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Effect of local instability on capacity of steel beams exposed to fire

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ABSTRACT

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

1. Introduction

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

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

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

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

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influence of critical factors including fire scenario, end restraints, thermal gradients, load level, and failure criteria in evaluating fire resistance (flexural only) of restrained steel beams. The authors infer that in axially restrained steel beams, the influence of flange local buckling on the fire response is insignificant due to the development of tensile catenary action. However, effect of shear loading, web local buckling and instability of steel beams at elevated temperatures is not taken into consideration in evaluating fire response of steel beams [6–9].

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

The occurrence of local buckling in steel members under high temperature conditions has been studied by few researchers [10–13]. Uy and Bradford and Heidarpour and Bradford applied finite strip method to study effect of local buckling in cold formed steel structural members subjected to elevated temperatures. Results from these studies show that presence of high flexural and shear loading can significantly influence the onset of local buckling in the web. Zhao and Kruppa conducted fire tests to study the flexural behavior of fire exposed composite steel beams [12]. The authors reported that these composite beams can experience local buckling at interior supports. However, no further recommendations were discussed, possibly because tested beams were subjected to dominant moments and thus the effect of shear and fire-induced instability could not be isolated. Recently, Wang et al. [13] presented an experimental investigation on the local buckling phenomenon in flange and web of thin-walled cross sections subjected to elevated temperatures. Data from these tests show that buckling resistance of stub columns decrease with increase in temperature and Eurocode 3 provisions predict higher buckling load in columns than those observed in fire tests.

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

2. Efefct of sectional instability on flexural and shear capacity

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

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

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

Element	λ	Flexural capacity		Shear capacity	
		λ_p	λ_r	λ_p[a]	λ_r[b]
Flange	$b_f/2t_f$	$0.38\sqrt{\frac{E}{f_y}}$	$1.0\sqrt{\frac{E}{f_y}}$	–	–
Web	h/t_w	$3.76\sqrt{\frac{E}{f_y}}$	$5.70\sqrt{\frac{E}{f_y}}$	$1.10\sqrt{\frac{k_vE}{f_y}}$	$1.37\sqrt{\frac{k_vE}{f_y}}$

b_f is flange width; t_f is the flange thickness; h is the depth of section and t_w is the web thickness.
kv is 5 for unstiffened webs with $\lambda \leq 260$.
[a] Limit of inelastic web buckling.
[b] Limit of elastic web buckling.

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

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

Flange and web (sectional) slenderness are compared against slenderness limits (given in Table 1) to classify "shape of" steel beams at

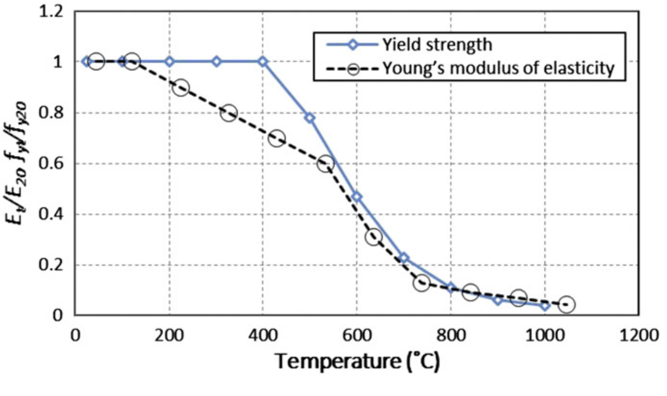

Fig. 1. Degradation of strength and stiffness properties of steel at elevated temperatures.