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Effect of Temperature-induced Moment-Shear Interaction on Fire Resistance of Steel Beams

M.Z. Naser[1], V.K. Kodur[2]

[1]Glenn Department of Civil Engineering, Clemson University, Clemson, SC, 29634, USA (mznaser@clemson.edu, www.mznaser.com)

[2]Civil and Environmental Engineering, Michigan State University, East Lansing, MI, 48824, USA (kodur@egr.msu.edu)

1.0 ABSTRACT

The interaction between bending and shear effects in steel beams can be amplified under fire conditions due to rapid degradation in strength and stiffness properties of steel, together with temperature-induced local instability effects. This paper presents temperature-induced moment-shear (M-V) interaction phenomenon in compact (Class 1) steel beams. Results generated from numerical studies are utilized to quantify the effects of temperature-induced critical parameters influencing moment-shear interaction, shear and flexural sectional capacity, as well as instability in steel beams under fire conditions. The major findings of this work are two folds: (1) occurrence of temperature-induced instability adversely reduces shear capacity, as compared to flexural capacity, and (2) this rapid degradation in shear capacity trigger moment-shear interaction phenomenon at elevated temperatures. Eventually, this shifts failure mode in steel beams towards a shear dominant failure mechanism on the interaction envelope.

Keywords: Fire, Moment-shear interaction, Steel beams, Instability

2.0 INTRODUCTION

In most building applications, steel beams are primarily subjected to load effects arising from bending moment, and thus main consideration in design of beams is to satisfy bending requirements under both ambient and fire conditions [1, 2]. However, in certain scenarios, shear effects can dominate response of steel beams especially when subjected to certain loading configurations such as high concentrated (point) loads acting on beams, as in the case of transfer girders and beams connecting to offset columns in buildings [3]. Also, shear effects can be significant in beams with slender webs, such as built-up sections and deep beams. Recent studies have highlighted the susceptibility of steel beams and structures to shear and interaction-based failure, especially under fire conditions [4-9].

In a general sense, it is possible that a beam can be subjected to bending moment alone, but not to shear alone since shear is the derivative (i.e. rate of change) of bending moment. As a result, beams are often subjected to combination of bending and shear. In critical regions of a beam, closer to location of maximum shear force and bending moment, interaction of bending moment and shear effects can accelerate plastification in the beam (and eventually failure). Despite this "natural" vulnerability to pre-mature failure, provisions in most design standards such as AISC [10], AASHTO [11] and AS 2327.1 [12] do not specifically account for combined effects of moment and shear under fire conditions.

In fact, current AISC and AASHTO design specifications neglect moment-shear interaction in hot-rolled W-shaped steel sections whether at ambient or fire conditions. Only in the case of Eurocodes 3 [13], dealing with steel structures, moment-shear interaction in W-shaped beams at room temperature is to be considered under these two conditions: 1) when applied shear force does not induce web buckling, and 2) when shear force exceeds half the plastic shear resistance [13]. When moment-shear interaction is deemed to occur, Eurocode 3 applies a reduction to moment capacity to allow the web to be fully utilized in resisting shear force [13].

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The same Eurocode provisions allow extending room temperature design expressions to fire conditions if appropriate material property reduction factors are applied to account for temperature-induced degradation in strength and stiffness. Eurocode 3 provides recommendations on degradation of steel properties as a function of temperature and assumes degradation in modulus of elasticity and yield strength of steel to start at 150 and 400°C, respectively. As a result, temperature-induced web buckling, which is influenced by degradation in modulus of elasticity, could occur at 150°C, prior to any degradation in yield strength of steel (which starts at 400°C). This contradicts key design aspects that promote a ductile failure through web yielding over shear buckling. Hence, extendibility of Eurocode 3 moment-shear interaction design expressions to fire conditions can be debatable.

In order to explore the underlying mechanics of such phenomenon, this paper investigates effect of load ratio and temperature-induced instability on the development of moment-shear interaction in fire exposed W-shaped hot-rolled compact (Class 1) steel beams.

3.0 RESPONSE OF STEEL BEAMS UNDER FIRE CONDITIONS

In case of beams in buildings, level of bending moment (as compared to moment capacity) is usually much higher than that of shear force (when compared to shear capacity). Hence, in many cases shear effects are negligible and bending dominates behavior and failure of beams. Figure 1a illustrates response and failure mechanism of a typical steel beam subjected to dominant bending moment and exposed to fire. It can be seen that failure of this beam occurs once bending moment exceeds reduced level of moment capacity resulting from temperature effects. In a similar manner, failure of a beam loaded with high shear forces (and minor bending moment) occurs once the shear capacity falls below the level of applied shear force (see Fig. 1b).

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There could also be a third scenario in which failure occurs due to interaction of bending and shear stresses located at transition zones. A transition zone is defined as a section (in a beam) where resistance mechanism changes from flexure-based into shear-based dominant loading mechanism. This situation, wherein interaction between moment and shear can dominate response of beams, is illustrated by tracing response of a typical steel beam from loading stage to failure under fire exposure as shown in Fig. 1c. This figure shows a simply supported beam subjected to a uniformly distributed load and two point loads and these point loads produce varying bending moment and shear force across the span of the beam.

As can be seen in Fig. 1c, there are three different critical sections. The first critical section, located at mid-span of the beam, is where peak bending moment occurs and the failure can occur at this section once temperature-induced degradation in moment capacity reaches below the level of bending moment due to applied loading. The second critical section, located close to end supports, is mainly dominated by shear effects, and failure at this section could occur when temperature-induced degradation in shear capacity reaches below the level of shear force. In between these two critical sections lays a transition zone where bending and shear effects are quite high (but do not reach maximum values), and the combined effects can lead to failure in this region. The effect of combined forces, together with temperature-induced degradation in sectional moment and shear capacity, combined with temperature-induced instability, significantly complicates stress distribution, load path, and failure mechanisms.

A number of previous studies focused on moment-shear interaction effects at ambient conditions [1, 2]. Results from these studies have shown that interaction between bending moment and shear force is quite weak at ambient conditions due to two main reasons, 1) bending effects govern failure of simply supported beams, i.e. beam plastifies upon reaching its flexural capacity which occurs much earlier than reaching shear capacity, and 2) compact webs tend to yield before they buckle when subjected to combination of moment and shear loading. While these observations hold true at ambient conditions, the same observations may not be consistent with results from recent studies of fire exposed steel beams [5, 6]. To this date, very little research has been carried out on the interaction of bending and shear effects in beams under fire conditions [14-16].

4.0 MOMENT-SHEAR INTERACTION AT ELEVATED TEMPERATURES

In order to quantify moment-shear interaction phenomenon, Basler [17] developed the following interaction equation mainly derived for slender webs at ambient temperature.

$$\left(\frac{M_u - M_{yf}}{M_p - M_{yf}}\right)^2 + \left(\frac{V_u}{V_p}\right)^2 = 1 \qquad (1)$$

where, M_u and V_u are the applied bending moment and shear force, respectively, M_p and V_p are the plastic flexural and shear capacity, respectively and M_{yf} is the yield moment considering the flanges only.

Equation 1 has gone through number of revisions and an improved equation (Eq. 2) was finally adopted by the AISC [10] and AASHTO [11] specifications.

$$\frac{M_u}{M_p} + 0.625\frac{V_u}{V_p} = 1.375 \qquad (2)$$

However, the above expression, along with moment-shear interaction phenomenon, was discarded in recent editions of AISC manual as a result of a comprehensive survey commissioned by American Iron and Steel Institute (AISI), the Federal Highway Administration (FHWA), and the American Society of Civil Engineers (ASCE) and conducted by White et al. [18].

White et al. [18] statistically evaluated moment-shear interaction effects reported in numerous experiments. Their analysis concluded that moment-shear interaction either does not occur or its effect is minor and can be neglected. White et al. [18] recommendations were founded based on the fact that it is possible to independently consider the effect of bending moment and shear force rather than a combination of these effects. However, a follow up study carried out by Lee et al. [19] revealed that ignoring moment-shear interaction in beams loaded with high-moment and high-shear loading could lead to unconservative designs specifically at the juncture of web and flange where normal and shear stresses can be substantial. Despite these findings, current editions of AISC and AASHTO continue to neglect moment-shear interaction effects.

Unlike North American codes, Eurocode 3 still requires checking combined effects of flexural and shear interaction in the design of steel beams. Eurocode 3 accounts for moment-shear interaction in cases where, 1) shear force does not induce shear buckling of the web plate, and 2) when the shear force exceeds half the plastic shear resistance of that particular section. When moment-shear interaction occurs, Eurocode simply reduces the moment capacity to some degree using a shear interaction coefficient, ρ, such that:

$$M_p = (1 - \rho^2) f_y Z \qquad (3)$$

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where, $\rho = \frac{2V_u}{V_p} - 1 = \frac{2V_u}{A_v f_y/\sqrt{3}} - 1 \geq 0$

In Eq. 3, the plastic shear capacity, V_p, is valid for $b_f\, t_f/t_w\, h_w \geq 0.6$ as long as the average shear stress in the section does not cause web yielding, and $A_v = A_g - 2b_F\, t_f + (t_w + 2_r)t_f \geq h_w t_w$, where A_g is the gross-sectional area, b_f is the flange width, h_w is the clear web depth, r is the radius of fillet at the web–flange joint and t_w and t_f are web and flange thicknesses, respectively.

Equation 3 has been revised based on results from tests on steel beams that showed web to be able to carry a considerable shear even when steel sections reach the plastic moment. Thus, Eurocode 3 provisions now combine Eqs. 1 and 3 into Eq. 4 and these equations are intended for plastic failure analysis:

$$\frac{M_u}{M_p} + \left(1 - \frac{M_f}{M_p}\right)\left(\frac{2V_u}{V_p} - 1\right)^2 = 1 \tag{4}$$

All of the above presented equations were derived from classical analysis on flat plates with different aspect ratios and boundary conditions. The fundamental form of these equations is given by Eq. 5 and simply represents a "load-to-capacity" ratio:

$$\left(\frac{\sigma}{\sigma_{cr}}\right)^2 + \left(\frac{\tau}{\tau_{cr}}\right)^2 = 1 \tag{5}$$

where, σ and τ are the applied bending and shear stress, respectively, σ_{cr} is the critical stress for pure bending and τ_{cr} is the critical stress for pure shear. Both of these critical stresses are calculated as:

$$\sigma_{cr} = \frac{k\pi^2 E}{12(1-\nu^2)(b/t)^2} \tag{6}$$

$$\tau_{cr} = \frac{k\pi^2 E}{12(1-\nu^2)(^h/_{t_w})^2} \tag{7}$$

where, E is elastic modulus, v is Poisson's ratio, b, t, h, t_w are width and thickness of plates subjected to bending and shear and k is a plate buckling coefficient which is a function of both plate aspect ratio and wavelength parameter related to restraint conditions along the longitudinal boundaries (i.e. fixed or simply supported etc.). The plate buckling coefficient of unstiffened web in bending and shear equals to 23.9 and 5.34, respectively [19, 20].

Earlier studies assumed fixed-fixed or simply supported boundary conditions when evaluating plate buckling coefficient and critical buckling stress in steel beams [19-21]. These studies have shown that following such assumptions lead to good correlation with experimental data at ambient conditions. Unfortunately, the validity of these assumptions under fire conditions has not been fully investigated or verified [22, 23]. A closer look into Eqs. 6 and 7 reveals that critical stress is a function of geometric features of web plates as well as modulus of elasticity. Since modulus of elasticity is a temperature-dependent property, the critical stress can then be significantly influenced by temperature rise and associated degradation in modulus of elasticity.

In order to illustrate the effect of temperature rise on bending and shear critical buckling stresses (i.e. correspondingly moment-shear interaction), the response of W-shaped fire-exposed steel beams is examined herein. When a W-shaped steel beam is exposed to fire (say standard fire conditions), thermal gradients develop along the depth of beam cross section (comprising of top flange, web, and bottom flange) due to the fact that the beam is often heated from the bottom side [24]. Due to the small thickness of the flanges, it can then be safe to assume that both bottom and top flange have uniform temperature rise (in which the bottom flange being much hotter than the top flange). The web, on the other hand, can experience large thermal gradients due to its high depth and being located between a hot (bottom) flange and a cooler (top) flange.

Since stiffness properties of steel degrade with rise in temperature, the stiffness of bottom flange will be much smaller than that in the top flange. Observations from recent fire tests on beams have shown that when temperature at the bottom flange reaches 600°C, temperature at the top flange (and top portion of the web) can be lower by 200-300°C [5, 6]. Thus, when temperature in the bottom flange reaches 600°C, the temperature on the top flange can be conservatively assumed to be 250°C. The lower temperature of the top flange is attributed to two factors, 1) farther distance from top flange to fire exposure zone, and 2) the presence of a concrete slab which, due to insulating properties of concrete, act as a heat sink that attracts much of the temperature in the top flange.

Temperature-induced degradation in modulus of steel can be calculated as the product of room temperature modulus (E) by a reduction factor (β) applied to represent degradation in modulus at that temperature (as given in fire design standards i.e. Eurocode 3). Hence, when temperature in the top flange is 250°C, the degradation of elastic modulus in the flange (and top portion of the web) equals to $E_{250°C} = \beta \times E = 0.85 \times 210$ = 178.5 GPa (given that β = 0.85 at 250°C). This stiffness can be 2.7 times higher than that of the bottom portion of the web assuming it has a temperature similar to that of the bottom flange i.e. 600°C ($E_{600°C} = 0.31 \times 210$ = 65.1 GPa) [13]. This large variation in modulus causes reduction in overall plate stiffness and, most importantly, affect web restraint conditions (i.e. rigidity of flanges).

When temperature-induced degradation in stiffness in the bottom flange reaches this low level of initial stiffness, this flange may not be able to provide the same level of restraint as that at ambient conditions (or even as that of the top flange). In this case, the bottom portion of web has more flexibility to laterally move due to the weaker restraint provided by bottom flange. As a result, the assumption of having plate boundary conditions of fixed-fixed as proposed by Chern and Ostapenko [25] or simply-supported as proposed by Porter et al. [26] and Lee and Yoo [27] is questionable and may not hold true under fire exposure conditions. In fact, the actual restraint conditions is of more complex nature [28].

In order to verify this observation, a simple steel plate was modeled using a finite element model developed in ANSYS. The plate is discretized using "shell 181" elements (a four-noded element with six degrees of freedom: translations in the x, y, and z directions, and rotations about the x, y, and z axes). "Shell 181" was supplemented with elastic material properties, namely modulus of elasticity and Poisson's ratio. The plate had an aspect ratio of 4, thickness of 10 mm and was subjected to shear loading on all sides (and without any in-plane force or moment). A scale factor of *Depth/10000* was selected to induce initial imperfection to this plate [29]. Three loading scenarios were studied. In the first scenario, the plate was analyzed at ambient conditions where steel modulus of elasticity equals to 210 GPa. The analysis shows that the critical buckling stress for this plate equals to 428 MPa which is within 5% from that obtained using Eq. 7. Figure 2a shows von-Mises stress distribution in this plate.

In the second scenario, and in order to investigate effect of temperature rise on stress distribution, the plate was subjected to uniform temperature of 600°C. Hence, in this case, both top and bottom boundaries have similar stiffness. The stress distribution shown in Fig. 2b reveals that the plate has similar stress distribution (but with lesser magnitude) to that in the first scenario. This due to the fact that the only difference between the two cases is the use of a reduced modulus of elasticity of 65.1 GPa.

The effect of thermal gradients was also studied as part of the third scenario in which the plate was assumed to have three temperature fields of 100, 350 and 600°C (in top flange, web, and bottom flange, as shown in Fig. 2e). Results from the analysis shows that stress distribution (see Figs. 2c and 2e) as well as out-of-plane deformations (see Figs. 2d and 2f) are much different than that observed in the two scenarios due to the variation in stiffness as a function of temperature rise across the depth of the plate. These observations do not rule out the possibility of tension field action developing at elevated temperatures, especially when the strength-to-slenderness ratio of web becomes too small. These results also agree with observations from fire tests and clearly show that stress distribution and buckling behavior of a steel plate changes as a function of temperature-induced degradation in stiffness [5, 6]. More importantly, results of this analysis infer that this variation in shear stress, once combined with bending effects and applied to Eq. 6, can trigger changes in the moment-shear interaction behavior in fire-exposed steel beams that may trigger premature failure.

5.0 EVALUATION OF FLEXURAL AND SHEAR CAPACITY UNDER AMBIENT AND FIRE CONDITIONS

As discussed earlier, the main objective of this paper is to quantify the effect of temperature-induced strength degradation and instability effects on moment-shear interaction behavior of steel beams exposed to fire conditions. For these objectives to come through, the degrading flexural and shear capacity in steel beams need to be quantified first.

The current provisions for evaluating flexural capacity of beams under fire conditions are through extending room temperature design expressions available in AISC [10] and Eurocode 3 [13]. In these provisions, flexural failure can occur when a beam becomes fully plastic (reaching plastic moment capacity) and once flexural capacity falls below the bending moment resulting from applied loading. As such, flexural capacity of a W-shaped section is given as:

$$M_p = f_y Z_x \tag{8}$$

where, f_y is the yield strength of steel section, and Z_x is the plastic section modulus.

For evaluating flexural capacity under fire conditions, codal provisions extend room temperature design procedure with due consideration to temperature-induced reduction in yield strength of steel. All that is needed is to take into account appropriate reduction in yield strength of steel at a specified temperature (i.e. 100, 200, 300°C etc.) and this is given as:

$$M_{fire} = f_{y,T} Z_x \tag{9}$$

where, $f_{y,T}$ is the yield strength of steel section at temperature, T.

This approach simplifies flexural capacity calculations at elevated temperature and can be repeated at various temperatures, to derive the moment capacity-temperature response history.

In the case of shear response, shear capacity at room temperature can be evaluated as:

$$V_p = \tau_{yw} dt_w C_v \tag{10}$$

where, τ_{yw} is the shear yield strength of the steel web ($\tau_{yw} = 0.6f_{yw}$), t_w is the thickness of the web, d is the overall depth for hot-rolled beams, C_v is the web shear coefficient that depends on slenderness of web.

AISC provisions do not specifically state an extension of Eq. 10 to evaluate shear capacity-temperature response history. This is in contrast to the case of flexural capacity evaluation, where AISC allows Eq. 8 to be extended to Eq. 9. Naser and Kodur [15] have looked into the extendibility of Eq. 10 to fire conditions and found that this equation overestimates shear strength in fire-exposed hot-rolled beams as it does not specifically account for temperature-induced instability effects. As a result, they concluded that direct extension of Eq. 10 cannot be applied to account for evaluation for shear capacity under elevated temperature. They proposed modified equation to account for yield strength degradation as well as temperature-induced instability effects [15].

$$V_{fire} = \begin{cases} 0.6f_y\, A_w\, C_v, & T < 150°\mathrm{C} \\ 0.6f_{y,T}\, \beta t_w{}^2 C_{v,T}, & T \geq 150°\mathrm{C} \end{cases} \quad (11)$$

where, β is the critical slenderness parameter, $C_{v,T}$ is temperature-dependent web shear coefficient. Details on development of above equation can be found elsewhere [15].

A general procedure for evaluating interaction effects of shear force and moment in fire exposed steel beams is proposed through steps outlined in the flow chart shown in Fig. 3. The proposed procedure takes into account both flexural and shear limit states to evaluate failure in steel beams. The computed flexural and shear capacity (of a steel beam) is compared against effects of applied bending moment and shear force at various target temperatures (100, 200, 300°C etc.). Once flexural and/or shear capacity drops below level of bending moment and/or shear force, failure is said to occur. These steps outlined in the flow chart can be programmable into a simple and robust computer code.

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6.0 EFFECT OF TEMPERATURE-INDUCED CAPACITY DEGRADATION

It can be inferred from above discussion that one of the major differences in behavior of steel beams under ambient and fire conditions is the fact that strength and stiffness properties of steel degrades with rise in temperature. Since flexural and shear capacity reduces relative to degradation in strength properties, the reductions in properties can also change "load-to-capacity" ratio (often of constant magnitude at ambient conditions) leading to developing moment-shear interaction under fire conditions. In order to quantify such ratio, this section investigates the effect of temperature-induced property degradation on moment-shear interaction of steel beams.

Discussion in Sec. 5 presented number of design expressions that can be applied to evaluate both flexural and shear capacities. Once evaluated, these capacities are compared to existing bending moment and shear force actions, mainly to check if the "load-to-capacity" ratio satisfies interaction conditions. If one (or more) of these conditions is not satisfied, then moment-shear interaction is to be neglected. However, it is possible for a beam not to experience interaction effects at ambient conditions, but to develop such effects under high temperature conditions due to degradation in strength properties (and corresponding degradation in moment and shear capacity).

Temperature raise in steel and associated degradation in strength increases the ratio of bending moment-to-flexural capacity and shear force-to-shear capacity (such that $f_{y,550°C} << f_{y,25°C} \rightarrow M_{550°C} << M_{25°C} \rightarrow M_u/M_{550°C} >> M_u/M_{25°C}$). Since most codal provisions disregard moment-shear interaction in beams, and codes which account for such interaction does not account for this phenomenon under fire conditions, steel beams can be vulnerable and severely under-designed if subjected to moderate-to-high levels of bending moment and shear loading under fire conditions.

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In order to demonstrate the above observation, a simple example is carried out herein. In this example, a W18×40 made of Grade 345 MPa possessing flexural and shear capacity of 443 kN.m and 752.4 kN is selected for analysis [10]. This beam is subjected to bending moment and shear force equivalent to 40% of its moment and shear capacity (i.e. $M_u = 0.4 \times M_p = 0.4 \times 443 = 177$ kN.m and $V_u = 0.4 \times V_p = 0.4 \times 752.4 = 301$ kN). Inferring to the discussion presented in Sec. 4.0, both AISC and AASHTO provisions do not account for moment-shear interaction effects and hence according to these provisions, moment-shear interaction in the selected W18×40 beam is deemed minor and could be neglected. In the case of Eurocode provisions, the applied shear force level equals to 40% and does not exceeds half the plastic shear resistance. As a result, the moment-shear interaction effects in this beam are also assumed to be minor and could be neglected.

In the case this beam is exposed to fire and temperature in beam (steel) reaches 550°C. This rise in temperature leads to significant degradation in strength properties in the range of 62% ($f_{y,550°C} = \beta \times f_{y,25°C} = 0.62 \times 345 = 214$ MPa). As a result, moment and shear capacity of this beam will also reduce, such that:

$$M_{p,550°C} = 0.62 \times 443 = 274.7 \text{ kN.m}$$

$$V_{p,550°C} = 0.62 \times 752.4 = 466.5 \text{ kN}$$

Since the magnitude of applied loading ($M_u = 177$ kN.m and $V_u = 301$ kN) is often maintained under fire conditions, the ratio of bending moment-to-flexural capacity and shear force-to-shear capacity at 550°C increases (from 40%) to about 64 and 65%, respectively:

$$\frac{M_u}{M_{p,550°C}} = \frac{177}{274.7} = 0.64, \frac{V_u}{V_{p,550°C}} = \frac{301}{466.5} = 0.65$$

While AISC and AASHTO provisions neglect interaction effects, revisiting Eurocode provisions to check for moment-shear interaction shows that shear force exceeds half the plastic shear resistance at 550°C, and hence moment-shear interaction can occur. This simple example clearly shows that moment-shear interaction can develop under fire conditions due to temperature-degradation in strength properties. Figures 4a and 4b further trace "load-to-capacity" ratio in the selected beam as a function of elevated temperature. It can be seen from trends plotted in these figures that as temperature rises, higher property degradation occurs in material properties and the applied "load-to-capacity" ratio increases which accelerates development of moment-shear interaction effects. In other words, at ambient condition, the moment-shear interaction in this beam is minor and lays in the low moment-low shear region. However, with the rise in temperature, the level of moment-shear interaction increases until failure occurs.

Figure 4c shows the effect of increasing "load-to-capacity" ratio for bending (or shear), with levels ranging between 25, 40 and 80%, as a function of temperature rise on failure of steel beams. These results agree with that shown in Fig. 4a, and further demonstrate the rapid rise in "load-to-capacity" ratio at temperatures higher than 400°C, occurring due to the rapid degradation in yield strength of steel at temperatures exceeding 400°C. Figure 4c also shows that failure (i.e. load/capacity ratio = 1) under a high load/capacity ratio can occur at lower temperatures. For example, failure occurs at 635, 550 and 500°C in case of applied loading with magnitude of 25, 40 and 80%, respectively.

In order to further investigate moment-shear interaction behavior of beams, fire response of three W18×40 beams, referred to as "Beam 1", "Beam 2" and "Beam 3", is traced when subjected to different patterns of loading. These beams are subjected to high shear-low moment

(Beam 1), medium shear-medium moment (Beam 2), and low shear-high moment (Beam 3), respectively. The high shear-high moment and low shear-low moment loadings were calculated as 75 and 25%, respectively, of shear and flexural capacity of W18×40 section while the medium loading was equivalent to 40% of shear and flexural capacity of the same section.

Figure 5 traces developed moment-shear interaction behavior of these beams at temperatures of 25, 500, 600 and 700°C. It can be seen from plotted data that "Beam 1" and "Beam 3" reaches failure envelope at a lower temperature as compared to "Beam 2" which is loaded with medium levels of shear and moment. This is due to the fact higher levels of applied loading develop larger level of stresses within beam cross section, specifically in web (in case of high shear forces i.e. "Beam 1") or in flanges (in case of high bending moment, i.e. "Beam 3"). At temperature of 600°C, these large stresses can reach reduced yield strength of steel which leads to beam plastification. In order for the same magnitude of stresses to develop in "Beam 2", further degradation to strength properties need to take place which occurs at higher temperature than that reached in "Beams 1 and 3". Thus, "Beam 2" fails at slightly higher temperature than that of "Beams 1 and 3".

7.0 EFFECT OF TEMPERATURE-INDUCED INSTABILITY

Number of studies have shown that temperature-induced instability can start in steel beams at temperature as low as 150°C [14, 15, 30]. This is due to degradation in modulus of steel at temperatures beyond 150°C. Once modulus degrades, buckling capacity of plates (web or flanges) reduces and causes additional losses to shear (or flexural) capacity. More specifically, recent fire tests and numerical studies have pointed out vulnerability of steel web to temperature-induced instability [15, 16]. This vulnerability arises from the fact that web is usually thinner and more slender than flanges. Further, steel webs are exposed to higher thermal (fire) loading; since web has larger surface area and exposed to the fire from two sides. Hence, strength (and modulus) properties of steel in web can degrade at a rapid rate than that in flanges. As a result, shear capacity of steel beams can degrade at a much higher rate than flexural capacity since small area of web is main contributor to shear capacity (when compared to the area of two flanges). It should be noted that previous studies have shown that losses due to temperature-induced web instability effects can be as high as 20-25% [5, 15].

The effect of temperature-induced instability on moment-shear interaction of a typical steel beam (similar to "Beam 1" shown in previous sections) can be seen in Fig. 6a. This figure shows that effect of temperature-induced instability starts to be apparent at temperatures exceeding 150°C i.e. compact steel section transforms to a non-compact (and possibly into slender) section with further rise in temperature [4]. The difference in moment-shear interaction response by including instability effects and that of when instability is excluded is about 5% at temperature range between 150-500°C. This variation then rapidly grows to 10 and 17% at temperatures of 600 and 700°C, respectively. Incorporating temperature-induced instability effects cause the beam to fail at slightly lower temperature of 603°C (as compared to 635°C when the instability effect is neglected, see Fig. 4b).

Data plotted in Fig. 6b also shows that occurrence of instability further reduces shear capacity, as compared to flexural capacity. In other words, temperature-induced instability shift V_u/V_p ratio towards shear dominant failure on the interaction failure envelope (V_u/V_p =1). As a result of this instability, the beam fails in shear mode and not in simultaneous flexural/shear mode (see highlighted portion of Fig. 6). Figure 6 also shows that magnitude of instability effects can be substantial at higher temperatures (and higher level of loading).

8.0 CONCLUSIONS

This paper presents temperature-induced capacity degradation under combined effect of moment and shear in fire exposed steel. While this study presented a fundamental understanding on development of moment-shear interaction under fire conditions, there is scope for further research to extend this study towards tracing different states of moment-shear interaction and exploring the use of fiber-reinforced concrete (FRC) in concrete slabs to better stabilize composite beams subjected to combined loading effects. Finally, the following key conclusions can be drawn:

1. While moment-shear interaction in steel beams is minor and can be neglected at ambient conditions, steel beams can experience significant moment-shear interaction effects that can get amplified under severe fire conditions due to the rapid degradation in shear capacity trigger moment-shear interaction phenomenon at elevated temperatures
2. Development of temperature-induced instability effects can reduce moment capacity and shear capacity of steel beams under fire conditions by about 20-25%.
3. Temperature-induced instability adversely reduces shear capacity, as compared to flexural capacity; thus, effectively shifting failure in steel beams towards a shear dominant failure mechanism on the interaction envelope.
4. The adverse effects of temperature-induced sectional instability are more apparent in the case of shear capacity, as oppose to flexural capacity. The occurrence of such sectional

instability can shift failure of compact steel beams towards the shear dominant region on the moment-shear interaction envelope.

9.0 ACKNOWLEDGMENT

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10.0 NOTATIONS

M_u applied bending moment

V_u applied shear force

M_p plastic flexural capacity

V_p plastic shear capacity

M_{yf} yield moment considering the flanges only

A_g gross-sectional area

b_f flange width

h_w clear web depth

r radius of fillet at the web–flange joint

t_w web thicknesses

t_f flange thicknesses

σ_{cr} critical stress for pure bending

τ_{cr} critical stress for pure shear

E elastic modulus

v Poisson's ratio

b, t width and thickness of plates subjected to bending

h, t_w width and thickness of plates subjected to shear

k plate buckling coefficient

f_y yield strength of steel section

Z_x plastic section modulus

$f_{y,T}$ yield strength of steel section at temperature

τ_{yw} shear yield strength of the steel web

d overall depth for hot-rolled beams

C_v web shear coefficient that depends on slenderness of web

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LIST OF FIGURES:

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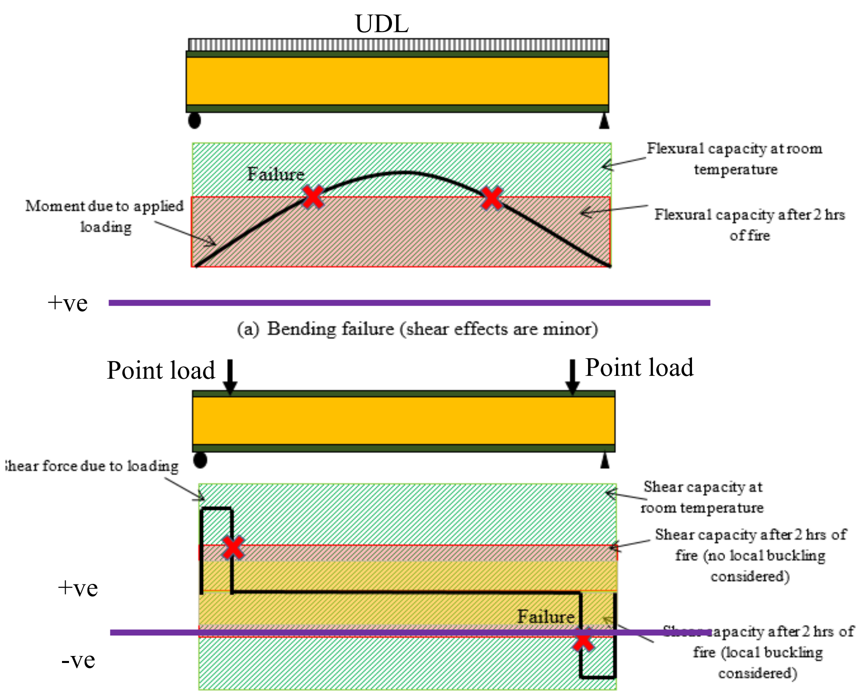
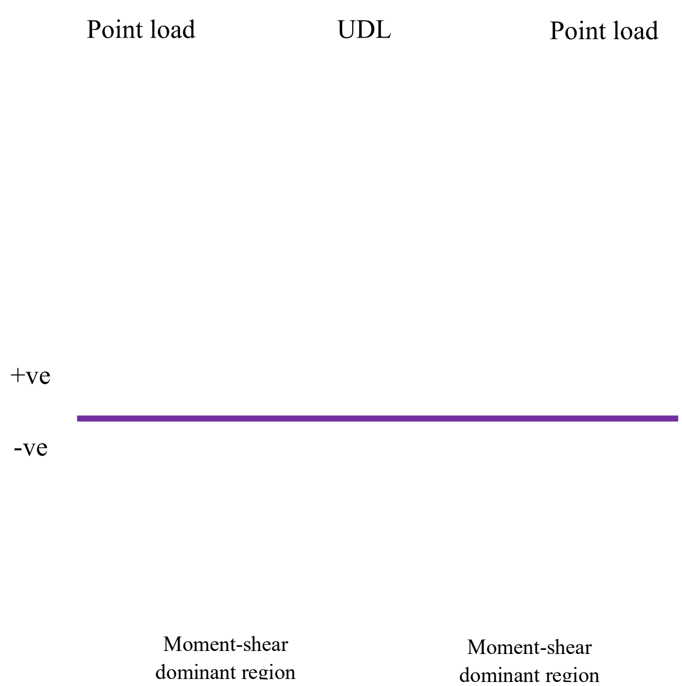


(a) Bending failure (shear effects are minor)

(b) Shear failure (bending effects are minor)



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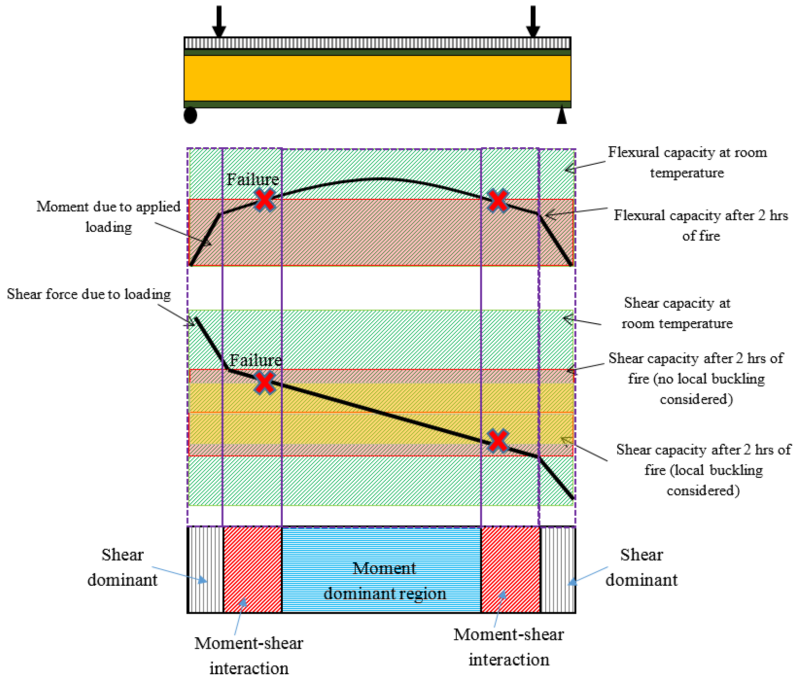
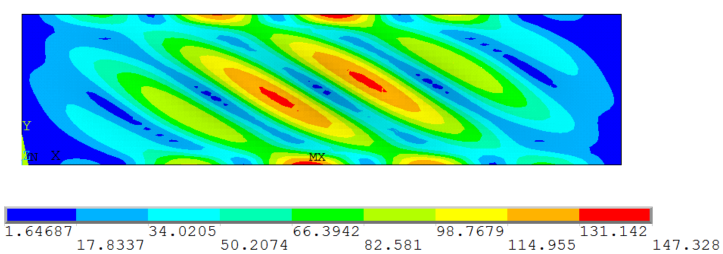
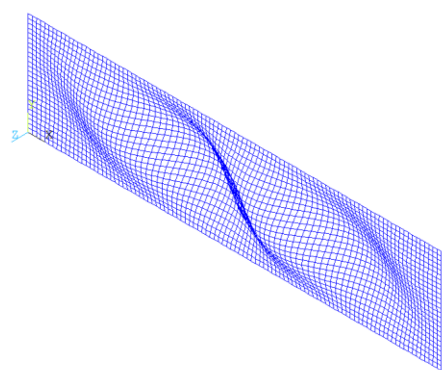


(c) Failure due to moment-shear interaction

Fig. 1. Variation of bending moment and shear force in beams under fire conditions



(a) Stress distribution of steel plate with modulus of 210 GPa



(b) Deformed shape

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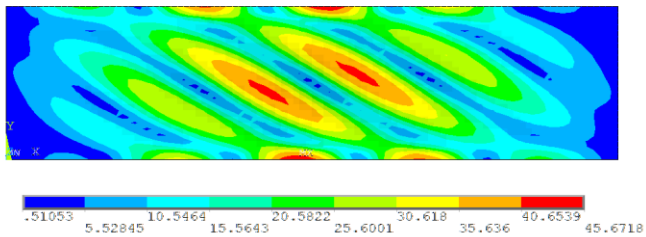
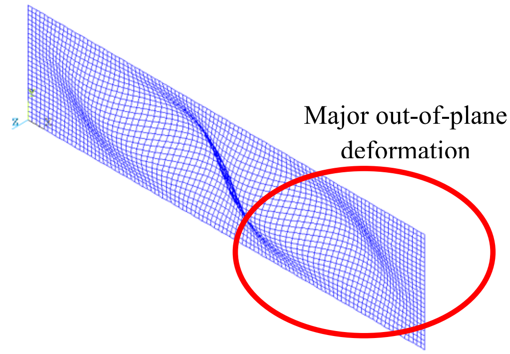
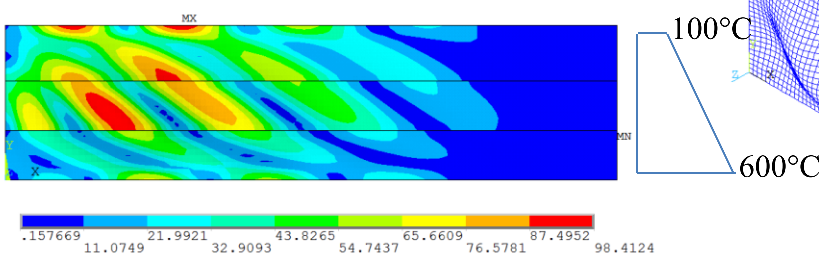
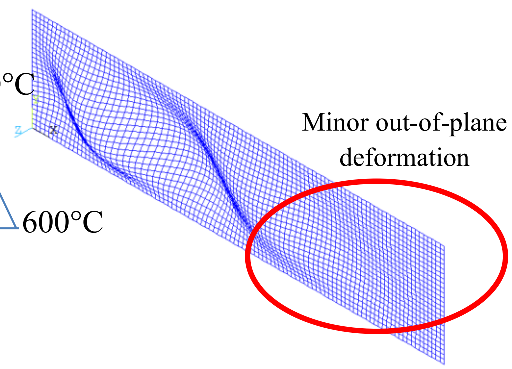




(c) Stress distribution of metal plate with modulus of 65.1 GPa

(d) Deformed shape





(e) Stress distribution of steel plate where section has varying modulus of 210, 157.5 and 65.1 GPa

(f) Deformed shape

Fig. 2 Von-Mises stress distribution in steel plate (web) under various conditions

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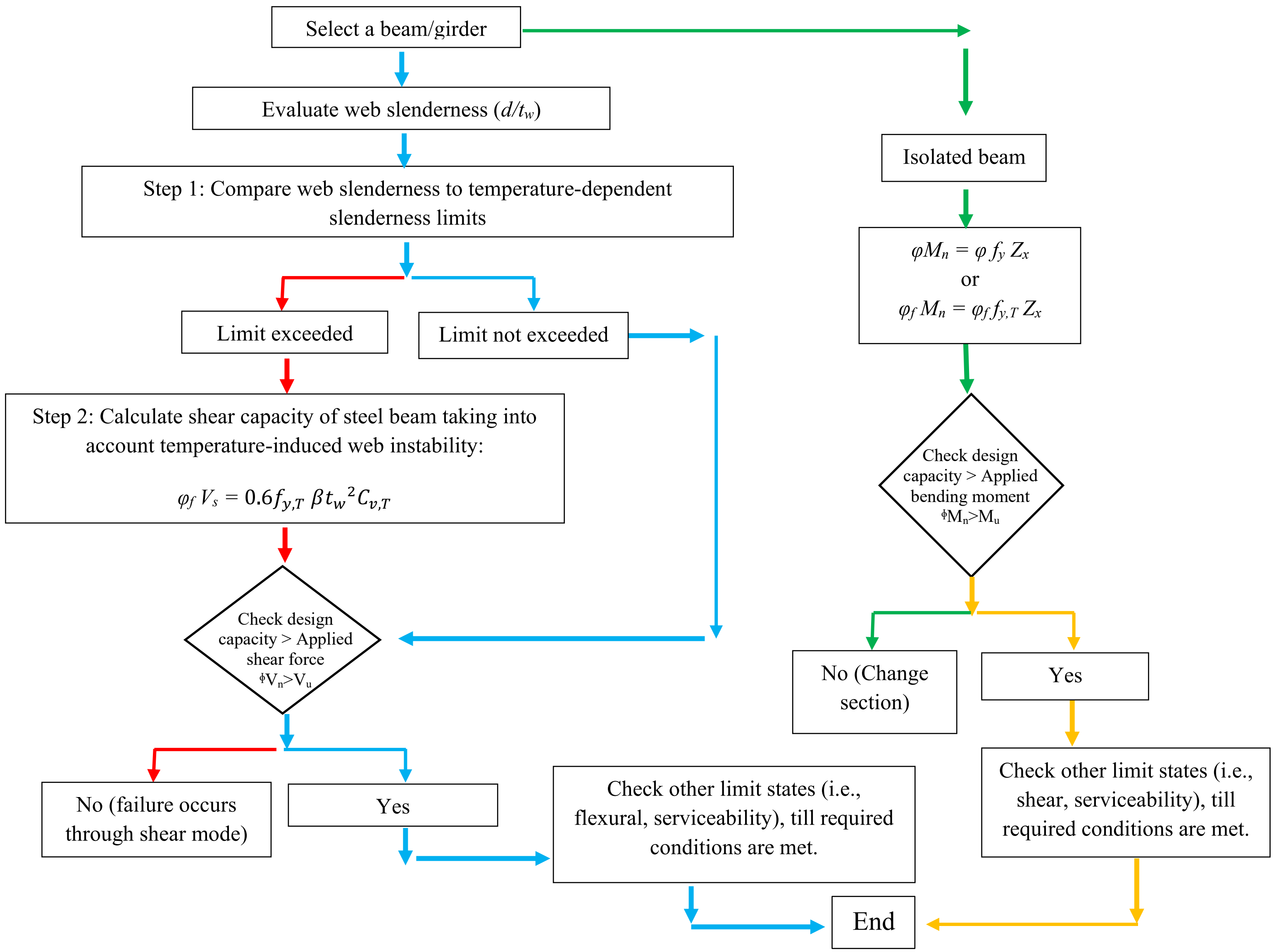

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Fig. 3 Evaluation of flexural and shear capacity in fire exposed steel beams

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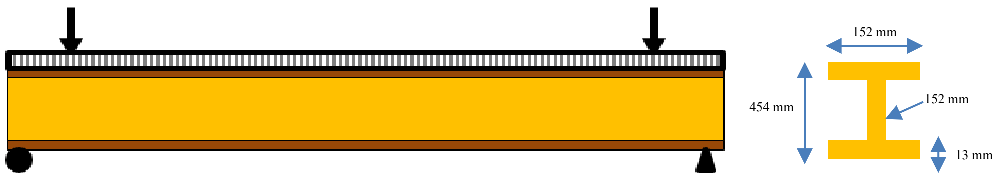
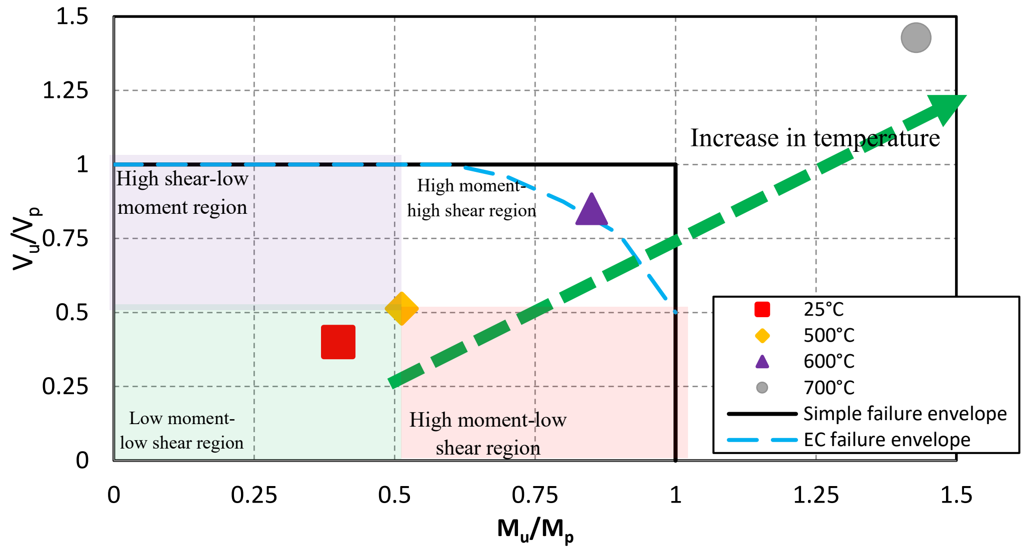
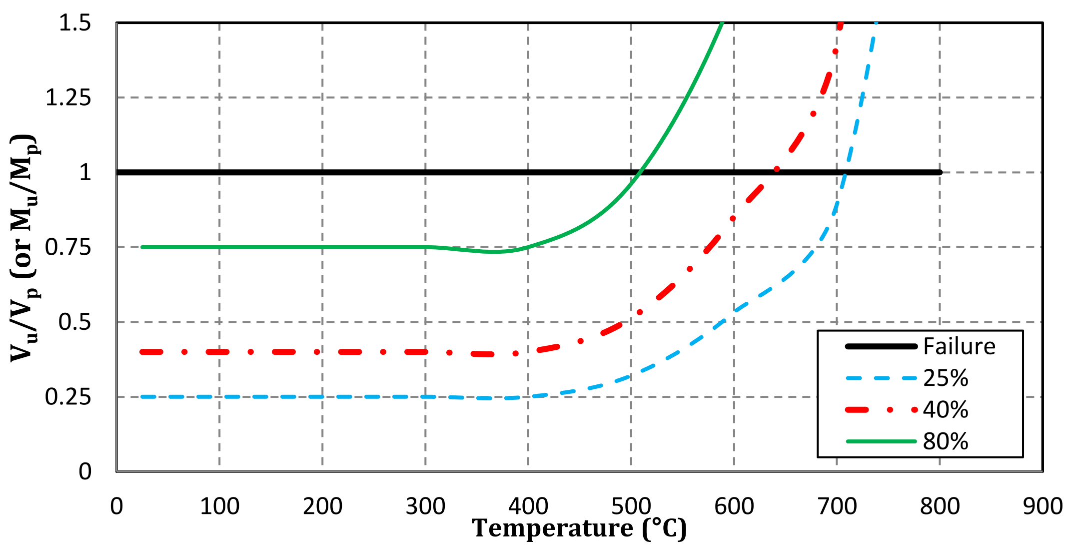


(a) Typical loading conditions



(b) Development of moment-shear interaction under fire conditions for load-to-ratio level of 40%



(c) Development of moment-shear interaction for various “load-to-capacity” ratios under fire conditions

Fig. 4 Effect of temperature-induced property reduction on moment-shear interaction

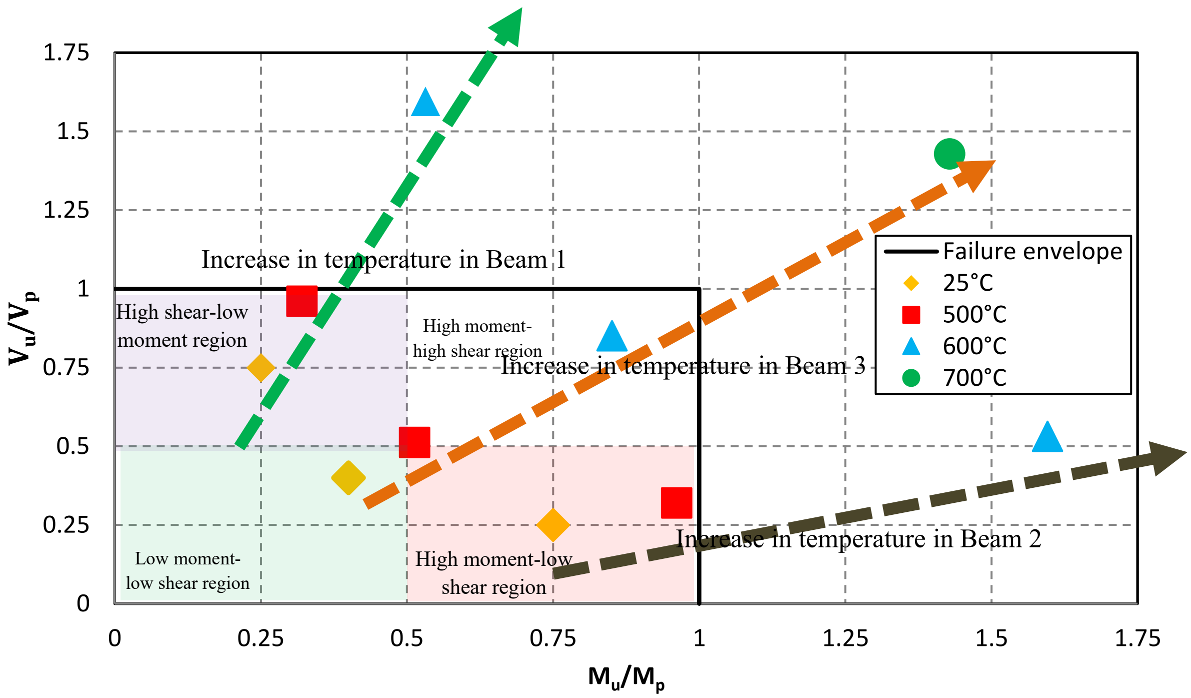


Fig. 5 Moment-shear interaction in Beams 1, 2 and 3

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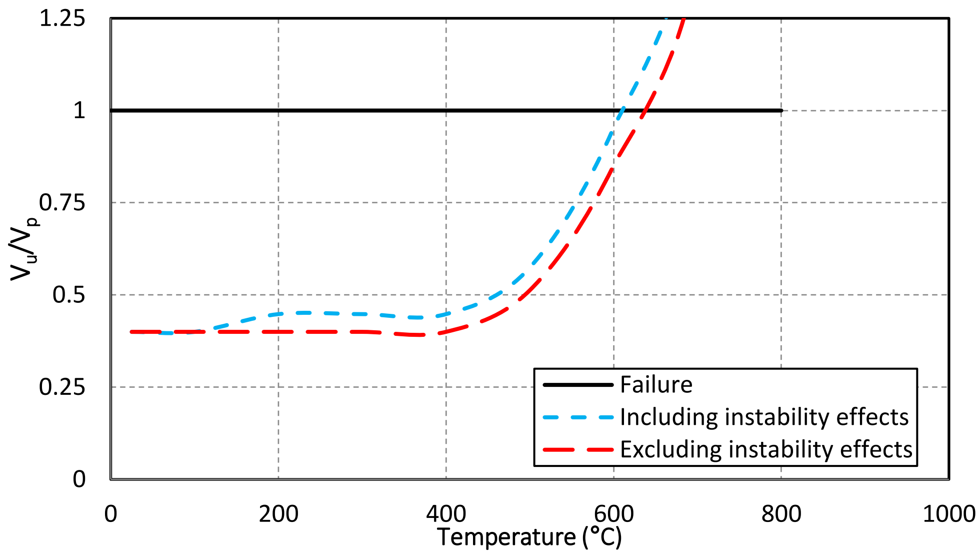
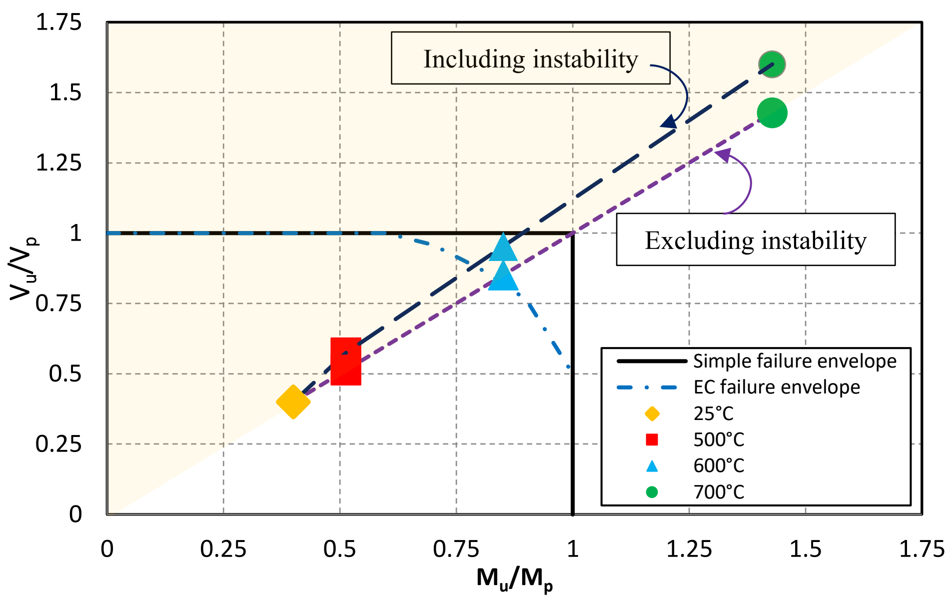


(a) Variation in normalized shear response



(b) Variation in moment-shear interaction response

Fig. 6 High-temperature moment-shear interaction in beams with and without temperature-induced instability effects (PS. the shear dominant region is highlighted)