Improved Details for Fire-Induced Steel Single Plate Shear Connections

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ABSTRACT

The objective of this paper is to identify cost-effective improved modification details for single plate (i.e., shear tab or fin plate) shear connections in a fire. Using ABAQUS, we developed a finite element model of a single plate connection that was tested full scale in Cardington. We modified several connection details to find modifications that led to increased strength and/or ductility, where an increase in either one also meant an increase in fire resistance time. The results illustrate some simple modifications to the connection design that improve the connection's fire performance.

INTRODUCTION

Simple shear connections are vulnerable to large compressive and tensile forces as well as imposed rotations induced by a natural fire during both heating and cooling phase. Recent fire events and full-scale experiments of steel buildings [2, 8, 11] have demonstrated that (1) the load and rotation capacity of the connections generally govern the behavior of floor systems in a building under fire and (2) shear connections are especially vulnerable during the cooling phase of the fire when large tensile force develop. Such results have been investigated and confirmed by numerical methods finite element software ABAQUS [5, 9].

We previously developed a finite element model of a single plate shear connection in a subassembly from a Cardington building full-scale fire test [5]. The model, which used a simplified method to structurally and thermally include the effect of the concrete slab, accurately captured the beam web and flange buckling and the failure modes like bolt shear and bolt bearing.

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Using that validated model, our objective in this paper is to investigate the following modifications to single plate connection details to evaluate if they can lead to improved performance under fire: bolt grade, bolt holes type, adding a doubler plate on the beam web, the thickness of the connection plate, bolt pretensioning, the gap distance between the beam and beam support, and the distance between the bolt hole and the edge of the plate, and the relationship between this distance and the bolt design. A more detailed discussion of this study will shortly be published elsewhere [10]. In this paper, we highlight some of the significant observations.

FINITE ELEMENT MODEL

A finite element model is developed based on a portion of a floor system from a full-scale building experiment performed in Cardington as shown in Figure 1 [11]. Figure 2 shows the floor subassembly and connection finite element model. The FE model details are given by Garlock and Selamet [5].

Ambient temperature material properties of different components of the subassembly are taken directly from Cardington experiment measurements [2]. Eurocode reduction factors [4] were used to reduce the stress-strain material properties at elevated temperatures of the connection members except the bolts. For the bolts, Kirby's suggested reduction factors were used [7].

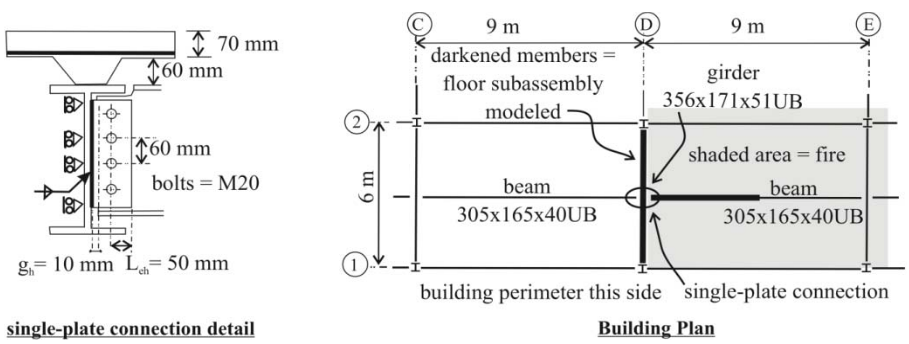


Figure 1. Structural design of the 2003 Cardington building tests [11].

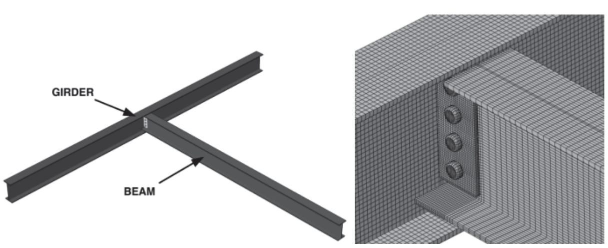


Figure 2. Finite element model of the (left) subassembly, (right) connection detail.

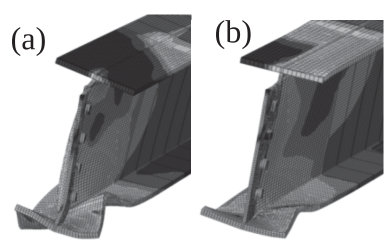


Figure 3. Buckling deformations at the maximum rotation (60 min) of (a) STD bolt (Test 0) and (b) SSLP bolt (Test 3).

ABAQUS implicit solver is employed for the finite element (FE) analyses. FE contact mechanics algorithm software simulated a realistic interaction between the connection parts (i.e. contact pressure and friction). To represent the slab, simple spring elements that only have vertical stiffness are used and calibrated with experimental data. The FE model was validated with the Cardington test data as described by Garlock and Selamet [5].

RESULTS OF CARDINGTON FE MODEL

During the entire heating period, the beam was in compression and during the cooling period it was in tension. The FE results indicate that such tension lead to connection failure by bolt bearing on the beam web holes. “Failure” in our model is defined as the point where the field equations stop converging due to excessive plastic strains (larger than 20% strain) in the connection. The connection in the actual experiment had large bearing deformation on the beam web holes but no failure.

During the fire growth, the web and bottom flange sections bear almost all the compressive axial force in the connection region until they buckle. During the fire decay, the beam goes into tension and the web section carries almost all the tensile axial force. The results suggest that both compression and tension in the beam near the connection region is carried by an area less than the full beam cross section.

CONNECTION DETAIL MODIFICATIONS

This section describes the modifications to the connection details of the Cardington design to evaluate their effects on the fire performance of the floor system. The fire demand (time-gas temperature history) of all studies was the same as the one measured in the Cardington. Table I shows the parameters and results for the eight different studies (tests) that were based on modifications to the “Original” Cardington connection detail described in [5] and Figure 1, which we define as Test 0. The modifications are shown in bold font.

Bolt Grade (Test 1)

The effect of bolt strength is examined by reducing the ultimate strength of bolts by 40%, that is, by changing the grade from 8.8 to 5.6. The ambient temperature ‘measured’ [8, 11] yield and ultimate strength of Grade 8.8 are 695 MPa and 869 MPa, respectively. The ambient temperature ‘nominal’ yield and ultimate strength of Grade 5.6 are 300 MPa and 500 MPa, respectively.

Results show that the reduction of bolt strength does not affect the beam axial force and moment in the beam during heating phase; however Test 1 with Grade 5.6 bolts fails by bolt shear at 192 kN of axial force in tension. The Cardington connection model (Test 0) with Grade 8.8 bolts fails due to web bolt bearing at 359 kN of axial force in tension (see Table I). Both failures happen during the cooling phase.

TABLE I. TESTS WITH VARIOUS MODIFICATIONS (IN BOLD) TO THE SINGLE PLATE CONNECTION. TEST IS THE ORIGINAL CARDINGTON CONNECTION DESIGN.

	TESTS		0	1	2	3	4	5	6	7	8
PARAMETERS	Bolt grade		8.8	**5.6**	8.8	8.8	8.8	8.8	8.8	8.8	8.8
	Bolt-hole size (mm) [1]		20	20	**24**	**22x26**	20	20	20	20	20
	Shear plate thick. (mm)		10	10	10	10	10	**6**	10	10	10
	Doubler Plate thick. (mm)		-	-	-	-	**4**	-	-	-	-
	Bolt pretension (kN)		-	-	-	-	-	-	**142**	-	-
	Gap g_h (mm)		10	10	10	10	10	10	10	**19**	**19**
	L_{eh} (mm)		40	40	40	40	40	40	40	40	**70**
RESULTS	**Limit State** [2]		WB	BS	WB	WB	NF	PB/WB	WB	WB	NF
	Time of Events (min)	@ contact [3]	14	14	15	15	15	14	14	20	22
		@ P_{max} (Comp.)	20	20	20	20	20	20	20	26	26
		@ end [4]	123	101	126	140	202	178	165	165	202
	Rotation (mrad)	@ contact	43	41	34	34	46	46	45	84	89
		@ P_{max} (Comp.)	53	46	35	36	52	49	52	98	96
		max. rotation	97	96	78	74	81	83	108	136	127
	P(kN)	P_{max} (Comp.) [5]	718	720	720	732	712	703	716	549	565
		P_{max} (Tension) [5]	359	192	357	369	622	348	401	407	433
		$P_{c\ max}$(Tension) [6]	316	157	323	358	555	286	352	368	396
	T (°C) [7]	@ P_{max} (Comp.)	260	260	260	261	260	260	260	431	431
		@ P_{max} (Tension)	256	391	295	311	128	112	240	248	251
	Δ (mm) [8]	max. deflection	344	349	350	366	368	325	336	349	354

[1] STD, OVS and SSLP bolt hole sizes
[2] Limit States: WB=web bearing, PB=plate bearing, BS=bolt shear, NF=no failure
[3] contact is when the beam bottom flange contacts the girder
[4] end of analysis when convergence cannot be reached or when analysis is completed at 202 min (e.g. Test 4 and 8)
[5] P_{max} is the maximum beam axial force
[6] $P_{c\ max}$ is the maximum tensile force in the beam coped web section (does not include the upper or lower flange)
[7] T is the average beam temperature at midspan
[8] Δ is the deflection at beam midspan

Bolt-hole Type (Test 2 and Test 3)

Three different bolt types are tested: standard bolt-hole type (STD) with equal bolt and hole diameter (20 mm), oversized bolt-hole type (OVS) with larger hole diameter (24 mm) and short slotted bolt-hole type (SSLP) with slightly elliptical hole shape (22x26 mm). These cases represent Tests 0, 2 and 3, respectively. A larger bolt-hole size allows the beam to rotate and expand more independently from the connection and this modification affects both buckling of the web and the lower flange (Fig. 3). It is seen that after 60 minutes of the fire, the beam of the STD design (Test 0) deforms much more than the SSLP design (Test 3) as well as the OVS design (although not shown).

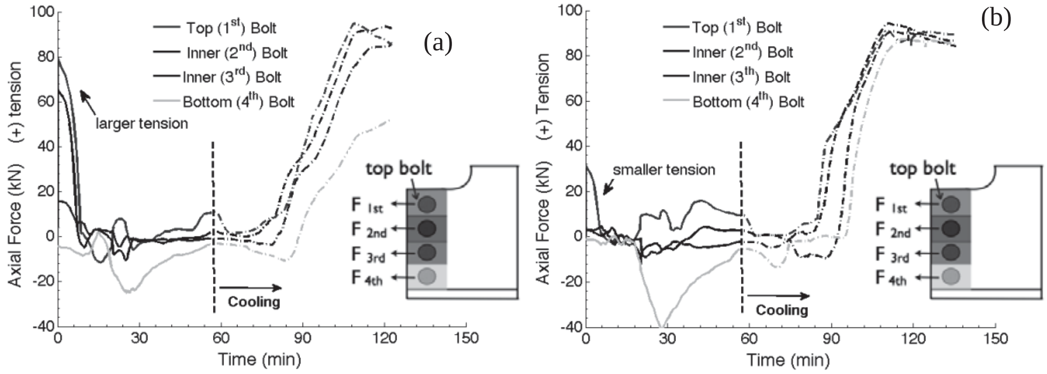


Figure 4. Bolt regions in the coped beam web and regional axial forces for (a) Test 0 and (b) Test 3.

When the tensile failure in bolt bearing (beam web or single plate) is considered as a design criterion at ambient temperature, uniform forces are assumed to act on all the bolt regions. The tensile capacity of the connection is found by multiplying the capacity of a single bolt by the number of bolts. This methodology is valid if the connection is not significantly deformed. However, due to the extent of lower flange buckling, as seen in Figure 3a, the bottom bolt in Test 0 is significantly bent out-of-plane at the end of the analysis, whereas such distortion is far less in Test 3. Figure 4a and 4b plot the four bolt section (internal) forces for the STD design (Test 0) and for the SSLP design (Test 3), respectively. In the STD design, the last bolt region (F_{4th}) takes lesser load during fire decay compared to other three bolt regions above it. Such behavior is due to significant plate distortion near the last (bottom) bolt. The SSLP design carried nearly equal tension in all four bolt regions since there was less distortion near the last bolt. The OVR design (not shown) performance in the last bolt region was between STD design and SSLP design as expected. Selamet and Garlock [10] propose an equation for bolt bearing capacity that is modified for reduced capacity due to such distortions.

Doubler Plate (Test 4)

In Test 4, 4 mm thick doubler plates (web stiffeners) were added to the original connection (Test 0). Since the web is 6 mm thick, adding 4 mm makes the combined thickness equal to the shear tab that is 10 mm thick. The FE results (Table I) indicate that adding a doubler plate to the beam web improves the connection performance: the buckling develops more gradually, the connection tensile strength increases by nearly 73% to 622 kN, and the subassembly survives the fire until gas temperature cools down to 70 °C (after 3 hours) with less visible plastic deformation in the connection region.

Thickness of the Single Plate (Test 5)

In Test 5, we reduced the thickness of the shear tab from 10 mm to 6 mm to match the beam web thickness which still satisfies the flexural buckling strength design of a shear tab according to Jaspart [6]. Significant deformation in both the shear tab and the beam web are observed (Test 5). The original design (Test 0) develops minimal deformations of the shear tab whereas the beam web develops excessive deformations. Since in Test 5, both the shear tab and the beam web are deforming, the connection is more ductile and failure happens one hour later than the design with the thicker shear tab (Test 0). The connection strength (maximum P in tension) stays almost the same in both tests since it is controlled by beam web bearing and the beam web was not modified.

Pre-tensioned Bolts (Test 6)

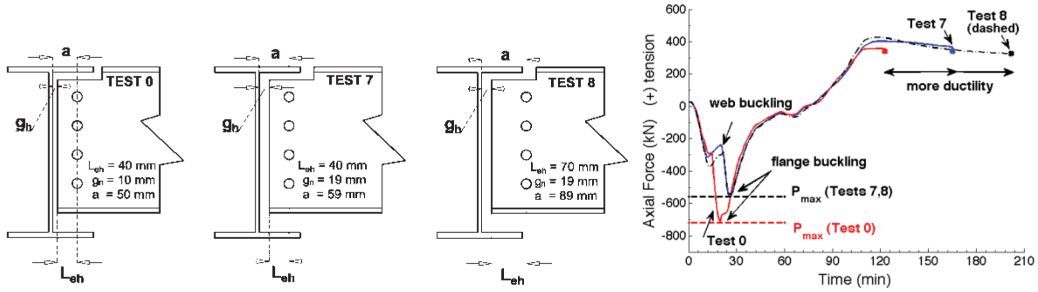
In Test 6, we pretension the bolts to allow frictional forces to develop against slip of the components and hence prevent excessive rotation. According to LRFD provisions [1], 142 kN is applied to each bolt, which is equivalent to 0.7 times the nominal tensile bolt strength for M20 bolts.

The FE study indicates that the region where normal forces act on the contact surface is relatively small around the bolts and the entire plate surface is not engaged in friction. The fire induced forces and moments between the components overcome the frictional resistance forces. However, larger contact shear (tangential) forces between the beam and plate around the bolt-holes are observed in Test 6 (pretension) compared to Test 0 (no pretension). Such forces act against the bearing deformation of the bolt-holes during the fire decay when large beam tensile axial force is observed. Therefore, the maximum beam tensile strength at midspan of Test 6 is about 10% larger (401 kN) than that of Test 0 (359 kN). This additional strength adds about 40 minutes to the survival of the connection during cooling.

Gap Distance (Test 7 and Test 8)

Changing the gap distance (see Fig. 5) will change rotation at which the bottom flange contacts the girder (g_h). Test 0 had g_h =10 mm, and the distance from the bolt center line to the edge of the beam (L_{eh}) equal to $2d_b$= 40 mm. In Tests 7 and 8, we increase g_h to 19 mm, and additionally for Test 8, we increase L_{eh} to $3.5d_b$= 70 mm. The Steel Construction Manual (SCM) [1] defines the value $a = g_h + L_{eh}$ and it limits $a \leq 89$ mm (3 ½ ’’). Figure 5 shows a sketch of the 3 connection designs (Tests 0, 7 and 8) and the values of g_h and L_{eh}. All 3 designs meet the SCM’s limit of the parameter a.

Figure 8b plots the axial force in the beam near the connection region. It is seen that the web and flange buckling is delayed slightly for Test 7 and Test 8 compared to Test 0 due to a larger gap distance g_h. Also, the peak compressive axial force in Tests 7 and 8 is about 20% less than that for Test 0. Furthermore, Figure 8b confirms that Tests 7 and 8 have more ductility where the connection is able to withstand the force for about 40 more minutes than Test 0. In these tests, the beam is permitted to rotate more before contact between girder and beam flange is established. Therefore, more of the thermal elongation is developed in the beam bending curvature compared to Test 0, which contacts the girder sooner and is therefore more axially restrained from elongating, leading to a larger compressive force in the beam (see Table I).



(a) (b)

Figure 5. (a) Geometric details of connections with different L_{eh} and g_h (b) axial force near connection (0.1m from support) with different gap distances.

SUMMARY AND CONCLUSIONS

A finite element model of the single plate shear connection used in the full-scale experiments (Cardington) indicates that beam web bearing failure of the bolt holes was the limit state reached at the end of the analysis. The experiments show large bearing deformations but no failure. FE analysis indicated that near the connection region, the axial forces (tension and compression) are carried by an area that is less than the full cross-section. After the bottom flange contacted the supporting member, only the web and bottom flange carried the compressive force. The tensile forces that develop later are carried only by the web.

The modifications studied and the conclusions are the following:

<u>Bolt Grade</u>: A larger bolt grade (e.g. A490 or G8.8) is recommended for increased capacity.

<u>Larger bolt holes</u>: Using OVR or SSLP bolt holes in the beam web reduces buckling deformations of the beam near the connection allowing evenly distributed forces in the bolts. However, the overall global performance of the connection was only slightly affected by these larger hole sizes because they were not large enough. The oversized hole added only an extra 2 mm movement in each direction and the short slotted added only 3 mm in each horizontal direction.

<u>Adding web doubler plate</u>: Adding a plate to the beam web near the connection so that the combined beam web thickness (6 mm) plus doubler plate thickness (4 mm) equals the shear tab thickness (10 mm) increases the connection strength considerably. In fact, the analysis terminated without failure of the connection.

<u>Matching the single plate (shear tab) thickness to the web thickness</u>: When the shear tab thickness is reduced to 6 mm to match the web thickness, the overall connection strength (controlled by bearing) stays the same (as expected). However, the ductility in the connection is increased because bearing deformations develop in the shear tab in addition to the beam web, which results in additional axial flexibility and an increased fire resistance of nearly one hour.

<u>Pretensioning the bolts</u>: Adding pretensioned bolts to the single plate connection does not engage the entire plate and beam web surface into friction, however the tangential contact between these components around the bolt hole region creates

larger contact shear forces which resist the bearing deformation of the beam web bolt holes and thus slightly increase the tensile capacity of the connection and increase the time to failure.

Increased gap distance: The distance between the end of the beam and the supporting girder face was increased from 10 mm to 19 mm. This delayed the contact between the beam bottom flange and the supporting girder, which resulted in the following: (1) larger tensile strength since the flange buckling deformations were smaller and the tensile forces were therefore more evenly distributed between the bolt holes, (2) smaller maximum compression since there is less axial restraint from thermal elongation and (3) about 40 minutes of added fire resistance before bearing connection failure.

Increased distance from bolt hole centerline to the end of the beam (L_{eh}): The bearing capacity of the connection depends on the thickness of the beam web (t) and L_{eh}. In Test 8, the L_{eh} is increased from $2d_b$ to $3.5d_b$ and a significantly larger connection tensile capacity is achieved; the model survived the fire without failure. Increasing L_{eh} or t will increase the bearing capacity of the connection; however, one must consider that the limit state may change to bolt shear.

ACKNOWLEDGEMENTS

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