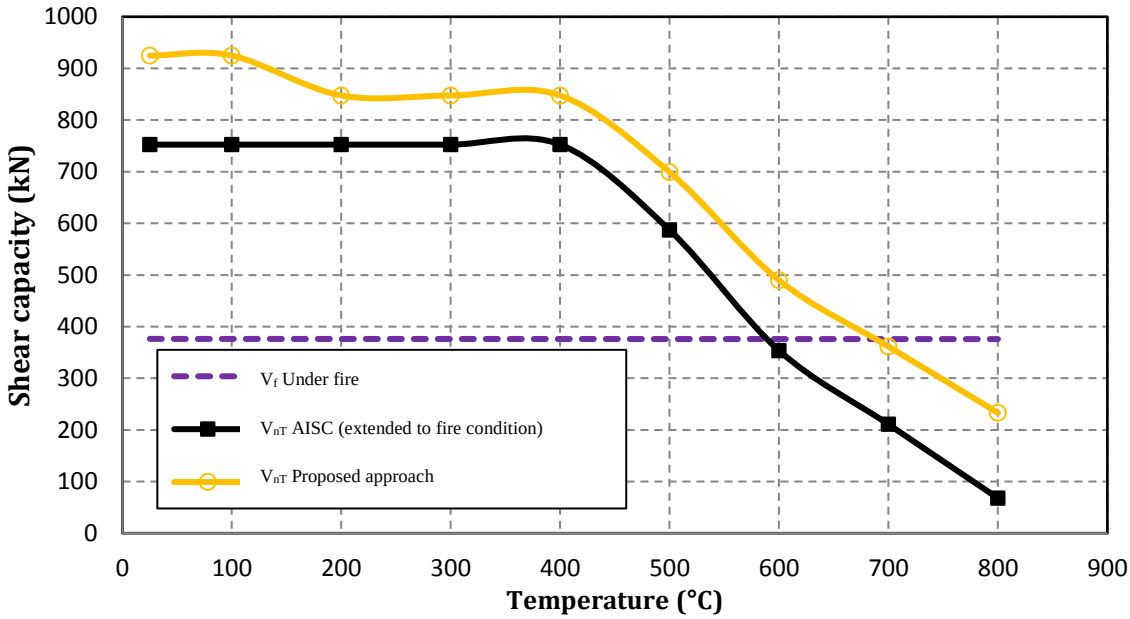

Fig. 11 Degradation of shear capacity of composite beam under fire exposure