A Comparison between the Single Plate and Angle Shear Connection Performance under Fire

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ABSTRACT

The strength and stability of connections in a floor system is an integral part of a building structure. A connection is subjected to large compressive and tensile forces during heating and cooling phase of a fire, respectively. Since shear connections are only designed for gravity loads that produce shear, their behavior in a floor assembly at elevated temperatures needs to be investigated. This paper compares the behavior of three types of shear connections (single plate, single angle and double angle) under fire conditions using the finite element software ABAQUS. The single plate shear connection was validated by a full-scale building fire tested in Cardington. Adopting Eurocode and AISC provisions on the shear connection design, the Cardington connection was redesigned using the single and double angles. While the single plate connections can provide substantial rotational ductility and tensile strength, it could fail during cooling phase of a fire by bolt-hole bearing or bolt shear. The bolted double angle connections are generally more ductile in tension which is advantageous during cooling phase; however they are prone to develop prying forces which could cause the failure of the bolts. In all of the connection models, the beam near the connection experiences local buckling at elevated temperatures.

INTRODUCTION

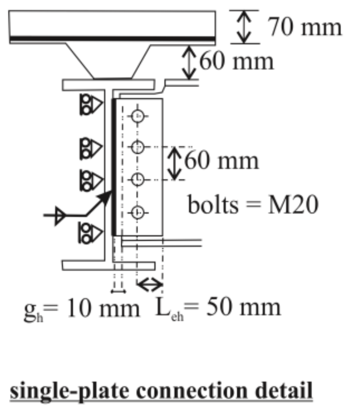
Structural floor systems undergo significant geometric changes due to thermal expansion of steel during a fire event. Additionally, the strength and stiffness of steel reduce dramatically at elevated temperatures. Hence, connections become an integral part of building construction because they could be subjected to large axial forces, moments and strain reversal during cooling period of a fire [Ramli-Sulong et al 2007]. Current design codes [AISC 2005 and ECCS 2001] are based on isolated member tests subjected to standard fire conditions. Such tests do not reflect the behaviour of a complete building under either normal temperature or fire conditions [Wald F. et al. 2006]. The complex interaction of structural components such as angles, plates, bolts and supporting members can be accurately captured by conducting large scale experiments or utilizing three-dimensional finite element models. The finite element modeling of the three different types of shear connections (single angle, double angle and single plate) under the same boundary conditions, loading and natural fire scenarios enables us to compare the strength, ductility and failure modes of these connections and their effect on the global behavior of the floor subassembly. This paper uses the finite element software ABAQUS [Simulia 2008]. The single plate shear connection was validated by a full-scale building fire tested in Cardington [Garlock and Selamet 2010; Wald, F. et al 2006]. Adopting AISC provisions on the shear connection design at ambient temperature, the Cardington connection was redesigned using the single and double angles.

Previously, Ramli-Sulong et al. [2007] used the component based method to estimate the axial force and moment capacity of double angle connections. However, the method was limited in capturing the ductility of the connection. Liu et al. [2002] conducted several furnace experiments with axially restrained unprotected beam with a double angle connection. He found that there were clear indications that the bolt holes in the beam web had been elongated as a result of the large bearing forces. The connections, although designed to carry only shear, developed significant moment because of the high rotations and the contact between the lower beam flange and the column. The combination of high connection moment and axial compression force causes lower flange buckling in the beam near the connection. The connection then loses its moment resistance. This paper aims to observe and compare the behavior of the three types of shear connections under a natural fire, which has not been studied previously.

SINGLE PLATE CONNECTION IN A SUBASSEMBLY

Geometry

We previously modeled a subassembly of the Cardington building test using the finite element software ABAQUS. The geometric details are given in Figure 1. The connection between the secondary beam and the girder is a single plate connection, which is designed according to Eurocode provisions [ECCS 2001] at ambient temperature. We modeled only half of the bay and used necessary boundary conditions and loading for symmetry. Further, the thermal and structural effect of the composite slab in the experiment is taken into account by applying a three-sided fire boundary on the beam and by using *Connector elements in ABAQUS, respectively. The use of *Connector elements to represent the additional flexural stiffness provided by the slab is explained thoroughly in Garlock and Selamet [2010].



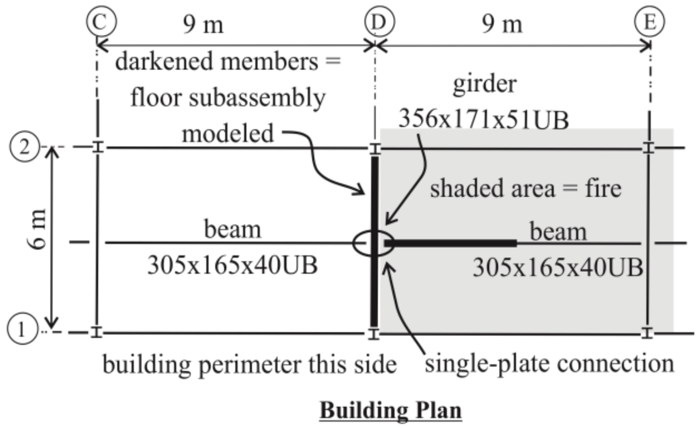


Figure 1: Details of Cardington [Wald, F. et al 2006] (a) single plate connection and (b) the building compartment and the subassembly (illustrated with thick lines).

Fire Scenario

In Cardington, a moderate fire scenario is used with the fire load of 40 kg/m^2 with wooden cribs covering the compartment floor area. In our model, the secondary beam is heated on three-sides with the average (recorded) thermocouple readings at the beam midspan taken from the Cardington test. The range of the beam midspan (structural) temperature and the gas (fire) temperature history is shown in Figure 2. The intensity of fire is slightly reduced near the connection region due to local massitivity.

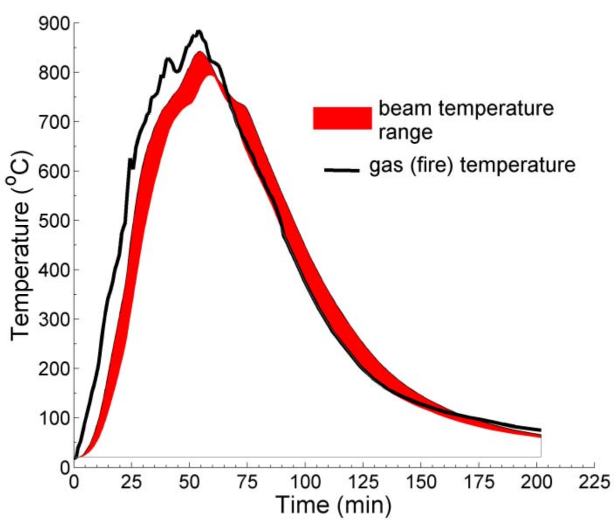


Figure 2: The temperature history of the recorded compartment fire and the range (min-max) of beam (305x165x40UB) temperature in Cardington building test.

Material Properties

In the finite element model, the steel properties of the plate, bolts, beam and girder are taken directly from the measured values at ambient temperature in the Cardington building test. Young's modulus E of steel is taken as 207GPa for all components. The strength and stiffness of hot-rolled steel are reduced according to Eurocode provisions [ECCS 2001]. The elevated temperature properties of quench-tempered bolts are applied using Kirby [1995]. The steel plasticity is modeled using 'von Mises' yield criteria with isotropic strain hardening. No material softening (decreasing branch of the stress-strain curve) is included as this would greatly decrease the convergence character of the problem. Table 1 shows the ambient temperature yield and ultimate (strain hardening) strength of the subassembly components. The single plate, single angle and double angles are modeled using Grade 43 strength. Figure 3 shows the (engineering) stress-strain curve of four components at ambient temperature and 600 ^{o}C. These curves are converted to true stress and strain for the finite element analysis.

Table 1: Steel properties at ambient temperature.

Materials	**S275** (beam)	**S355** (girder)	**Grade 43** (plate/angle)	**Grade 8.8** (bolt)
yield strength (MPa)	303	396	275	695
ultimate strength (MPa)	469	544	430	869

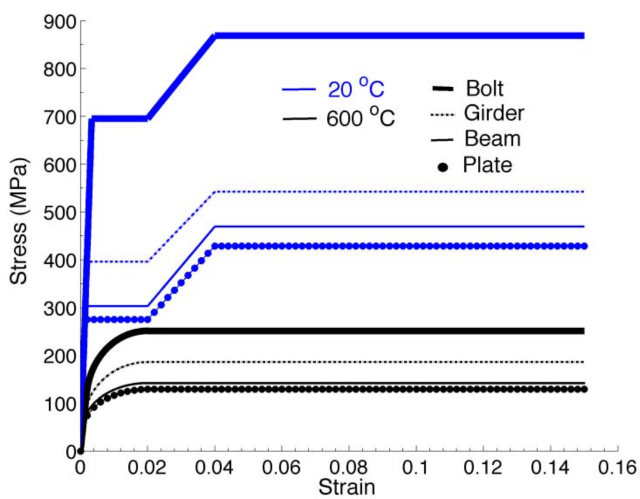


Figure 3: Eurocode steel stress-strain curves with strain hardening for plate/angle, bolts, 305x165x40UB (beam) and 356x171x51UB (girder) at 20 ^{o}C and 600 ^{o}C.

DESIGN OF SINGLE AND DOUBLE ANGLE CONNECTIONS

The single and double connections are designed for ambient temperature conditions for the Cardington building scenario using AISC provisions [AISC 2005]. The geometric details of the three connections are illustrated in Figure 4. As seen in Table 2, the connections are designed with similar shear and tensile strength according to AISC provisions. All three connections have beam web bolt-bearing as the governing limit state in shear and tension as stated in Equation J3-6a in Steel Construction Manual [AISC 2005]. The single plate and single angle connections have four bolts and are almost identical in geometry. The double angle connection has a smaller angle thickness (6 mm) and has 3 bolts centered in the beam web. Further, the bolt end distance (L_{eh}) is 50 mm in the beam web. Such geometric change is necessary to keep the ambient temperature shear and tensile strength of the double angle connection similar to the strength of the single plate and single angle models.

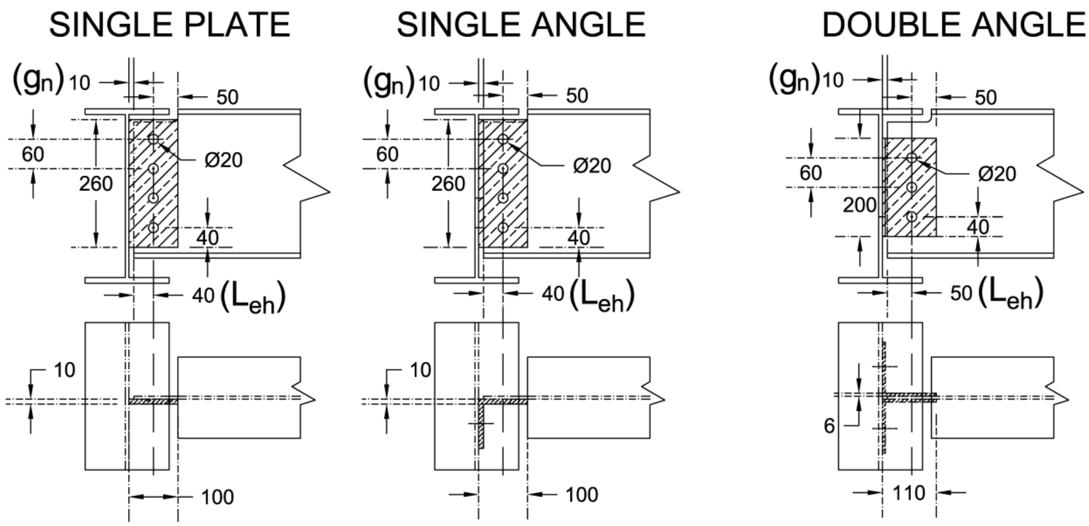


Figure 4: Geometric details of the three shear connection types (dimensions are in millimeters).

Table 2: Shear and tensile strength of single plate, single angle and double angle connections at ambient temperature.

dimensions in mm	**Shear strength (kN)**	**Tensile strength (kN)**
Single Plate (P10-260x100)	349	294
Single Angle (L10-100x100)	349	294
Double Angle (2L6-110x110)	294	296

Neither of the angle models are designed considering the prying action because the expected tensile force in the beam is not known apriori. Figure 5 shows the prying action of an angle. Here, T is the maximum expected tensile force in the angle (per bolt), Q is the prying force (per bolt) and B is the allowed tensile force in the bolt. The geometric parameters are a', b' and p (tributary length per bolt).

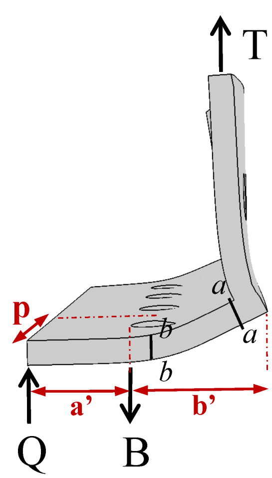


Figure 5: Prying action and illustration of forces T, Q and B in an angle.

The general formula for the minimum angle thickness t_{min} to avoid failure due to prying action is stated by Thornton [1985] in Equation 1.

$$t_{\min} = \sqrt{\frac{8Tb'}{pF_y(1+\delta\alpha)}} \qquad \text{Eq. 1}$$

Here, F_y is the yield strength of the angle and δ is the ratio of the net area at section *b-b* and the gross area at section *a-a* (both shown in Figure 5). The coefficient α is the ratio between the moment per unit width at section *b-b* and the moment per unit width at section *a-a*. For the single angle model; if we assume that T is equal to 400kN (:= 100kN per bolt) and B is the ultimate tensile capacity of Grade 8.8 bolts, the equation suggests that t_{min} should be about 25 mm (~1 inch) with $\alpha = 1$ and $\delta = 1$. Similarly, t_{min} = 28 mm (~1.1 inch) for the double angle model. Hence, both the single angle (t = 10 mm) and the double angle thickness (t = 6 mm) used in the model suggest that the prying action will likely occur due to large tensile forces in the beam during fire.

THE FINITE ELEMENT MODEL

Meshing

Several convergence studies for the three-dimensional finite element connection models were implemented previously by the authors [Selamet and Garlock 2010]. For the bolts and the bolt-hole regions, 20 elements are found to be sufficient to accurately represent the shear and tension capacity of the region. Further, minimum of 3 elements through the angle/plate thickness is necessary to capture the bending behavior of the angles or the plate. Generally, finer meshes are utilized around the connection region, where large concentration of forces and instabilities such as the local buckling are expected. Fully integrated (C3D8) and incompatible (C3D8I) hexagonal elements are used near connection region and at contact areas, whereas reduced integrated (C3D8R) elements are used for the rest of the subassembly.

Imperfection Study

Due to the eccentricity of single plate and single angle connections, no imperfection is introduced prior to static analysis since deformations will inherently develop as the beam starts to expand at elevated temperatures due to the unsymmetrical nature of the connection. For double angle connections, however, imperfections need to be introduced, because of the perfect symmetry at the beginning of the analysis. The imperfection geometry is found using the eigenvalue extraction method by ABAQUS. The eigenmode of the subassembly is scaled to 1% and introduced to the general static analysis.

Analysis

An uncoupled thermo-mechanical analysis is performed since the heat transfer is independent of the structural response of the subassembly. The creep property of steel is not considered; hence an uncoupled thermal and structural analysis is computationally efficient. For gravity and fire loading, a purely static analysis is performed, where several stabilization techniques are utilized in order to establish convergence especially at the onset of the beam web and lower flange buckling at the connection.

OBSERVATIONS AND RESULTS

Out of the three subassemblies, only the single plate model failed by bolt bearing in the beam web at 123 minutes into the fire as seen in Figure 6. Both the single angle model and the double angle model (see Figure 7 and Figure 8) survived the entire fire duration without significant deformation in the beam but with excessive bending and significant plastic deformation in the angle due to the large tension in the beam. Additional details of observations are listed in Table 3.

Table 3: Response details of finite element models

PARAMETERS	SINGLE PLATE	SINGLE ANGLE	DOUBLE ANGLE
time @ analysis end (min)	123	202	202
temperature @ analysis end (°C)*	231	62	62
$P_{c,max}$ (compression-kN)**	745	750	750
$P_{t,\ max}$ (tension-kN)**	395	305	280
connection limit state	beam web bolt bearing	excessive angle bending, but no failure	excessive angle bending, but no failure
web buckling (time)	yes (@12 min)	yes (@ 11.5 min)	no
lower flange buckling (time)	yes (@ 20 min)	yes (@ 20 min)	yes but minimal (@ 20 min)
girder contact (time)	yes (@ 14 min)	yes (@ 14 min)	yes (@ 15 min)
other observations	n/a	girder top flange buckling near connection	girder top flange buckling near connection

* average beam midspan temperature
** at beam midspan

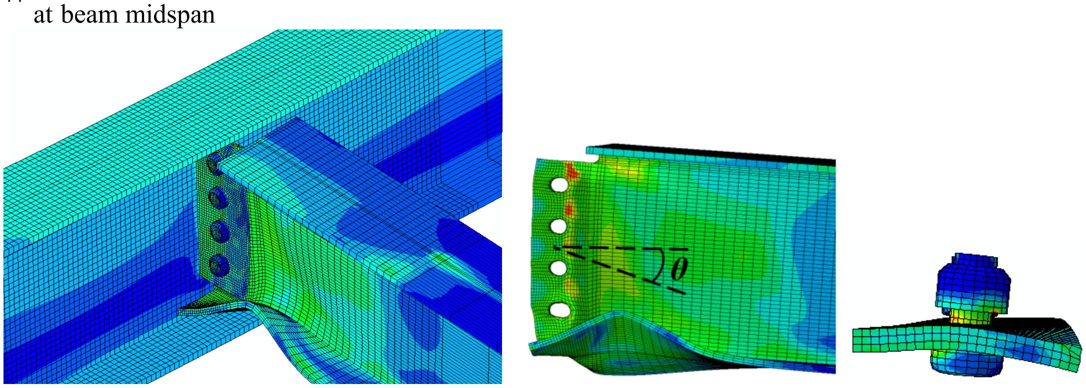


Figure 6: Mises contours and deformations of single plate connection subassembly at the end of analysis (123 minutes). θ represents the connection rotation angle.

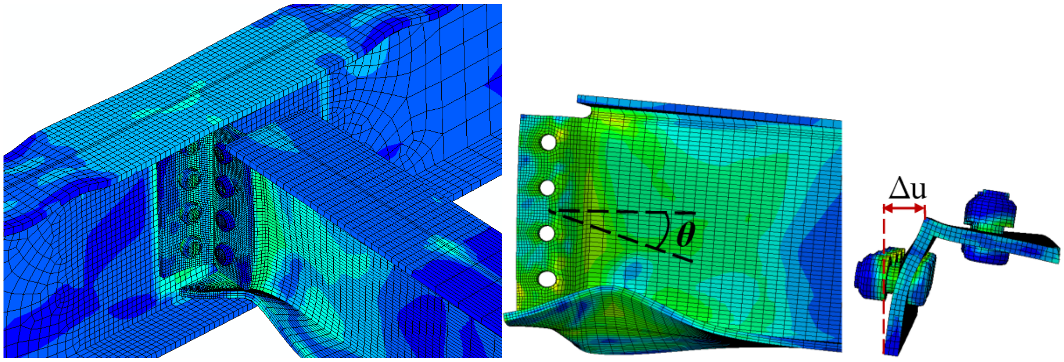


Figure 7: Mises contours and deformations of <u>single angle connection</u> subassembly at the end of analysis (202 minutes). Δu represents the horizontal uplift of the angle heel. θ represents the connection rotation angle.

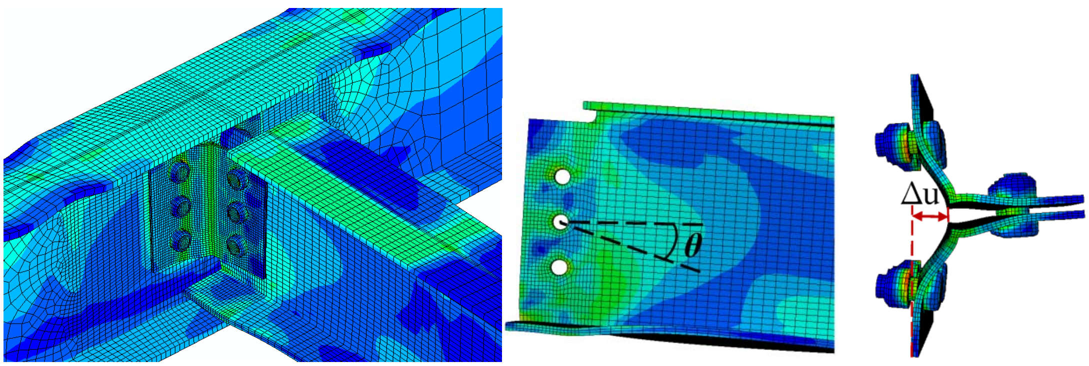
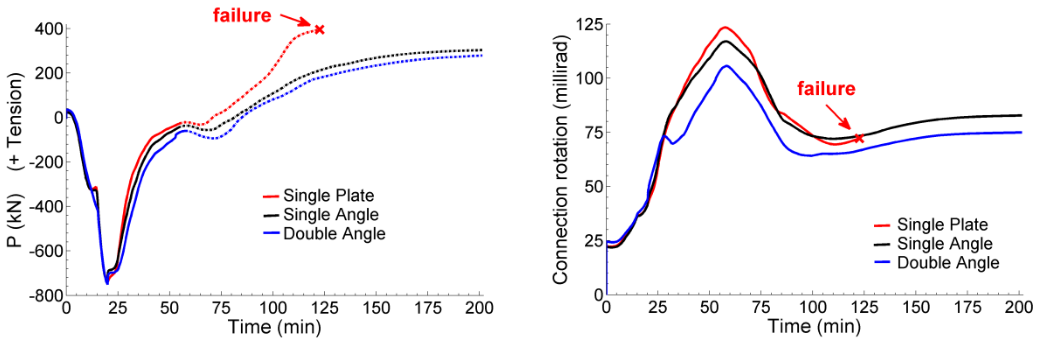


Figure 8: Mises contours and deformations of <u>double angle connection</u> subassembly at the end of analysis (202 minutes). Δu represents the horizontal uplift of the angle heel. θ represents the connection rotation angle.

The axial force time history in Figure 9 illustrates the differences in structural fire behavior between the angles and the plate as a shear connection. In previous papers [Selamet and Garlock 2010], we have shown that the maximum compression in the beam during heating phase of a fire was governed by the buckling capacity of the beam web and lower flange near the shear connections. The single and double angle shear connections also exhibit this behavior. All of the three subassemblies have the same axial capacity in compression which is governed by the beam lower flange buckling. The single plate and single angle connections do not provide enough lateral resistance to the beam web; hence the web buckling occurs in early stages of fire. The double angles provide significant resistance to the beam web, and we observe no web buckling in this model. The extent of lower flange buckling is similar for single plate and single angle models, but it is much less severe for the double angle because it is not preceded by the web buckling.

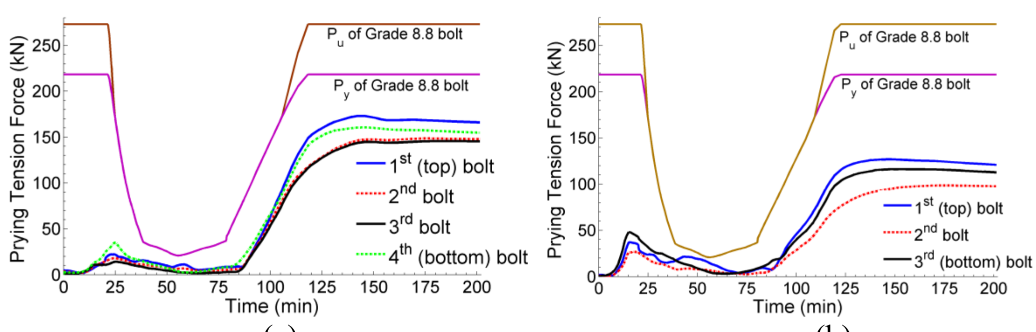
As seen in Figure 9, the tensile strength capacity of both the single angle and double angles is lower than that of the single plate connection. Although the maximum tensile strength of angle connections is lower, the angles exhibit ductile behavior and survive the fire. As the beam contracts during the cooling phase, the tensile resistance of the single plate itself (ignoring the other connection parts) is governed by the gross yielding of the plate, whereas the tensile resistance of the angles itself is governed by the bending capacity of the angle leg. The additional deformation provided by the angle leg significantly decreases the beam tensile force because it adds flexibility to the connection. Hence, the subassembly with the single angle connection survives the fire and the beam does not fail by the bolt-bearing limit state. On the other hand, the double angle connection has thinner angles (6 mm) and thus a smaller bending resistance. This model survives the fire, but experiences significant plastic deformation (see Figure 8) whereas the beam bolt region does not deform.



(a) (b)

Figure 9: (a) Axial force time history in the beam (dashed lines represent the cooling of fire) and (b) connection rotation θ (clockwise is positive, see Figures 6, 7 and 8).

All three shear connections exhibit similar moment time history (although not shown). This indicates that the moment at a shear connection is dependent mostly on the slab (composite deck) behavior and the gap distance (g_n) between the beam and girder, and it does not necessarily depend on the type of the shear connection. Since the gap distance (see Figure 5) and the *Connector resistance representing the slab structural behavior are kept the same for all models, the beam moment time history as well as the connection rotation (shown in Figure 9b) are similar for all three shear connection designs.



(a) (b)

Figure 10: Bolt tension (due to prying action) for (a) the single angle and (b) the double angle connections (due to symmetry, only 3 bolts on one angle are shown). P_u and P_y represent the ultimate and yield bolt strength, respectively.

Figure 10 plots the tension in the bolts (*B* in Figure 5), which includes the prying action for (a) the single angle and (b) double angle connections. Figure 10a shows that the bolts in the single angle exhibit significant tensile forces; however they do not reach their yield or ultimate tensile strength. The three bolts in the double angle model (Figure 10b) exhibit smaller tensile forces as expected because the tension in the beam is split evenly into two angles.

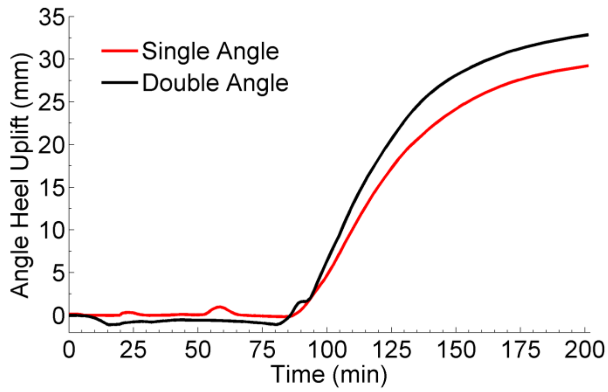


Figure 11: Angle heel uplift or relative displacement Δu (see Figure 7 and 8) for the single and double angle connections.

Figure 11 shows the uplift of the angle heel (at the top) for both models. The double angle connection experiences a larger uplift (Δu) because of its thinner angles. Since both angle models have a large Δu (~30-35 mm), we can assume that such deformation absorbs some of the beam contraction during cooling phase of the fire. Such flexibility in the connection prevents the beam web from failing by bolt-bearing limit state.

CONCLUSION

In this paper, we designed and modeled single and double angle connections using the same boundary, loading and fire scenario of the single plate connection subassembly which is adopted from the Cardington building test. The finite element models are able to capture the large deformation and the change of contact conditions accurately for the duration of fire. Our observations suggest that the angle connections have a more ductile fire response compared to the single plate although their maximum tensile strength is lower. Both the single and double angle connections survive the natural fire with significant plastic deformation in the angle legs whereas the single plate connection fails by bolt bearing in the beam web. Using different type of shear connections did not change the overall response of the subassembly during heating, but had significant influence on the overall response during cooling. We can confirm this by looking at the beam axial force diagram where the onset of the lower flange buckling and the maximum beam compression for all three connection models are almost identical. Further, the models exhibit similar connection rotation and beam midspan deflection behavior. During cooling, the shear connections have different responses in terms of failure limit state, deflection and maximum tensile strength.

During the heating phase of fire, all connections experience local buckling in the beam lower flange when the gap distance (g_n) closes and the girder-beam contact is established. The lower flange buckling also marks the maximum compression in the beam. The beam web buckling occurs in the coped beam only for connections with plates/angles on one side of the beam web (single plate and single angle).

As the beam contracts during the cooling phase of fire, the bending of the angle legs due to large tensile force in the beam creates the phenomenon called 'prying action'. The use of high strength bolts eliminates the possibility of a bolt failure in tension; however the bolt tensile forces get close to the bolt yielding capacity. The angle connections provide additional flexibility that the single plate connections do not have. The use of thin angles for the double angle connection creates significant plastic deformation in the angle, and it is recommended to use caution when selecting the angle thickness. In conclusion, we see a significant benefit using the angle connections for a more robust fire connection design.

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