Experimental Investigation of the Robustness of Fin Plate Connections in Fire

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Abstract

Fire hazards and full-scale structural tests have indicated that steel connections could be subjected to large deformations and fracture in fire. This is not currently considered in design approaches because the connections are assumed to heat up more slowly than the structural frame members, and therefore retain more relative strength. A project at the Universities of Sheffield and Manchester is currently investigating the robustness of common types of steel connections when subjected to fire. This paper reports on a part of the test results on fin plate connections. The test results illustrate that bolts have their strength reduced faster than hot-rolled steel with increase of temperature, and failure of fin plate joints is quite often controlled by bolt failure in shear. As a result of bolt shear fracture, fin plate connections have unexpectedly low resistance and ductility when subject to elevated temperatures and large rotations.

1. Introduction

Current design codes generally consider that steel connections will be heated more slowly than beams or columns in fire situations, and are therefore less likely to be the critical components in fire safety design. However, evidence from the collapse of the WTC buildings (FEMA, 2002) and full-scale fire tests at Cardington (Newman *et al.*, 2004) indicates that connections may often be the weakest link in a structural frame in fire conditions. This is because, at ambient temperature, connections are designed to transfer shear and/or moment, whereas in fire they can be subjected to additional compressive or tensile forces due to restraint to thermal expansion or to catenary action arising from large deflections. At very high temperatures, beams lose most of their bending capacity, and develop axial tensile forces which, in combination with large deflections, may support the lateral loads by second-order effects. In consequence, the connections may eventually be subjected to large rotations and significant tensile forces. Under such conditions there is a clear possibility of connection fracture, which may lead either to fire spread to upper floors, or to progressive collapse of the building.

In the past, connections have been extensively investigated to determine moment-rotation behaviour. However, the importance of connections in providing tying resistance to hold the whole structure together should not be overlooked (BSI, 2001). Previous researches (Yu and Liew, 2005) have shown that in a fire situation connections can be subjected to significant tying force when the beams are heated and deformed to high deflection so that they perform like cables to resist upper floor loads.

When this behaviour was identified as a factor which could be used in design to enhance the fire resistance of structural steel frames, it was implicitly assumed that the connections between beams and columns had sufficient rotational ductility, and could transfer any catenary force up to the tensile capacity of the beam section at elevated temperatures. However, the actual behaviour of steel connections under such circumstances has never been investigated. The tests which are currently in progress at the University of Sheffield were designed to understand the behaviour of common steel connections when subjected to significant catenary forces. A total of four connection types will be studied. They are the flush endplate, flexible endplate, fin plate and web cleat connections. This paper reports 14 test results on fin plate connections. See Yu *et al.* (2007) for a detailed description of the test setup. Apart from the eight tests using three Grade 8.8 M20 bolts reported previously, this paper presents results of a further six tests using modified specimens.

2. Test results

Table 1 lists all the test results. The first column gives a simple description to the test specimen by three numbers *(Number of bolts - Grade of bolts - Bolt diameter)*. For connections with three bolts, the specimen geometry is shown in Figure 1. When six bolts are used the fin plate width is extended to 150mm and two columns of bolts are used at 50mm spacing. During a test the specimen is heated to a specific temperature and then loaded to failure. The second column in Table 1 shows the temperature. The connection is loaded by a tensile bar, which is aligned at a certain angle to the axis of the beam. Different angles apply different combinations of shear and tying force to the connection. Two initial angles, of 55° and 35°, are used for each complete range of temperatures. Due to movement of the loading system and rotation of the beam itself, the loading angles change during a test. The loading angles at the beginning and end of each test are listed in columns 3 and 4 of Table 1. The initial angles are not exactly the same as the design values, depending on the assembled positions of the loading system components. The final two columns of Table 1 show the maximum resistances and the connection rotations at maximum resistance.

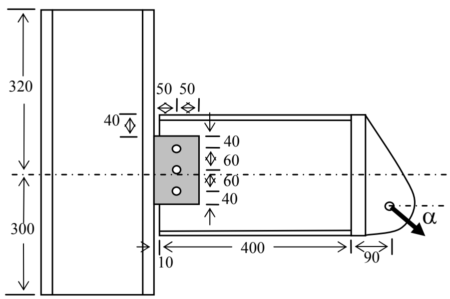

Figure 1. The geometry of the test specimen.

The failures in the eight tests using three Grade 8.8 M20 bolts follow certain patterns. At ambient temperature, the bolts deformed the bolt holes on the beam web before experiencing shear fracture. At elevated temperatures, the bolts sheared prematurely, causing little deformation of the bolt holes. Deformations to the beam webs and the bolts are shown in Figure 2. It appears that the bolts become weaker relative to the connected steel plates at elevated temperatures. The bolts used in the joints were tested at the University of Sheffield for tensile strength at ambient and elevated temperatures (Hu *et al.*, 2007). Reductions in the maximum resistances of the connections relative to their resistances at ambient temperature basically follow the strength reduction of the bolts, as shown in Figure 3.

(a) Ambient temperature (b) Elevated temperature (c) Bolt deformation

Figure 2. Failure of specimens with three Grade 8.8 M20 bolts.

Table 1. List of test results.

Test	Temp. (°C)	Initial α (°)	Final α (°)	Force (kN)	Rotation (°)
3-8.8-20	20	53.85	32.41	145.95	8.107
3-8.8-20	450	51.47	41.37	70.48	6.093
3-8.8-20	550	53.44	42.68	34.81	6.558
3-8.8-20	650	53.09	44.02	17.99	6.255
3-8.8-20	20	33.80	34.06	185.11	7.805
3-8.8-20	450	39.04	33.52	84.47	6.237
3-8.8-20	550	40.94	31.51	37.46	7.121
3-8.8-20	650	40.50	30.60	19.30	7.367
6-8.8-20	550	41.56	32.21	81.12	6.853
6-8.8-20	550	55.99	46.60	67.01	4.782
3-10.9-20	20	36.53	29.80	213.0	10.62
3-10.9-20	550	40.85	23.90	56.82	11.50
3-8.8-24	20	37.38	29.67	203.1	8.339
3-8.8-24	550	42.10	29.06	74.02	7.855

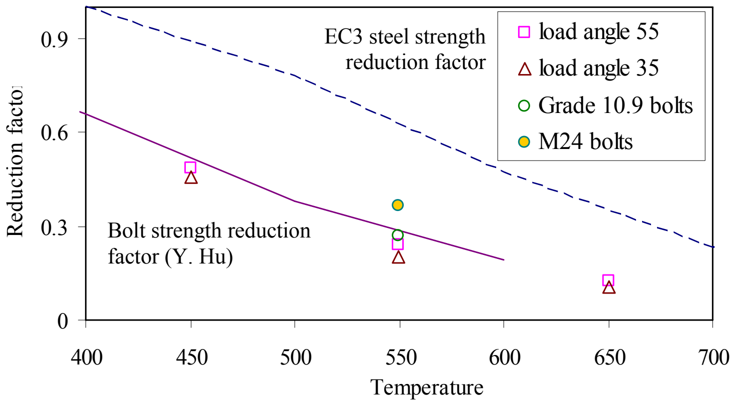

Figure 3. Reduction of the connection resistance at elevated temperatures

Figure 3 also shows the strength reduction curves for normal steel and for bolts according to EC3: Part 1.2 (CEN, 2005a). It is clear that bolt strength deteriorates much faster than hot-rolled steel. By using two columns of bolts, the resistances of fin plate connections are almost doubled. The failure is still controlled by bolt shear, and the bolt holes on the beam web show little deformation. Bolt shear is an undesirable failure mode, lacking ductility. The specimen was modified to check if increasing the bolt resistance can change the failure mode and enhance the tying resistance of the connection in fire. Two possible measures to enhance the bolt resistances were tried; increasing the bolt grade to 10.9 and increasing the diameter to 24mm. They were tested at ambient temperature and at 550°C. The reductions of resistance at 550°C are shown in Figure 3 relative to their resistances at ambient temperature. At elevated temperatures, the connections still failed by shearing their bolts, but the beam webs showed significant bearing deformation, as shown in Figure 4. At ambient temperature, the bolts remained intact and the beam web fractured in block shear. The deformed shape of the beam web is shown in Figure 4.

(a) Grade 10.9 at elevated temperature (b) M24 at elevated temperature (c) Grade 10.9 or M24 at ambient temperature

Figure 4. Failure of specimens with three Grade 10.9 or M24 bolts.

Comparison of the force-rotation responses to other tests at the same temperature are shown in Figure 5 for ambient temperature and Figure 6 for 550°C.

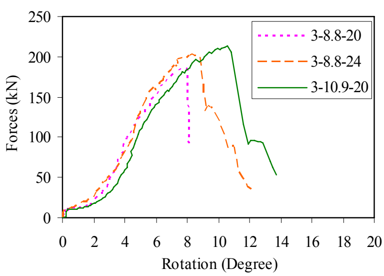

Figure 5. Comparison of responses at 20°C for various tests.

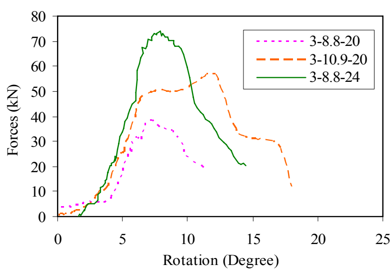

Figure 6. Comparison of responses at 550°C for various tests.

At ambient temperature, the use of stronger bolts slightly enhances both the maximum resistance and the ductility. The failure modes using grade 10.9 and M24 bolts are actually the same. The resistance using M24 bolts is smaller, probably because, by using larger bolts, the edge distance was reduced. At 550°C, using stronger bolts almost doubled the maximum resistance of the connection. As the ultimate failure is still by bolt shear fracture, the ductility of the connection for all three cases should actually be the same. Response of the connection with Grade 10.9 bolts was untypical. The top bolt appeared to have failed very early and at a much lower resistance than the other two bolts. This can be seen from the deformation of the bolt holes on the beam web, as shown in Figure 4. The lower two bolt holes clearly underwent significant bearing deformations, but the top bolt hole had no signs of deformation. The peak resistance of the connection, shown in Figure 6, was reached when the middle bolt reached its maximum load. It seems probable that, if the top bolt had behaved similarly to the others, the peak resistance of the connection with Grade 10.9 bolts would have been higher. In all the tests, the failures were controlled by bolt shear fracture. This indicates that bolts lost their strength at elevated temperatures faster than the structural steel. Considering that the temperatures of steel beams in catenary action may be above 700°C, the use of fin plate connections seems inadvisable where large connection rotations are anticipated.

3. Comparison with design guides

In the UK, the "Green Book" (SCI & BCSA, 2002) is commonly used for the design of simple connections. In this section, the resistances of the tested connections are evaluated using the method recommended by the Green Book and EC3: Part 1.8 (CEN, 2005b). The connections were subjected to shear, tension and moment. The resultant force applied to each individual bolt was then calculated assuming a plastic distribution of all the forces. EC3: Part 1.8 gives formulas to check both the shear and bearing resistances of individual bolts. These are shown as R_b and R_s in Table 2.

Table 2 Comparison of the forces per bolt to design guide recommendations

Test	F (kN)	R_s (EC3)	R_b (EC3)	R_b (Green Book)	R_{min}/F (EC3)	R_{min}/F (Green Book)
3-8.8-20	171.09	99.01	73.02	60.24	42.68%	35.21%
3-8.8-20	84.04	51.40	45.31	37.38	53.91%	44.48%
3-8.8-20	42.22	28.20	31.82	26.25	66.79%	62.17%
3-8.8-20	21.76	25.76	17.82	14.70	81.87%	67.54%
3-8.8-20	189.15	99.01	73.02	60.24	38.60%	31.85%
3-8.8-20	87.53	51.40	45.31	37.38	51.76%	42.70%
3-8.8-20	40.23	28.20	31.82	26.25	70.09%	65.24%
3-8.8-20	20.10	25.76	17.82	14.70	88.66%	73.14%
6-8.8-20	41.82	28.20	15.79	26.25	37.77%	62.77%
6-8.8-20	43.27	28.20	15.79	26.25	36.51%	60.67%
3-10.9-20	210.97	110.89	73.02	60.24	34.61%	28.55%
3-10.9-20	54.58	31.64	31.82	26.25	57.96%	48.09%
3-8.8-24	206.61	142.57	74.14	72.29	35.89%	34.99%
3-8.8-24	77.53	40.61	32.21	31.50	41.67%	40.63%

The Green Book aims to prevent shear fracture of bolts by limiting the ratio of the plate thickness to the bolt diameter, and therefore checks for the bearing resistance only. Bearing capacities are based on limiting the deformation at working loads to an acceptable level (approximately 1.5mm) and factoring this capacity for use at the ultimate limit state (Way and Salter, 2003). The UK model for a fin plate connection attempts to ensure that bolts are critical in plate bearing rather than shear, thereby providing adequate rotation capacity (Moore and Owens, 1992). However, as the bolt bearing capacity is not based on fracture it is possible that the actual failure mode is bolt shear. Comparison of the minimum resistances given by EC3 and the Green Book to the calculated bolt forces are shown in the last two columns of Table 2. Although both design guides represent the failure mode as plate bearing, they give very conservative predictions of the minimum individual bolt resistances, especially at ambient temperature.

4. Conclusions

Utilization of catenary action to enhance the fire resistance of structural steel beams calls for investigations into the capacity of steel connections to resist the tying forces generated within the beams. This paper reports 14 test results on typical fin plate connections subjected to combinations of shear and tension forces. The test results show that the resistances of fin plate connections are significantly affected by temperature. It was observed that, when normal M20 Grade 8.8 bolts are used, all fin plate connections fail by bolt shear fracture. Reduction of connections' resistance relative to their resistance at ambient temperature follows that of the bolts. Using stronger Grade 10.9 bolts or M24 bolts failed the connections by block shear of the beam web at ambient temperature. At elevated temperatures, the failure was still controlled by bolt shear, but the maximum resistance was significantly enhanced.

The EC3: Part 1.8 and the UK Green Book imply different failure modes from those observed, but they give very conservative estimations of the minimum individual bolt resistances. In general, bolt shear fracture tends to govern the failure of fin plate connections at elevated temperatures. In consequence, specifying fin plate connections seems inadvisable where large connection rotations are anticipated.

Acknowledgement

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