Proceedings of the
Annual Stability Conference
Structural Stability Research Council
Toronto, Canada, March 25-28, 2014

Effect of Shear on Stability of Steel Girders Under Fire Conditions

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

Abstract
In current practice, failure in beams under fire conditions is evaluated based on flexural limit state without any consideration to shear effects. However in certain scenarios, fire exposed beams and girders can experience temperature induced sectional instability due to shear rather than flexural effects. This paper presents a three-dimensional nonlinear finite element model for evaluating behavior of fire exposed steel girders. This model, developed in ANSYS, is capable of predicting fire response of steel girders taking into consideration flexural, shear and deflection limit states. The validated model is utilized to study different conditions under which shear parameters dominate the response of fire exposed steel girders. Results from numerical studies show that shear capacity can degrade at a higher rate than flexural capacity under certain loading scenarios and hence, failure can result from shear effects prior to that due to flexure. In addition, web slenderness and reserve shear capacity are also found to influence the onset of sectional instability in fire exposed steel girders.

Keywords: steel girders, fire resistance, shear, flexure, finite element modeling

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 capacity (typically moment capacity) at the critical section of the member drops below the applied moment due to loading, failure occurs. The time to reach this failure is referred to as fire resistance. The failure time under fire conditions can be severely affected by stability consideration at material, sectional and member levels. 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 derived based on flexural limit state only. This flexural limit state, although valid for most common scenarios, may not be representative in certain situations where shear effects can be dominant in a fire exposed member.

The most common example where shear forces can be dominant in beams is concentrated loads acting near end of beams connecting to offset columns in buildings (Hall 1954). Shear can also control the design of transfer girders, coped beams, short span beams and deep beams/plate

[1] Professor, Civil and Environmental Engineering, Michigan State University, Email: kodur@egr.msu.edu
[2] Ph.D. Candidate, Civil and Environmental Engineering, Michigan State University, Email: nasermoh@msu.edu

girders. Transfer girders and deep beams, used in lobbies and conference halls, can be subjected to high shear force, resulting from concentrated loads arising from supporting columns or walls (Garlock et al. 2012). Coped beams are those beams with reduced cross-sectional area near supports to facilitate connection of a beam to other beam/column joints.

Further, in beams with slender webs, such as deep beams and plate girders, reserve shear capacity can be much lower at ambient conditions and thus shear effects can trigger failure in fire exposed deep beams. Webs in I-shaped sections or plate girders can be thin compared to that of flanges. These thin webs experience rapid rise of temperature under fire exposure as compared to flanges. Therefore, shear capacity of steel beams can degrade at a much higher rate than flexural capacity; since area of the web is main contributor to shear capacity. Temperature induced loss of strength and modulus properties of steel in the web can cause severe instabilities at material and sectional levels and can lead to onset of global instability.

Temperature induced sectional instability can occur in steel beams when internal stresses reaches close to the yield strength limit. At this point, steel starts yielding and undergoes plastic deformations which in turn can produce sectional instability. Local buckling is a highly complex phenomenon and can significantly alter structural response of fire exposed girders. In beams with thin webs, occurrence of local buckling can negatively affect shear response under fire conditions (Kodur and Naser 2013).

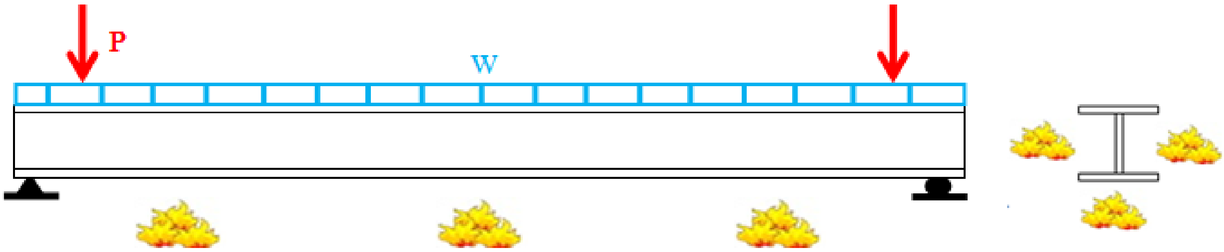
A review of literature clearly show that most previous studies focused on fire behavior of beams pre-dominantly subjected to bending effects (Kodur and Dwaikat 2010, Dwaikat and Kodur 2011, Aziz and Kodur 2013). These studies considered effects of various factors on flexural response of fire exposed steel beams such as restraint conditions, inelastic response, thermal gradients etc. However, the effect of shear parameters on fire response of beams was not considered.

To evaluate effect of shear on response of a fire exposed steel girder, 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 beams and girders subjected to significant bending moment and shear force. The model was validated against test data on beams and then the model was applied to examine the influence of shear on fire response of steel girders under different loading configurations and web slenderness.

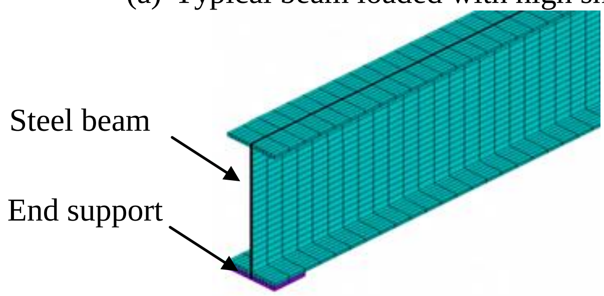
2. Numerical Model

To study the effect of shear on the response of beams under fire conditions, a finite element model was developed using ANSYS. Several parameters including geometric and material nonlinearities, temperature dependent material properties and various failure limit states are accounted for in the model in order to trace the realistic fire response of steel girders.

The three dimensional finite element model of the beam has geometry of a typical hot-rolled steel I-section commonly used in flexural members. For undertaking fire resistance analysis, the beam is discretized with different thermal and structural element, available in ANSYS. SOLID70 and SURF152 elements are used as thermal elements to simulate heat transfer to the beam under fire exposure. SOLID185 is also used for modeling the structural response of three-dimensional solid structure (ANSYS 2011). This finite element model is meshed using 35,000 quadrilateral-type elements. Figure 1 shows typical steel beam and associated finite element model.



(a) Typical beam loaded with high shear forces



(b) Isometric view of the finite element model

Figure. 1: Typical steel beam and developed finite element model

For undertaking fire resistance analysis, temperature-dependent thermal and mechanical properties of steel are to be input to the finite element model. These thermal and mechanical properties of structural steel are assumed to vary with temperature as per Eurocode 3 recommended relations (CEN 2005). Material nonlinearity of steel is accounted for through a bi-linear elasto-plastic stress-strain model of steel based on the Von-Mises plasticity yielding criterion. For fire insulation room temperature thermal properties are used in fire resistance analysis due to lack of information on high temperature thermal properties.

In order to simulate the response of fire exposed steel girders, two stages of analysis are to be carried out at each time step. The first stage examines heat transfer between fire source and steel girder. In this stage, temperature profiles and gradients are generated based on fire scenario the girder is exposed to. Then, these cross-sectional temperatures are input to the second stage of simulation to carry out structural analysis. In the structural analysis, both temperature and loading is applied simultaneously and the mechanical behavior i.e., mid-span deflection, stress and strain state of fire exposed beam is evaluated. Sectional capacity can be obtained as well. Utilizing internal bending and shear stresses generated from structural analysis, moment and shear capacity is evaluated. These stresses, generated at individual nodes/elements, are integrated across the depth of the steel section, using supplementary external routine, to arrive at moment and shear capacities at any time step.

At each time step, the internal moment and shear capacitates at critical sections as well as deflections were compared against different limiting criteria namely flexural, shear and deflection limit states to check failure of the fire exposed girders. Flexural and shear failure occur once the bending moment (or shear force) due to applied loading exceed the moment (or shear) capacity at a critical section. Also to check failure, mid-span deflection is compared against deflection limit state used in BS 476 (BS 1987). The beam is said to fail, when the beam attains a deflection of (L/20) or rate of deflection reaches (L^2/9000d); where L and d are the span and depth of the beam, respectively.

3. Validation of Numerical Model

Since there is lack of published fire test data on steel beams subjected to high shear forces, the above finite element model was validated using data from tests on conventional steel beams. Kodur and Fike (2009) reported detailed results from fire resistance test on a W12×16 A992 steel beam exposed to ASTM E119 standard fire. The beam was insulated with 50 mm thick spray applied vermiculite based fire insulation to achieve a 2-hr fire resistance rating. The beam was loaded with two symmetrical point loads 1.5 m away from end supports as shown in Fig. 2. This loading represents 31 and 5% of its room temperature flexural and shear capacity, respectively. The moment and shear capacity of the beam at room temperature is 102.2 kN-m and 440 kN, respectively as per AISC provisions (2011).

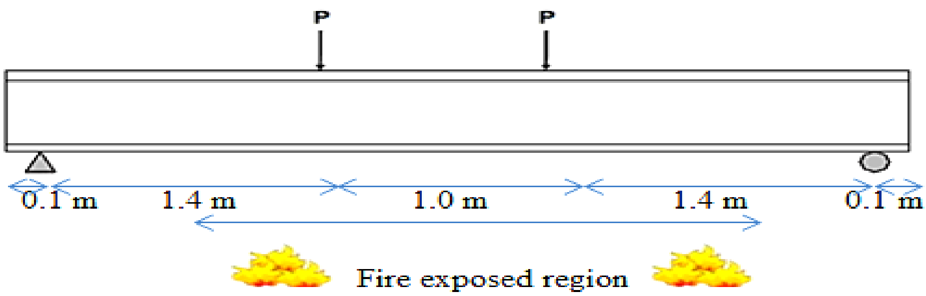
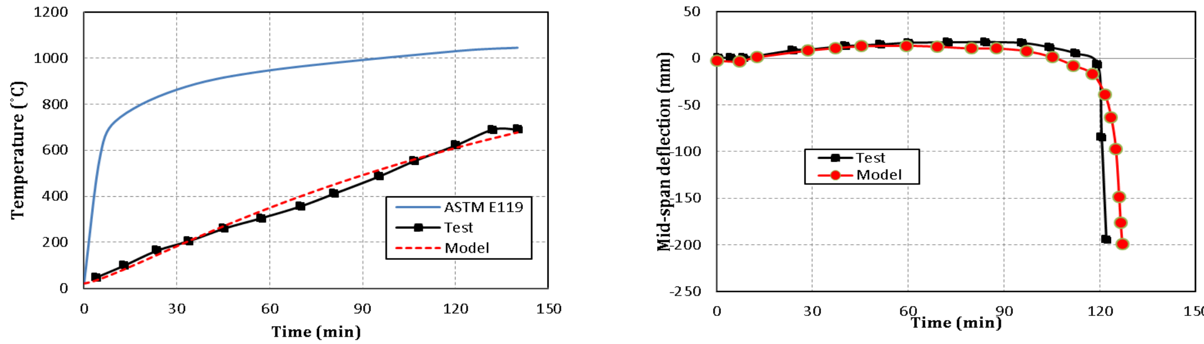


Figure 2: Tested beam used in validating the developed finite element model

The tested beam is analyzed using the above developed model. The various output parameters generated in the analysis i.e., cross-sectional temperature profile, mid-span deflection and failure mode are compared against measured data from fire test. Figure 3a shows a comparison of measured and predicted temperature (average of both flanges and web) in the steel beam as a function of fire exposure time. As can be seen, presence of insulation slows down rise of temperature in the steel section. There is a good agreement between predicted and measured temperatures up to the first 45 minutes. At 45 minutes, average temperature in steel section was around 350˚C. Beyond 45 minutes, the predicted steel temperatures (from model) tend to be slightly higher than the measured ones in temperature range of 350-600˚C. This can be attributed to differences in assumed and actual thermal properties of fire insulation at elevated temperatures. However, both measured and predicted temperatures converge toward the end of fire exposure.

A comparison of predicted and measured mid-span deflection response of the tested steel beam is shown in Fig. 3b. The beam undergoes only small deflection in initial stages of fire exposure and this remains constant till about 90 minutes. This can be attributed to the factor that steel does not experience significant degradation in strength in 20-400˚C but experiences moderate loss in elastic modulus in this temperature range. However, as the temperature in steel beam reaches 550˚C, at about 100 min, strength and stiffness properties of steel start to rapidly degrade which results in sudden increase in deflection. After 120 minutes of fire exposure, steel loses most of its strength and stiffness as the temperature of the beam rise to 600˚C, which produces runaway failure in the beam at 122 min.



(a) Thermal response (b) Structural response

Figure 3: Comparison of predicted and measured parameters as a function of fire exposure time

Figure 4 shows the degradation of moment capacity at mid-span section and shear capacity at support section with fire exposure time. The moment and shear capacity reserve at critical sections of beam, at the start of fire exposure is about 3.3 and 20 times that due to applied loading. This indicates that the beam has much higher reserve shear capacity the reserve moment capacity.

The moment capacity in the beam remains intact for the first 75 minutes due to lower average temperature in flanges (much below 350˚C) of steel beam. However, shear capacity starts to degrade at 35 min due to relatively faster rise in web temperature generated from higher web slenderness. After this, steel temperature in flanges and web rise beyond 350˚C. Then, both moment and shear capacity gradually degrade with increase in temperature in steel section. Degradation of both moment and shear capacity at critical sections continues till the beam fails at 130 min; when the moment capacity at mid-span drops below the moment due to applied loading. Due to higher reserve shear capacity (near end supports), the beam does not experience shear failure. Failure of this beam occurred at 122 min in fire test indicating reasonable agreement with predictions from model. The good comparison on predicted temperature and deflection, as well as failure mode, with test data indicated that the proposed model is capable of tracing the fire response of steel beams. Further, the moment and shear capacity degradation trends generated from the analysis follow expected trend, based on rational analysis, and thus the model is deemed to be capable of capturing overall fire response of steel beams.

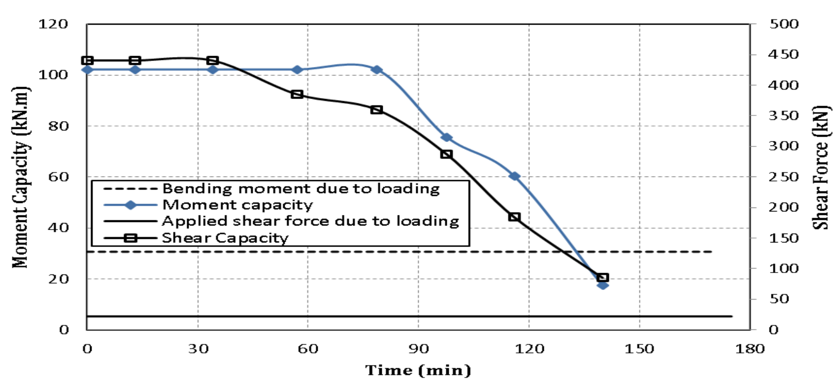


Figure 4: Degradation of moment and shear capacity in the tested beam

4. Case Studies

The above validated finite element model was applied to study the effect of shear parameters on the fire response in steel beams. The effect of loading pattern and web slenderness on shear capacity and fire response of beams is studied herein.

4.1 Effect of loading

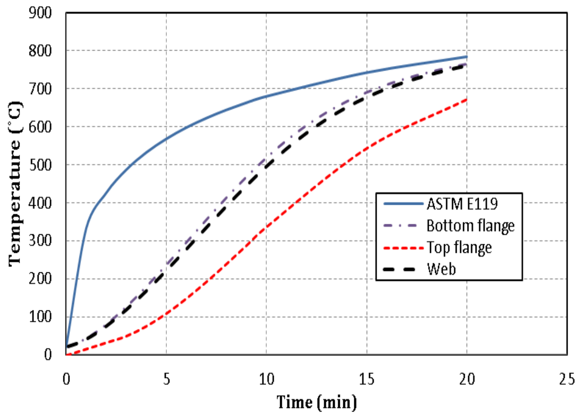
For numerical analysis, a simply supported beam of 9.14 m span and made of W16×31 section (AISC 2011) is selected. W16×31 section has a flange width of 140.6 mm and overall depth of 404 mm. The flange and web thicknesses are 11.2 mm and 7 mm, respectively. The beam is made of Grade 345 (MPa) steel and has continuous lateral support along its 9.14 m span. To illustrate the effect of shear arising from different loading patterns, two configurations of this beam, "Beam 1" and "Beam 2", were analyzed.

For fire resistance analysis, "Beam 1" is subjected to uniformly distributed loading (UDL) of 10.5 kN/m while "Beam 2" is subjected to UDL of 3 kN/m together with two concentrated loads of 258 kN applied at 0.3 m from end supports. This loading scheme generates same magnitude of peak bending moment at the critical section (mid-span), however shear force distribution along these beams would be different. These selected load levels represent about 50% of flexural and shear capacity at room temperature, which is similar to load levels encountered during fire conditions. The beams were designed as per AISC (2011) provisions and have room temperature flexural and shear capacity of 275 kN-m and of 584 kN, respectively. It should be noted that the loading on "Beam 2" was chosen to simulate a pure shearing state and this load set-up is similar to the one used by Basler et al. (1960) to study shear response of steel beams at room temperature.

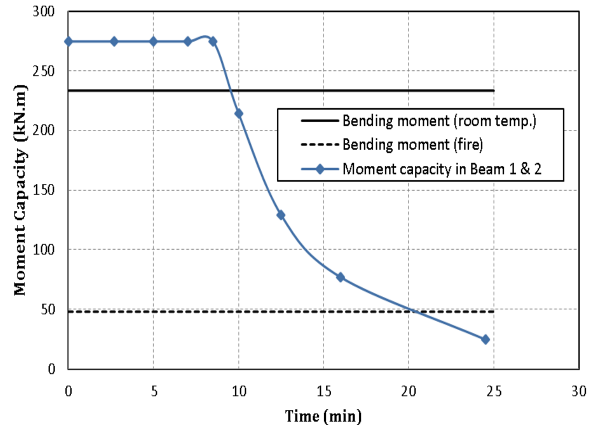
The above two beams were analyzed using the above developed model by subjecting them to combined loading and ASTM E119 standard fire exposure. Figure 5a shows temperature progression in the two beams with fire exposure time. Since these steel beams have same geometric and material properties and subjected to same ASTM E119 fire exposure, temperature rise in these beams is identical. It can be seen from Fig. 5a that steel temperature in web and bottom flange increases at a higher rate with fire exposure time as compared to that in top flange. Average temperatures in flanges and web reaches 427˚C and 500˚C at 10 min and 700˚C and 760˚C at 20 minutes into fire exposure. The different rate of temperature rise in flanges and web influences the rate at which moment and shear capacity degrade with fire exposure time.

At the start of fire exposure, Beams 1 and 2 have reserve moment capacity of 2.6 of that of the applied bending moment. However due to different applied loading patterns, Beams 1 and 2 have reserve shear capacity of 12.5 and 2.26 from that of applied shear force, respectively (as shown in Figs. 5b and c). It is clear that "Beam 1" has much higher reserve shear capacity than that of "Beam 2", hence these beams experience failure in different modes as explained below.

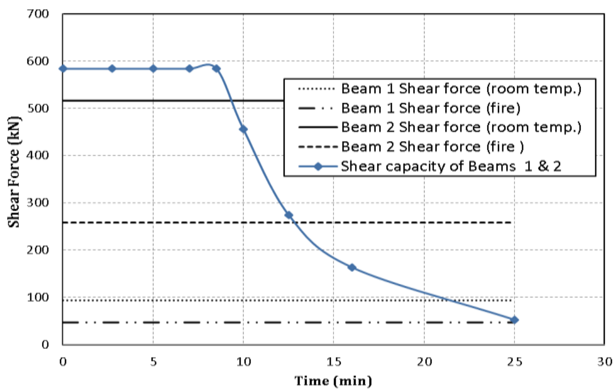
Figure 5b and 5c show the degradation of moment and shear capacity, along with bending moment and shear force generated due to applied loading. These plotted moment and shear capacities are at critical sections namely, mid-span for moment and location of point loading for shear force. Moment and shear capacity in both beams starts to degrade only after 9 minutes into fire exposure. This is due to temperature in lower flange and web crossing 400˚C at about 9 min as shown in Fig. 5a. As the temperature rise continues in web and flanges, further degradation of moment and shear capacity takes place until failure occurs in the beams. Furthermore, Fig. 5d shows the predicted mid-span deflection in these three beams as a function of fire exposure time. The mid-span deflections remain small for about 10 and 6 min in Beams 1 and 2, respectively. Then, deflections increase at a rapid pace leading to runaway failure in these two beams.



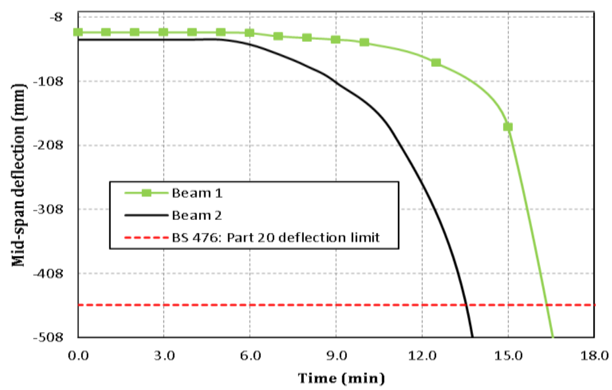
(a) Temperature in steel Beams 1 and 2



(b) Moment capacity degradation



(c) Shear capacity degradation



(d) Variation of mid-span deflection

Figure 5: Thermal and mechanical response of Beams 1 and 2 with fire exposure time

As shown in Fig. 5a, temperature rise in web is much higher than that in upper flange. Hence, strength loss in the web is higher than that in flanges. As the temperature rise continues in web, further degradation in strength contribution from web occurs. This degradation of strength once reaches reduced yield strength of steel, sectional instabilities sets in in the web proper to flanges. This instability can further decrease shear capacity and accelerate failure of beams via occurrence of web local buckling.

The above generated results were utilized to evaluate failure of beams under different limit states. Failure of the beam is said to occur when moment (or shear) capacity drops below applied bending moment (or shear force) or when mid-span deflection exceeds limiting deflection criterion. It is clear from Fig. 5b that “Beam 1” attains failure in flexural mode at about 14 min when the moment capacity drops below the applied moment due to loading (UDL). On the other hand, “Beam 2” fails in shear limiting state at 13 min, prior to onset flexural limiting state. It can be seen from Fig. 5d that runaway (large deflection) failure occurs at 17 and 14 min in Beams 1 and 2 due to significant degradation of stiffness resulting from temperatures in steel exceeding 550˚C. While “Beams 1” fails in flexural (moment) mode, “Beam 2” fails in shear limit state earlier to reaching deflection or flexural capacity limit states. Although the applied loading on these two beams resulted in similar bending moment, different loading pattern led to different shear response and failure modes. Thus, loading pattern can significantly affect the fire response of steel beams. Table 1 summarizes failure time in these beams. These results infer that a fire exposed beam under certain loading scenarios can fail through shear limiting state prior to attaining flexural or deflection limiting states. Additional studies based on different loading pattern can be found else were (Kodur and Naser 2013).

Table 1 Failure time of Beams 1 and 2

Beam	Loading	Failure time (min)			Failure mode
		Shear	Flexure	Deflection	
Beam 1	Uniformly distributed loading (UDL)	25	**14**	17	Flexure
Beam 2	Concentrated loading near supports + UDL	**13**	14	14	Shear

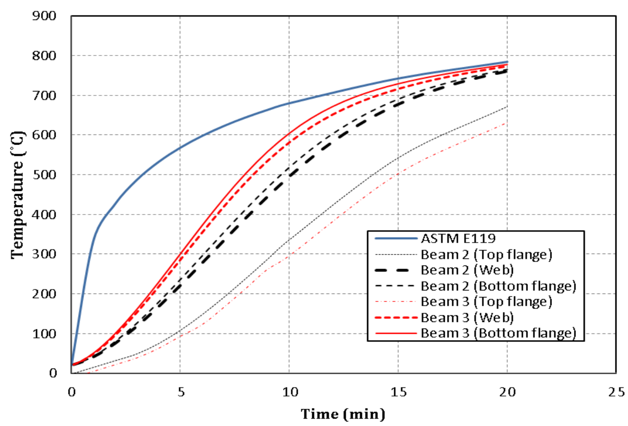
4.2 Effect of web slenderness

In typical steel beams, flanges mainly contribute to moment capacity, while web contributes to shear capacity. Thus, generally slenderness of web has significant influence on shear capacity of the beam. For optimum design, slenderness of web is much higher than that of flanges and hence web slenderness is a critical factor in determining shear capacity in a steel beam. The effect of web slenderness on shear capacity is studied by analyzing two fire exposed beams with varying web slenderness. “Beam 3” is a replicate of “Beam 2” shown above, but with different web thickness. Also, “Beam 3” is subjected to UDL of 3 kN/m together with two concentrated loads of 178 kN applied at 0.3 m from end supports. These two beams (Beams 2 and 3), subjected to ASTM E119 fire as well as gravity loading and were analyzed with the above developed model. Based on predicted response; failure of the beams is evaluated under different limit states.

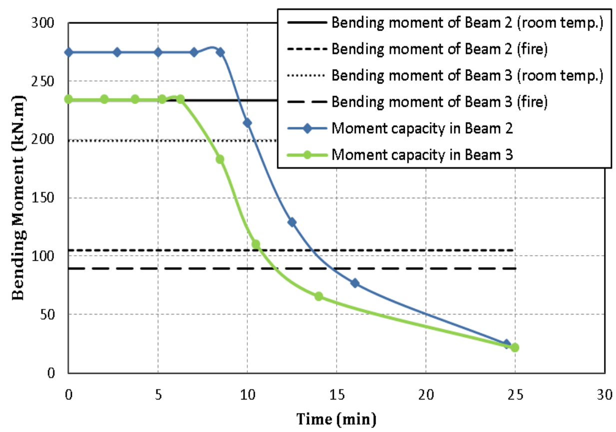
Both beams have similar flange slenderness ratio of 6.28, while web slenderness ratio for “Beam 2”and “Beam 3” are 57.82 and 100.13, respectively. To illustrate the effect of web slenderness on temperature rise, predicted temperature in the web of “Beam 2”and “Beam 3”are plotted in Fig. 6a as a function of fire exposure time. It can be seen that overall thermal response in “Beam 3” follows similar trend to that of “Beam 2” but temperature in web of “Beams 3” increases at a much faster pace due to slender web. Thus, faster degradation of strength and stiffness properties of web (and thus of beam) occurs in “Beam 3” as compared to that in “Beam 2”. In general, temperature rise in web of Beams 2 and 3 tend to be higher than average temperature of flanges. Hence, rapid temperature induced loss of strength in web (and shear capacity) of Beams 2 and 3 is expected as compared to gradual strength loss of flanges (moment capacity) in these two beams.

Both Beams 2 and 3 have reserve moment and shear capacity of 2.6 and 2.26 of that of the applied loading, as shown in Figs. 6b and c. Since reserve shear capacity is less than reserve moment capacity, these beams are likely to fail in shear. Figure 6b shows degradation of moment capacity with fire exposure time at the mid-span section of Beams 2 and 3. It should be noted that moment capacity at ambient conditions in these two beams are slightly different resulting from reduced web thickness; thus reduced plastic modulus. Figure 6c shows degradation of shear capacity as a function of fire exposure time. Since shear capacity is mainly governed by the size of the web, shear capacity at ambient conditions of "Beam 3" is much lower than that of "Beam 2" due to higher web slenderness (reduced web thickness). When exposed to fire, moment and shear capacity of Beams 2 and 3 start to degrade after about 9 and 6 min of fire exposure time, respectively. At this point, internal stress (due to applied loading) reaches reduced yield strength of steel, sectional instability occurs and also plastic deformation starts to accumulate. These deformations initiate sectional instability.

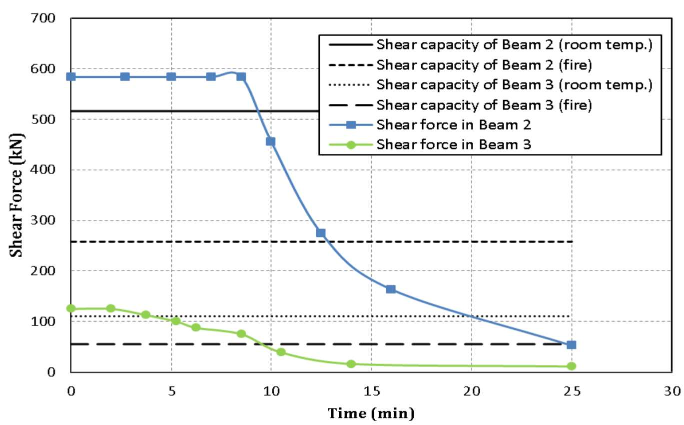
Figure 6d compares predicted mid-span deflection in Beams 2 and 3. In general, mid-span deflections are small in the initial stage of fire exposure and then increase gradually with fire exposure time. The deflections increase at a rapid pace towards final stage of exposure due to very high temperature in steel, and this lead to failure of beams. As expected "Beam 3", with higher web slenderness, undergo larger initial deflections as compared to that of "Beam 2".



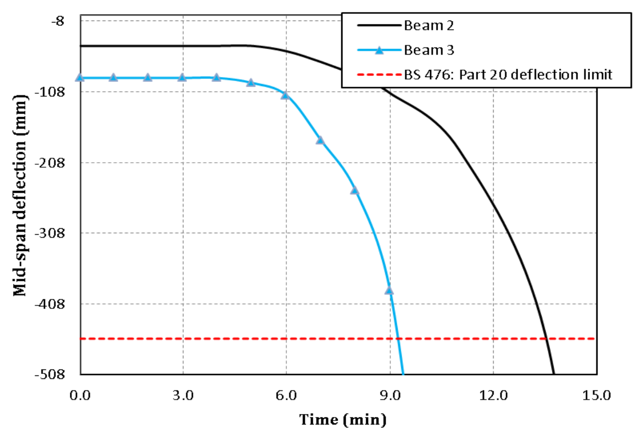
(a) Temperature in steel Beams 2 and 3



(b) Moment capacity degradation



(c) Shear capacity degradation



(d) Mid-span deflection

Figure 6: Thermal and mechanical response of Beams 2 and 3 with fire exposure time

Beam 3 experiences failure through flexural and shear limit states at 12 and 7 min, respectively as compared to 14 and 13 min in Beam 2. The corresponding mid-span deflection in Beams 2 and 3 (at failure) is 457 and 177 mm, respectively. This indicates that “Beam 3” fails at lower mid-span deflection and at earlier times as compared to that of “Beam 2”. Table 2 summarizes failure modes in these two beams analyzed with different web slenderness. These results clearly infer that web slenderness influences failure mode in fire exposed steel beams and can lead to shear failure prior to reaching flexural or deflection limit states.

Table 2 Failure in beams with different web slenderness

Beam	Web slenderness	Flange slenderness	Failure time (min)			Failure mode
			Shear	Flexure	Deflection	
Beam 2	57.82	6.28	**13**	14	14	Shear
Beam 3	100.13	6.28	**7**	12	9	Shear

It should be noted that there is only slight difference in failure times of these uninsulated steel beams. However, effect of different failure modes and corresponding failure times can be more apparent in fire insulated beams as discussed in Kodur and Naser (2013). Accordingly illustrated a steel girder insulated with 1 hr fire rated insulation system will fail in 65 and 55 min due to flexural and shear effects, respectively. Therefore, accounting for shear effects can significantly alter failure times of girders in certain situations. This can lead to unconservative fire resistance under certain scenarios.

5. Conclusions

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

1. The developed finite element model is capable of predicting fire response of steel beams where flexural or shear effects dominate the behavior of steel beams.
2. In a fire exposed steel beam, sectional instabilities can occur in web due to shear parameters prior to that in flange due to flexural parameters under certain loading and sectional configurations.
3. In fire exposed steel beams with higher slender webs, shear capacity can degrade at a higher pace than that of moment capacity. In such beams, failure can occur in shear limit state rather than flexural or deflection limit states.
4. Loading pattern, web slenderness and reserved shear capacity are found to influence the onset of sectional instability and degradation of shear resistance in fire exposed steel girders.

Acknowledgments

This material is based upon the work supported by the National Science Foundation under Grant number CMMI-1068621 to 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.

References

AISC. Specification for structural steel buildings 360–11. American Institute of Steel Construction Inc.; Chicago, IL 2011.

ANSYS. Finite element computer code. Version 14. Canonsburg (PA): ANSYS, Inc; 2011.

Aziz, E., Kodur, V. (2013). “An approach for evaluating the residual strength of fire exposed bridge girders.” *Journal of Constructional Steel Research* 88 34-42.

Basler, K., Yen, B., Mueller, J., Thürlimann, B. (1960). “Web Buckling Tests on Welded Plate Girders.” Welding Research Council Bulletin 64.

British Standard Institute. (1987). BS 476: fire tests on building materials and structures, part 21: methods for determination of the fire resistance of load-bearing elements of construction.

Dwaikat, M.M.S., Kodur, V.K.R. (2011). “A performance based methodology for fire design of restrained steel beams.” Journal of Constructional Steel Research 67 510-524.

European Committee for Standardization (CEN). EN 1993-1-2: 2005 Eurocode 3: design of steel structures. Part 1.2 General rules—structural fire design. Brussels: CEN; 2005.

Garlock, M., Paya-Zaforteza, I., Kodur, V.K.R., Gu, L. (2012). “Fire hazard in bridges: Review, assessment and repair strategies.” *Engineering Structures* 35 89-98.

Hall, W.J. (1954). “Shear deflection of wide flange steel beams in the plastic range.” Wright Air Development Center. Urbana, IL: University of Illinois.

Kodur, V.K.R, Dwaikat, M.M.S. (2010). “Effect of high temperature creep on fire response of restrained steel beams.” *Journal of Materials and Structures (RILEM)* 43 1327-1341.

Kodur, V.K.R., Fike, R.S. (2009). “Guidelines for improving the standard fire resistance test specifications.” *Journal of ASTM International* 2009 6 1-16.

Kodur V.K.R., Naser M.Z. (2013). “Effect of Shear on Fire Response of Steel Beams.” *Journal of Constructional Steel Research* under review.