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THERMAL RESPONSE AND CAPACITY OF BEAM END SHEAR CONNECTIONS DURING A LARGE COMPARTMENT FIRE EXPERIMENT

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ABSTRACT

The role of steel connections is essential in structural fire design and analysis for steel-framed composite structures. The current structural design provisions provide strength reduction factors of load-carrying members and their end-connection elements (e.g. bolts) at elevated temperatures, based on small-scale experiments under uniform heating conditions. The realistic temperature evolution in member connections, especially as part of full-scale floor assemblies exposed to a large compartment fire, has not been well characterized. A large compartment fire experiment was recently conducted on a 9.1 m by 6.1 m composite floor assembly as part of a two-story steel framed building. The test assembly had a total of ten shear-tab (fin-plate) connections subjected to combined fire and mechanical loading. This paper presents the measured thermal response of these connections in comparison with the corresponding Eurocode 3 predictions with two methods (1) incorporating the beam bottom flange temperature at midspan and (2) the section factor method. The results show that the Eurocode 3 methods conservatively predict the maximum temperature during heating and the cooling rate but overestimate the high-temperature strength of connections while using the section factor method. The predicted thermal responses are highly influenced by the fire protection sprayed on the connection region which was actually at least 43% thicker than the protection on the beams used in this test program. However, partial shear failure of bolts was witnessed in the test. This suggests that designing connections solely through temperature provisions may not guarantee a safe structural fire design. The axial load demand of the shear connection due to restraints to thermal elongation or contraction should be considered in future design guidance.

Keywords: Connections; fire test; Eurocodes; temperatures; capacity; shear tab

1 INTRODUCTION

In fire safety design of buildings, the structure is required to meet the desired performance objectives. At a minimum these include maintaining structural integrity such that compartmentalization is not compromised [1]. However, the accurate quantification of fire severity in a structure has been challenging due to lack of design tools validated against experimental data. Particularly, the thermal response of composite beam connections to a large compartment fire is one of such examples [2-4]. Currently in Eurocode 3 [5] and other similar provisions used in many other countries including the United States, the thermal gradient of the beam end connections is estimated using the empirical equations based upon the bottom flange temperature of composite floor beams. However, the bottom flange beam temperature utilized in these

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methods are assumed to be remote from the connection. The thermal gradient across the beam-end connections is then used to calculate the connection capacity incorporating strength reduction factors for bolts and welds. The reliability of those Eurocode 3 predictions is unknown especially for the fire-protected connections and full-scale typical beam sizes and spans within a large compartment fire [6, 7] due to lack of such experimental temperature data. Furthermore, even if the thermal response of the steel connection components was within the range predicted by Eurocode 3 methods, to what extent this will ensure a robust structural fire design is also uncertain, based on the lack of supporting experimental data and analysis.

The work presented in this paper aims to: (1) enrich the experimental data library for thermal and structural response of the shear-tab (fin-plate) connections with fire protection subjected to a full-scale large compartment fire; (2) examine the Eurocode 3 connection design (temperature and strength) utilizing experimental data [8] with a large compartment fire; and (3) identify gaps in knowledge or data for structural design of shear connections under fire conditions.

2 FIRE TEST

2.1 The test building and fire compartment

A two-story steel frame with composite floors has three by two bays in plan (18 m × 11 m) with a total building height of 7.2 m, and the fire was in a large compartment at the south-central bay having dimensions of 9.1 m × 6.1 m with a 3.8 m ceiling height (Figure 1). The composite floor assemblies were designed to resist an ambient design gravity load of 8.6 kPa. The test floor assembly was subjected to a floor load of 5.3 kPa following the ASCE 7 [9] load combination for extraordinary events (1.2 × dead load + 0.5 × live load).

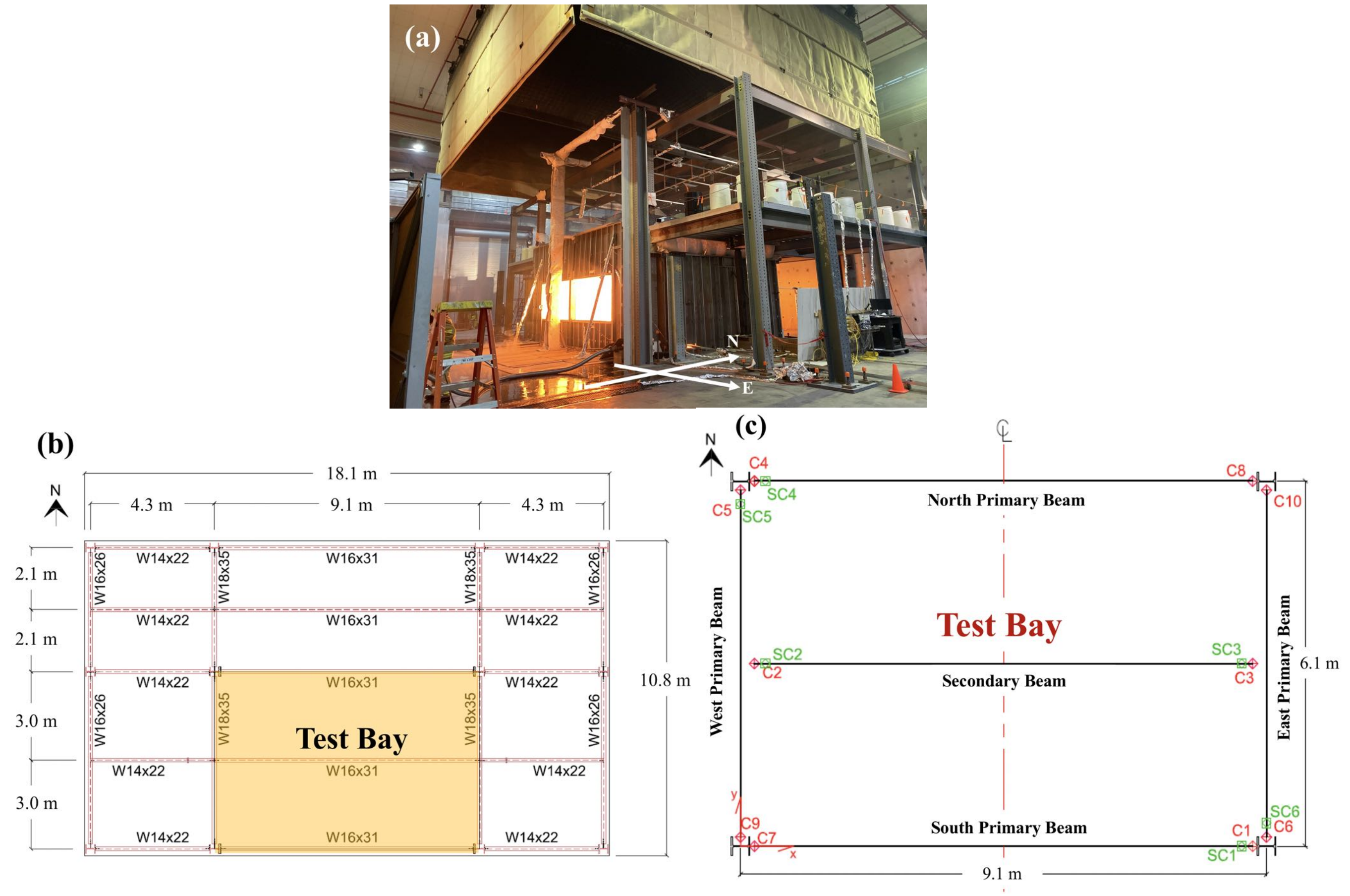

Figure 1. NIST large compartment fire experiment [8] on a steel-composite building. (a). Photo taken during the experiment; (b). Plan view of the test building and test bay location; and (c). Plan view of composite connection instrumentation locations in the experimental compartment, ten in total, from connection 1-10 (abbrev. as C1-C10).

For passive fire protection of exposed steel members, a medium density (ranging from 240 kg/m^3 to 350 kg/m^3 [10]) gypsum-based cementitious material, was sprayed on beams and connections exposed to fire. The design thickness of insulation on both south and north primary W16×31 beams, as well as on the west and east primary W18×35 beams inside the fire test compartment was 17.5 mm (11/16 inch) determined using the Underwriter Laboratory (UL) directory N791 [11] for the 2-hour restrained beam rating. Insulation thickness of the secondary W16×31 beam was 13 mm (7/16 inch), slightly thinner than the primary beams, determined using UL D949 [12] for the 2-hour restrained assembly rating. The exposed connection regions and columns were over sprayed with the same insulation material with the thickness of 25 mm or greater, i.e. 43% thicker than the primary beams, to ensure the 3-hour rating of columns.

2.2 The connections and relevant instrumentations

Two types of simple shear connections were used in this test program: standard shear tabs for the beam-to-column flange and beam-to-beam web connections; extended shear tabs for the beam-to-column web connections. All shear tabs were 9.5 mm in thickness and made of ASTM A36 [13] steel (the minimum yield stress of 245 MPa). The size of fillet welds was 6.3 mm. All structural bolts (Gr. A325 specified in the ASTM F3125 [14]) had a diameter of 19 mm. The dimensions of the short-slot holes (21 mm in width and 25 mm in length) drilled on the shear tabs conform to the AISC 360 specification [1]. Examples of the connections and mounted thermocouples[5] are demonstrated in Figures 2(a) and 2(b).

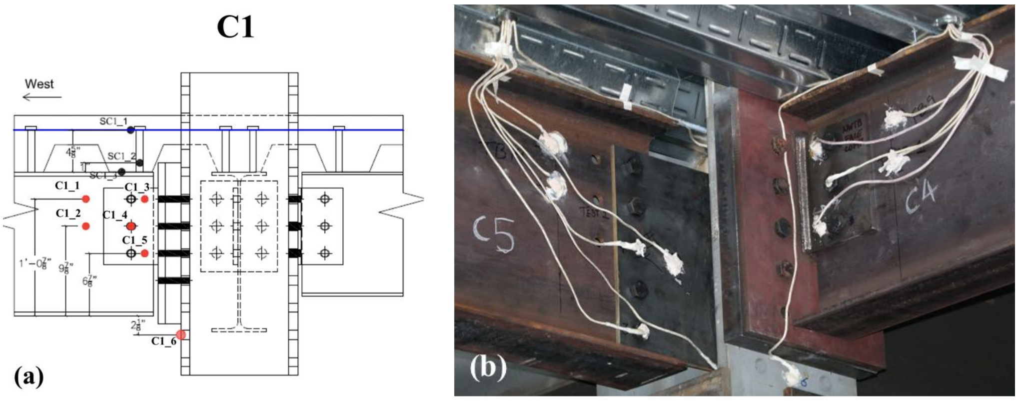

Figure 2. (a). Thermocouple instrumentation locations at connection C1, tagged as C1_1 to C1_6; and (b). Thermocouple instrumentation locations at connection C4 (standard shear tab) and connection C5 (extended shear tab), prior to SFRM installation.

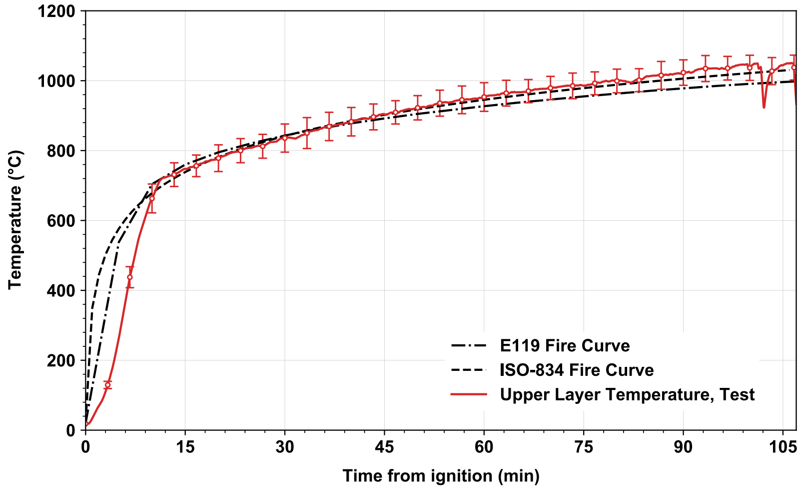

Figure 3. A comparison between measured[6] upper layer gas temperature within the test compartment, and standard fire curves (i.e. the ASTM E119 fire curve, and the ISO-834 fire curve)

[5] An expanded uncertainty of thermocouple locations is estimated to be ± 6 mm with a coverage factor of 2 (95% confidence interval)

[6] An expanded uncertainty of measured gas temperatures is estimated to be ± 8% at 1100 °C with a coverage factor of 2

2.3 Mechanical and fire loading

The vertical shear load imposed on the connections was in the range of 0.2 to 0.4 of their ambient design capacities during the fire experiment. The fire load (or exposure) was applied using natural gas burners [15-17] following the ASTM E119 temperature-time curve [18] lasting 107 mins equivalent to 921 MJ/m^2 with ± 1.5 MJ/m^2 uncertainty (95 % confidence interval) as the applied fire load density. At 107 min the heat release rate reached to its maximum value of 10.8 MW. As demonstrated in Figure 3, the test revealed that the measured time-temperature curve matched better with the ISO Standard Fire curve [19] (which is more severe than the ASTM E119 curve) after 45 mins. The maximum standard deviation of the upper layer gas temperature was around 70 °C throughout the heating regime. This standard fire environment was to ensure that the whole structural assembly was being challenged to a severe fire impact and for the research interest of steel connections. This impact included a large compressive force induced by the restraint to thermal expansion during the heating phase, a tensile force due to catenary action during heating, and a tensile force by the restraint to thermal contraction during cooling.

2.4 Gas temperature and steel temperatures

An example of the temperature comparison is presented in Figure 4, including upper layer gas temperature and temperatures of the west primary beam bottom flange (average of thermocouple TBi_5 and TBi_6) and connection C5. This comparison considers seven hours of testing duration including the natural cooling phase. After the burner was switched off at 107 min, the upper layer gas temperature decreased sharply from 1040 °C to 490 °C within approximately 10 mins into cooling. This drastic temperature decrease would likely induce large tension forces on the connections, due to the restraint to thermal contraction of the steel beam during the cooling stage.

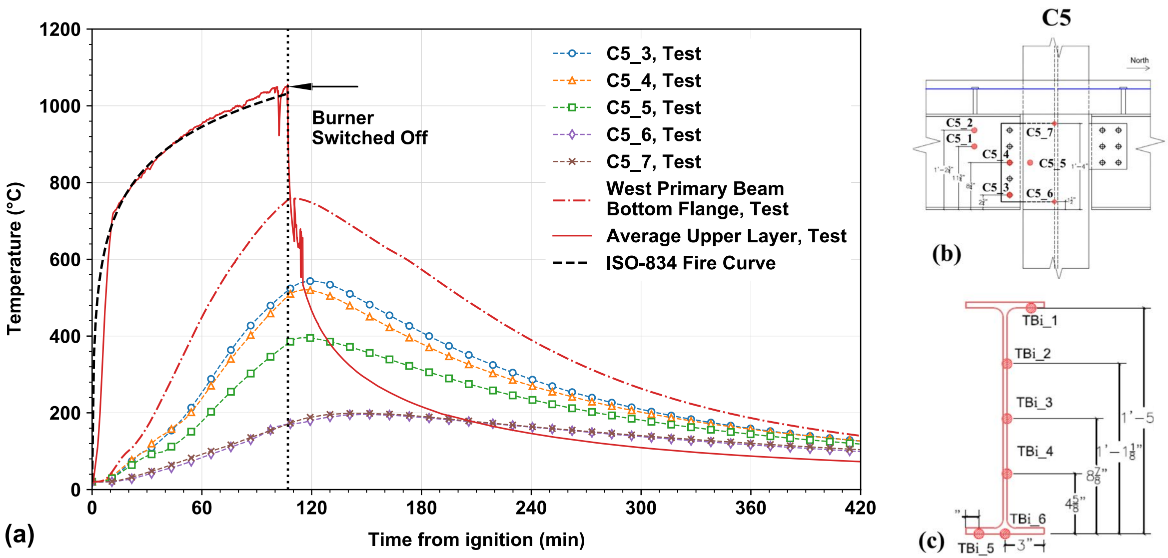

Figure 4. (a). Comparison of the time temperature curves for the compartment upper layer gas, west primary beam bottom flange, and steel connection[7] C5; (b). Thermocouple instrumentation locations at connection C5, tagged as C5_1 to C5_7; and (c). Thermocouple instrumentation locations at west primary beam midspan, tagged as TBi_1 to TBi_6.

The bottom flange temperature of the west primary beam reached its measured maximum value of 760 °C and cooled off gradually due to the presence of the fire protection. The connection temperatures followed a similar tendency but with relatively lower maximum values (e.g. 540 °C at C5_3) and a time delay to their peaks, provided that thicker SFRM was applied. The test showed that C5_3 and C5_4 (bolts) indicated higher component temperatures, compared to the measured temperatures at C5_6 and C5_7 (welds). This is believed to be because C5_6 and C5_7 were affected by the thermal shadow effect or the heat conduction loss to the connected column at this region. Those relative relationships of temperatures were further evaluated using Eurocode 3 methods, as detailed in the subsequent section.

[7] An expanded uncertainty of measured steel temperatures is estimated to be ± 4% at 970 °C with a coverage factor of 2

3 CONNECTION TEMPERATURES

3.1 Eurocode 3 method

According to the Eurocode 3, for beam-to-beam and beam-to-column connections where concrete floors are atop the beams, temperatures of the connections can be estimated based upon the bottom flange temperature of the connected steel beam at midspan. Considering the depths of the steel members used in the experiment (i.e. W16×31 and W18×35), are both greater than 400 mm, hence two equations are used:

when h is less or equal than $D/2$:

$$\theta_h = 0.88\theta_o \qquad (1)$$

when h is greater than $D/2$:

$$\theta_h = 0.88\theta_o \left[1 + 0.2(1-2h/D)\right] \qquad (2)$$

where θ_h is the temperature at height h (mm) of the steel beam, see Figure 5;

θ_o is the bottom flange temperature of the steel beam remote from the connection;

h is the height of the component being considered above the bottom of the beam in (mm);

D is the depth of the beam in (mm).

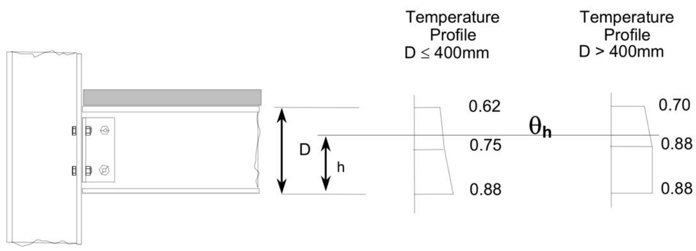

Figure 5. Thermal gradient within the depth of a composite connection (figure adapted from Eurocode 3 [5]).

If the gas temperature is known, the Eurocode 3 step-by-step section factor method can be used to estimate the steel beam bottom flange temperature θ_o as follows:

$$\Delta\theta_{a,t} = \frac{\lambda_p \, A_p/V(\theta_{g,t} - \theta_{a,t})}{d_p c_a \rho_a \, (1 + \emptyset/3)} \, \Delta_t - (e^{\emptyset/10} - 1)\Delta\theta_{g,t} \qquad (3)$$

$$\emptyset = \frac{c_p \rho_p}{c_a \rho_a} \, d_p \, A_p/V \qquad (4)$$

where A_p/V is the section factor for steel members insulated by fire protection material in (m^{-1});

c_a is the temperature dependant specific heat of steel in (J/kgK);

c_p is the temperature independent specific heat of the fire protection material in (J/kgK);

d_p is the thickness of the fire protection material in (m);

λ_p is the thermal conductivity of the fire protection in (W/mK);

ρ_a is the unit mass of steel in (kg/m^3);

ρ_p is the unit mass of the fire protection in (kg/m^3);

$\theta_{a,t}$ is the steel temperature at time t in (^{o}C);

$\theta_{g,t}$ is the ambient gas temperature at time t in (^{o}C);

Δ_t is the time interval in (seconds);

As detailed above, theoretically two analytical methods can be used to estimate temperatures of connection components: if the steel beam bottom flange temperature at midspan is known, then equations (1) and (2) can be employed directly; if the gas temperature within the compartment is known, then equation (3) can be used to estimate the steel beam temperature which is a main variable of equations (1) and (2). The following section will examine those two methods, utilizing the experimental data to evaluate the applicability of the Eurocode 3 provisions.

3.2 Comparison between the measurements and Eurocode 3 predictions - method 1

Following the Eurocode 3 convention, see Figure 5, dimensionless thermal gradients of all the test connections are summarized in Figure 6. These gradients were estimated when the connected structural beam members reached to their maximum deflections, at around 107 min. At this time, as summarized in Table 1, temperatures of the end connections of the secondary and primary W16×31 beams were in excess of 500 °C, whereas the end connections of the primary W18×35 beams were heated below 400 °C. The temperature discrepancy between the primary and secondary beam-end connections is mainly due to the difference in applied SFRM thickness, see Table 1. As shown in Figure 5, for components of the standard shear tabs connecting W16×31 beams, the dimensionless experimental temperatures vary from 0.35 to 0.7, less than the values calculated using the Eurocode 3 (0.7 to 0.88). For those of the extended shear tabs at the ends of W18×35 beams, moreover, the measured dimensionless temperatures range from 0.5 to 0.86, which remain below 0.88 calculated using equations (1) and (2). This comparison demonstrates that the Eurocode 3 provisions, estimating temperatures of the standard shear tabs as a function of the beam bottom flange temperatures at midspan, are conservative. It is also anticipated that under a natural fire (rather than in a standard fire), the temperature difference between the connections and the beam flange at midspan is greater. If a natural fire is fuelled by array of wood cribs, e.g. [3], the upper layer gas temperature is expected to be less uniform and highly influenced by other factors (e.g. distribution of fuel, ventilation, wood properties) when compared with the conditions in which the test fire is controlled by natural gas burners.

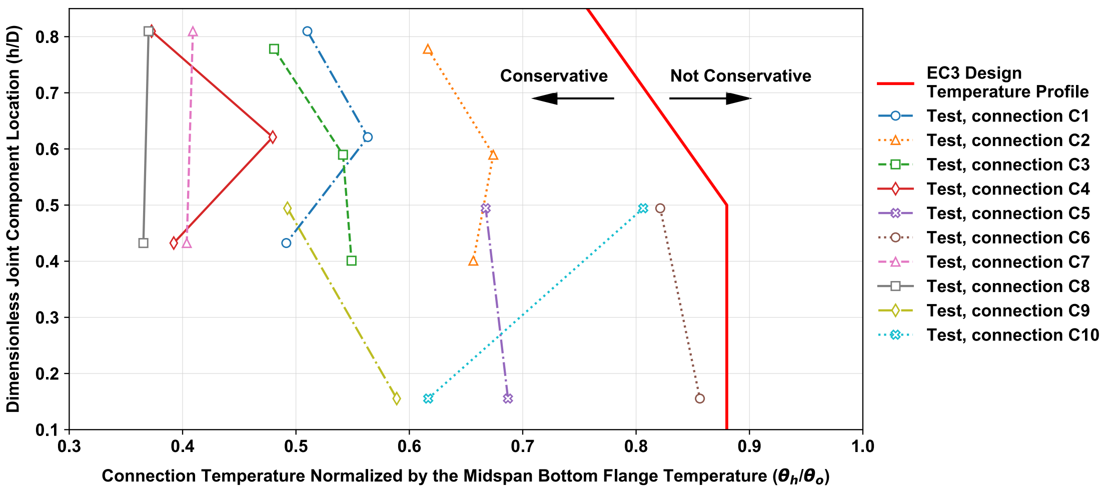

Figure 6. Thermal gradient within the depth of the composite connections, test vs. Eurocode 3

Table 1. Summary of the connections: connected members, types, SFRM thickness (± represented as standard deviation), and measured max component temperature.

Connection Tag	**C1**	**C2**	**C3**	**C4**	**C5**	**C6**	**C7**	**C8**	**C9**	**C10**
Connected Members	primary beam - column flange	secondary beam - primary beam	secondary beam - primary beam	primary beam - column flange	primary beam - column web	primary beam - column web	primary beam - column flange	primary beam - column flange	primary beam - column web	primary beam - column web
Member Dimension	W16×31	W16×31	W16×31	W16×31	W18×35	W18×35	W16×31	W16×31	W18×35	W18×35
Shear Tab Type	Standard	Standard	Standard	Standard	Extended	Extended	Standard	Standard	Extended	Extended
Fire Protection Thickness (mm)	28 ± 2	31 ± 3	29 ± 3	26 ± 2	27 ± 3	25 ± 1	29 ± 6	28 ± 4	24 ± 3	25 ± 2
Maximum Temperature (°C)	400	660	570	380	530	590	300	310	500	580

3.3 Comparison between the measurements and Eurocode 3 predictions - method 2

An example of comparison between the measured and predicted temperatures based upon the step-by-step section factor method (method 2) is presented in Figure 7. In this example, the connection components C1_4 and C4_4 were situated at the diagonal locations of the test compartment.

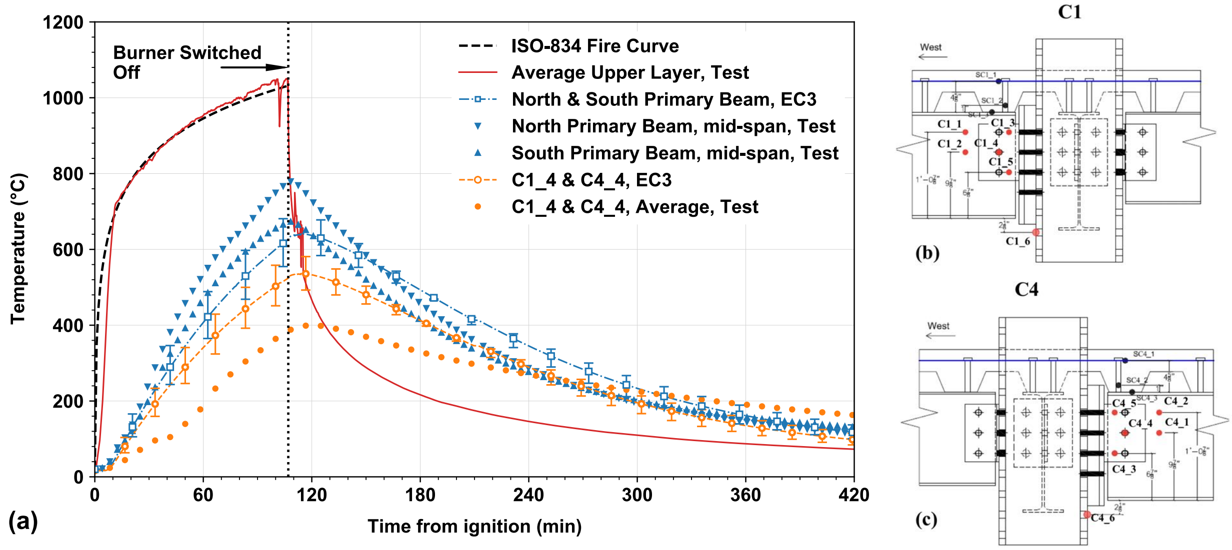

Figure 7. (a). Comparison between predicted temperatures with the measured temperatures: north and south primary beams at midspan, and steel bolt temperatures at same height C1_4 & C4_4; (b). Thermocouple instrumentation locations at connection C1, tagged as C1_1 to C1_6; and (c). Thermocouple instrumentation locations at connection C4, tagged as C4_1 to C4_6.

The measured upper layer gas temperature, $\theta_{g,t}$, is used as the input variable of equation (3) with a 5 s time interval for calculation. Note that the measured gas temperature during the heating phase of the experiment, closely matching with the ISO-834 standard fire curve, provides a good benchmark to examine the applicability of Eurocode 3 since standard fire curves (e.g. the ISO-834 fire curve) are a common fire situation considered for design. In addition, this study utilized the steel member density ρ_a of 7850 kg/m^3, and the temperature-dependent specific heat of the steel c_a according to Eurocode 3. The section factor of the W16×31 shape, A_p/V, was taken to be equal to 203 m^{-1} with a three-sided fire exposure. The SFRM thermal conductivity λ_p is 0.086 W/mK [20], and its specific heat c_p is assumed to be 1200 J/kgK [21]. There are also two SFRM-related parameters used for estimating uncertainties, including the unit mass, ρ_p, ranging from 240 kg/m^3 to 350 kg/m^3; and the applied thickness d_p. The measured value of d_p was 18 mm on average for the north primary beam, with 3 mm standard deviation and was 19 mm for the south primary beam with 2 mm standard deviation. The average temperatures of the north and south primary beams reached a maximum value of 780 °C and 670 °C, respectively, approximately 2 mins after the fire was extinguished (i.e., at 109 min). The Eurocode 3 prediction, incorporating the step-by-step method, on these primary beams suggests a maximum value ranging from[8] 580 °C to 690 °C at 115 min. The steel beam temperature predicted using the same method (equation (3)) appears to be lower than the corresponding values of measured temperatures, and yet the connection temperatures of the components C1_4 and C4_4 are conservative when calculated using equations (1) and (2). The predicted maximum temperature of those two connection components is approximately 540 °C, i.e. higher than their measured value of 400 °C on average.

The step-by-step section factor method using equation (3) is a preliminary step for calculating the steel connection component temperatures using equation (1) and (2). Therefore, it is worth investigating the accuracy of the section factor method in a more extensive manner, as presented in Figure 8 and further summarized in Table 2. For all steel beams, with an exception of the east primary beam, the predicted maximum temperatures of steel members are approximately 6% lower than the measured values on average.

[8] This range is due to the SFRM input property uncertainties: unit mass and thickness.

Furthermore, the average cooling rate[9] of all beams in the experiment is 110 °C/hour, about 9% greater than the predicted rate. One possible reason for the discrepancy between the prediction and the measurement, is due to the fire conditions achieved during the experiment. In the experiment, natural gas was used as the fuel which seldom generated smoke, applying cumulative radiation to the surface of the passive fire protection; however, in a real building fire, this situation is highly unlikely. The sooty smoke within the compartment upper layer, generated from a real fire, would obscure some of the radiation from the flames to the fire protection.

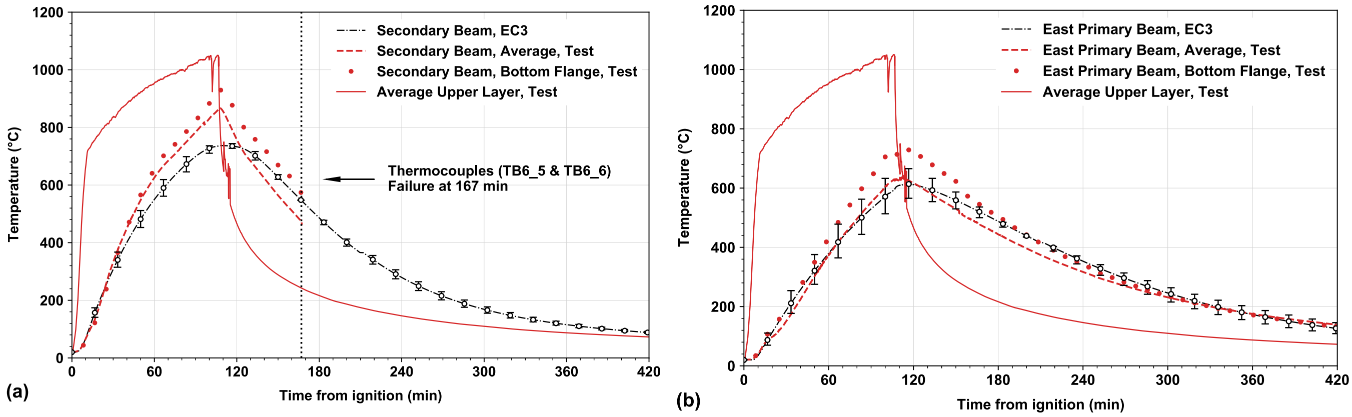

Figure 8. Comparison of the predicted with measured temperatures: (a). Secondary beam (thermocouples at the bottom flange of beam midspan, TB6_5 and TB6_6 failed at 167 min); and (b). East primary beam.

Table 2. Comparison between the test and the Eurocode 3 prediction on steel beam temperatures (for maximum temperature, Eurocode 3 method considers the upper bound prediction for comparison except for east primary beam; for cooling rate, Eurocode 3 method considers the mean value)

Beam at midspan	Maximum Temperature (°C)		Cooling Rate (°C/hour)	
	Test (Average)	Eurocode 3	Test (Average)	Eurocode 3
Secondary Beam	870	750	/	120
South Primary Beam	670	650	105	95
North Primary Beam	780	690	130	100
West Primary Beam	690	650	105	90
East Primary Beam	640	560 - 660	100	90
Average All Beams	730	680	110	100

Figure 9(a) and (b) present the predicted and measured values of maximum temperatures as well as cooling rates respectively, for all the measured connection components on the ten connections considered in this study. In Figure 9(a), the error bar of the test measured maximum temperature represents the standard deviation of two connection components on the same beam at two different ends; and the error bar of the Eurocode 3 predicted maximum temperature refers to the temperature variation due to the range of SFRM input variables, i.e. unit mass and thickness. Figure 9(a) suggests that Eurocode 3 tends to overpredict maximum temperatures of the connection components when actually heated to 400 °C or lower. However, the predicted temperatures (using Eurocode 3) become comparable to the measured temperatures of the connection components when actually heated in excess of 400 °C. It is important to repeat herein that this comparison is made under the situation where the SFRM thickness on the connection region was at least 43% thicker than the SFRM on the beams. To further examine the impact of SFRM thickness varying between the beam midspan and the connection region, Figure 9(a) also includes the comparison with

[9] The average cooling rate is calculated from the time of steel member at peak temperature, lasting five hours during the cooling phase.

another data from the long-span composite beam fire test carried out at NIST [22]. In this test, the SFRM thickness on the connection region was at least 68% greater than that on the steel beam. For this case, Eurocode 3 method overestimates the connection temperatures of which measured values were actually lower than 300 °C. Figure 9(b) explores the comparison between the measured and predicted cooling rates. It suggests that in most cases Eurocode 3 predicts much rapid cooling rates, from 70 °C/hour to 120 °C/hour for the connections used in this study, whereas the measured cooling rates significantly vary from 10 °C/hour to 110 °C/hour. This difference would be influenced by several factors, including the Eurocode 3 overestimation on maximum temperatures leading to a higher slope, the thicker SFRM applied on the connection region resulting in slow cool-down, or both.

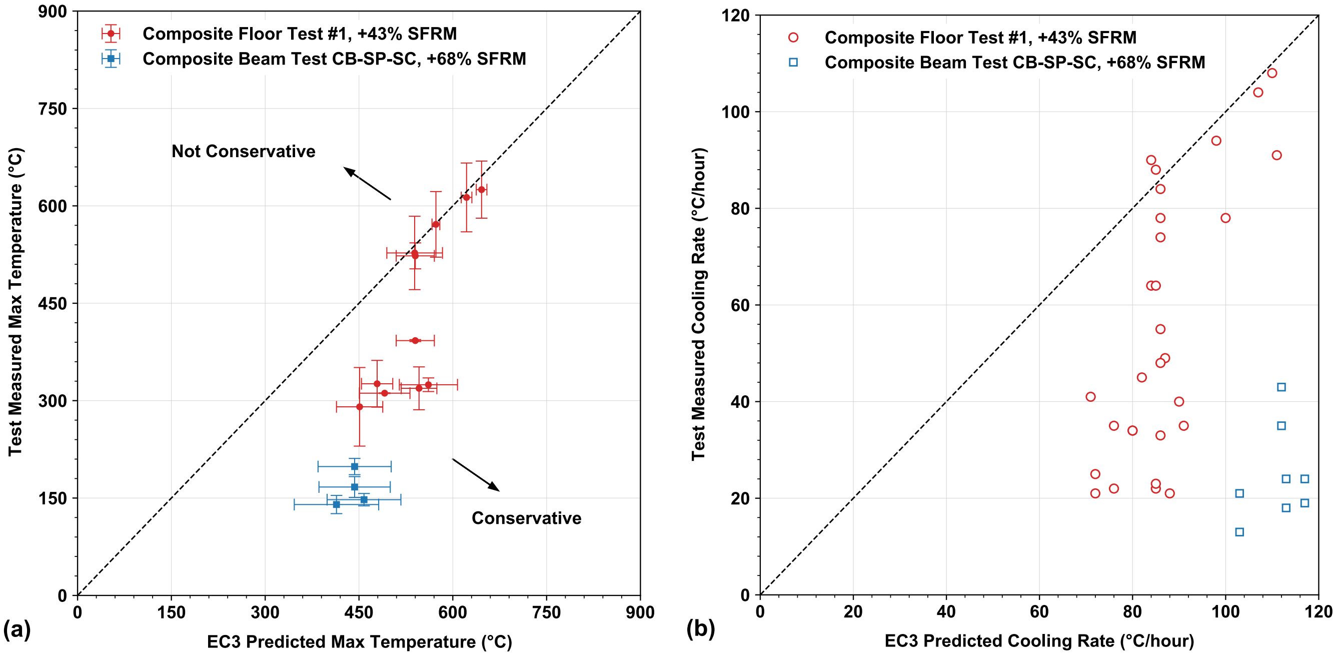

Figure 9. Comparison of the measured against Eurocode 3 predicted temperatures on all ten connections, i.e. 30 connection components in total plus another 8 connection components from composite beam test CB-SP-SC [22]: (a). Maximum temperature; and (b). Cooling rate within five hours.

4 CAPACITY OF THE CONNECTIONS

The shear capacity of welds and bolts used in the connection C2 was estimated using the Eurocode 3 reduction factors and experimentally measured temperatures. Figure 10(a) demonstrates that C2 would have failed at 575 °C (around 100 mins after the gas burner ignition). However, this behaviour was not witnessed during the experiment, see Figure 10(b) and (c).

Figure 11 illustrates the overestimation of connection temperature C2 predicted using the Eurocode 3 bolt strength reduction factor[10]. As shown, at 90 min, the bolt reduction factor decreases to as low as 0.11 when calculated using the measured bottom flange temperature of the steel beam at midspan or ranges from 0.20 to 0.31 when the Eurocode 3 step-by-step section factor method is used. Those two predictions are conservative, as compared to the actual 0.46 estimated via the temperature measurement at bolt C2_4.

However, in the case of C6 connection, the Eurocode 3 prediction of bolt strength reduction factor is less conservative, see Figure 12 at 90 min. This bolt reduction factor (based on the thermocouple measurement at this bolt) decreases to 0.53. This result is within the predictive range of 0.48 to 0.74 based upon the Eurocode 3 step-by-step section factor method but higher than the reduction factor 0.35 predicted using the measured beam bottom flange temperature at midspan. Although these predictions still imply no bolt failures, the post-fire inspection of the experiment discovered that three of five bolts from the connection C6 middle row to lower row experienced partial shear rupture failure, Figure 13. This was due to the

[10] Reduction factor: a ratio (≤1) between the steel bolt strength at high temperature and the strength at ambient temperature

combined large axial force and bending moment, induced by the thermal restraint and thermal bowing effects during the heating phase.

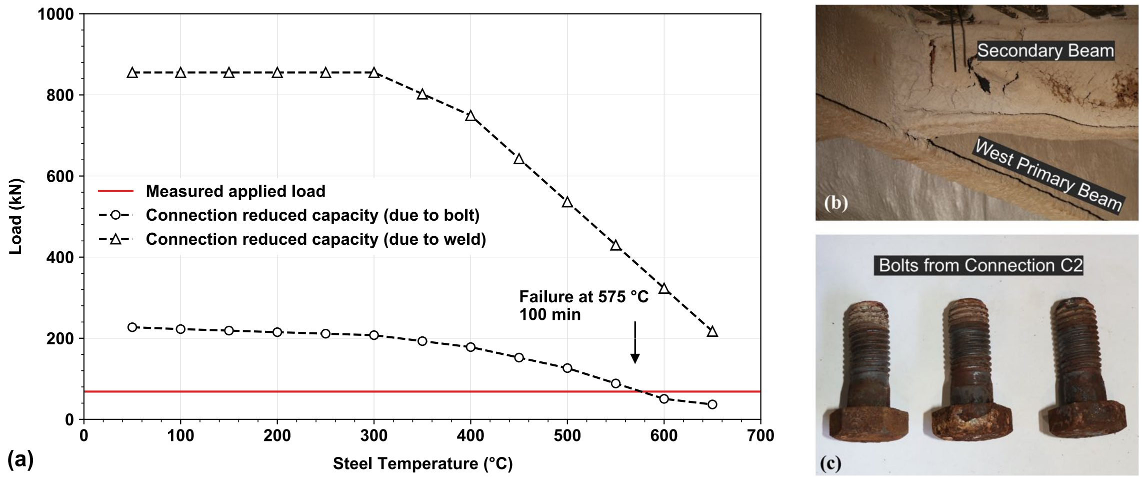

Figure 10. (a). Reduction of connection capacity with experimental increasing temperature at connection C2, against the corresponding measured applied load; (b). Post-fire inspection on connection C2 (before SFRM removed); and (c). Post-fire inspection on the bolts from connection C2.

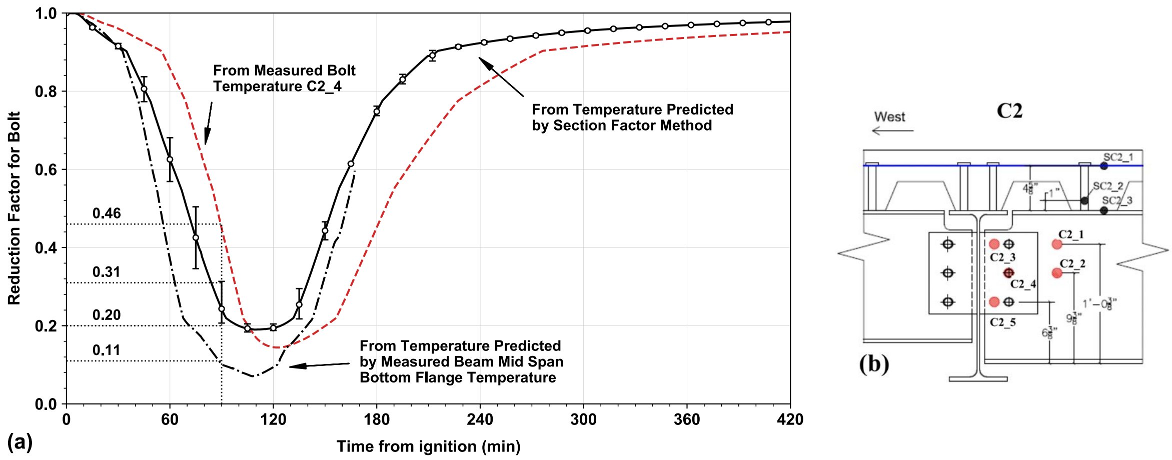

Figure 11. (a). Reduction factor of the bolt based on different methods and measured temperature at the C2 middle row, C2_4 (beam thermocouples at bottom flange failed at 167 min); and (b). Thermocouple instrumentation locations at connection C2, tagged as C2_1 to C2_5.

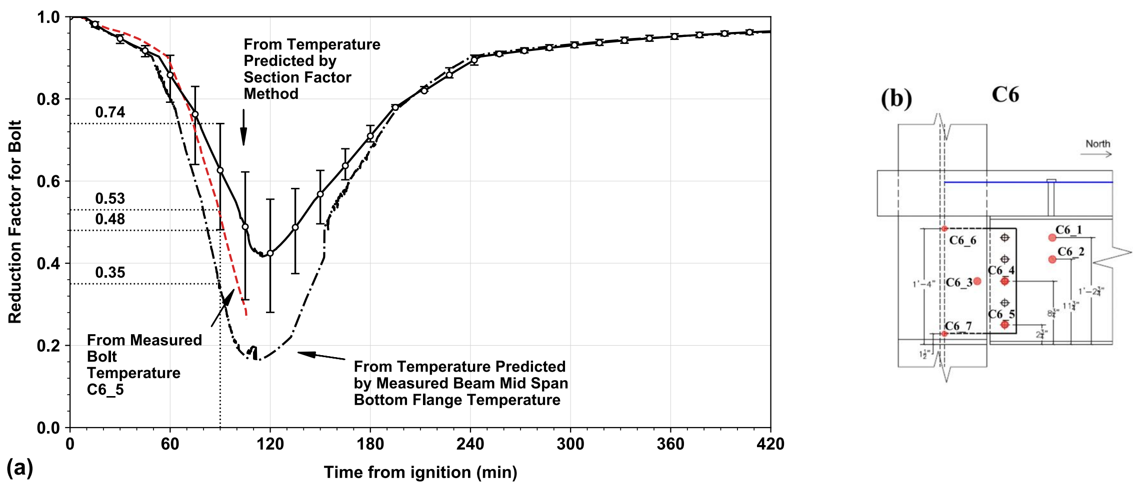

Figure 12. (a). Reduction factor of the bolt based on Eurocode 3 methods and measured temperature at the C6 lower row, C6_5 (bolt thermocouple failed at 106 min); and (b). Thermocouple instrumentation locations at connection C6, tagged as C6_1 to C6_7.

Figure 13. Post-fire inspection on the bolts from connection C6.

The findings in this section shows that Eurocode 3 method does not account for (1) additional sources constituting the capacity of a connection, such as the presence of slab and slab continuity to adjacent bays and (2) additional sources for the demand, such as thermally-induced axial forces due to the restraint to thermal expansion or contraction as well as catenary action. All these factors are needed to be incorporated for reliable estimation of the connection integrity.

5 CONCLUSIONS

The experimental results presented in this paper enrich the database that can be used for validation of computational models predicting beam end shear connections with fire protection under large compartment fires. The data acquisition of the measurements was successful, only 3 out of 45 thermocouple measurements on the total ten connection regions failed during the 7 hours (including cooling phase) of this structural fire test duration. This work demonstrates that the Eurocode 3 provision on the temperature prediction of connection components is conservative, provided that the fire protection at the connection region is at least 43% thicker than the protection on the beams. Finally, designing shear connections through temperature provisions may not guarantee a safe structural fire design. It is strongly recommended that the influence of the axial load demand (i.e. compressive/tensile load demands) of the connection *must also* be taken into account in future design guidance. This test indicated that the connection region can be subjected to varying axial forces during a fire event, such as a compressive force induced by the restraint of thermal expansion in the heating phase, followed by a tension force by catenary action and the contraction of beam members during cooling. The combined effects from high temperatures and fire-induced forces can lead to failure of connections designed only through Eurocode temperature provisions.

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DISCLAIMER

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