DUCTILITY OF SIMPLE STEEL CONNECTIONS IN FIRE

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Abstract*. This paper summarises the findings of a recently-completed large series of experiments to investigate the resistance of simple connections to combinations of shear, bending and tension forces at temperatures up to 650°C. A total of 64 isothermal tests was conducted on double-angle web cleat, fin plate, partial depth endplate and flush endplate connections to study their ability to resist large tensile forces whilst undergoing large rotations. The tests highlight the importance of considering the ductility of connections as well as their strength, and also reveal that their mode of failure may change with increasing temperature. High-temperature performance of bolts and welds is shown to be of particular importance.*

1 INTRODUCTION

In braced steel frames, the beam to column connections may be designed to function as nominal pins under gravity loads. However, if subjected to fire the connections may be required to resist not only vertical shear forces, but also significant axial loads, and to undergo large rotations. These conditions demand that connections possess good ductility. In the past, experimental investigations of the performance of simple connections in fire have been concerned with moment-rotation characteristics, and have largely neglected to consider the effect of co-existing axial forces. This paper summarises the findings of a recently-completed large series of experiments to investigate the resistance of simple connections to combinations of shear, bending and tension forces at temperatures up to 650°C. A total of 64 isothermal tests was conducted on double-angle web cleat, fin plate, partial depth endplate and flush endplate connections to study their ability to resist large tensile forces whilst undergoing large rotations.

2 TEST ARRANGEMENT

2.1 Furnace and test set up

Experiments were conducted in a $1m^3$ electric furnace illustrated in Figure 1. Each specimen consisted of a stub beam bolted to a short length of column. The column was suspended from a ring frame via a threaded rod passing through the top of the furnace and secured to an inclined support projecting into the furnace. The stub beam was supported by its connection to the column and was loaded at its remote end by a pin-connected bar passing into the furnace. A triangulated system of pin-connected bars was devised to allow the vertical displacement of a screw jack to apply an inclined tying force to the beam [1]. By adjusting the lengths of the bars, the applied force could be applied at different angles relative to the beam axis (nominally 35, 45 or 55°) resulting in changes in the shear:tensile force ratio. Tests were conducted isothermally at 20, 450, 550 and 650°C.

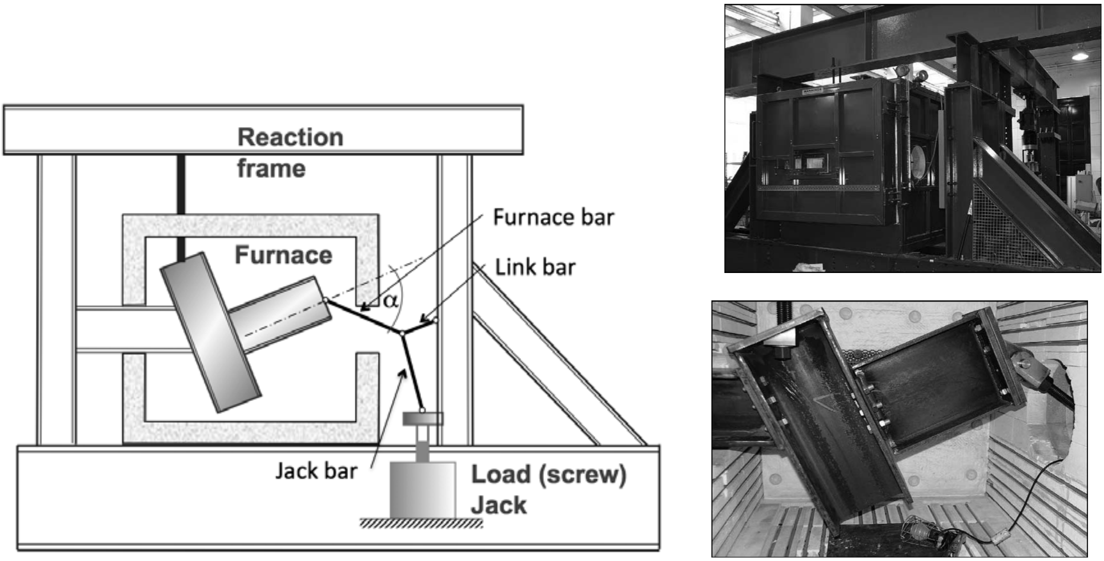


Figure 1: Sketch of test arrangement, photograph of furnace and a typical specimen.

2.2 Specimens and material properties

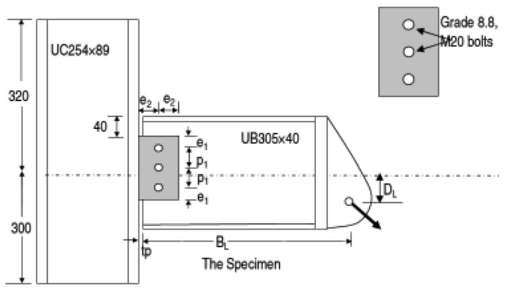
All test specimens used S355 254UC89 columns and S275 306x165UB40 beams. Connections were generally in accordance with UK design recommendations and used S275 fittings (plates and angles) with M20 8.8 fully threaded bolts. The test programme examined “simple connections” (i.e. connections designed to take vertical shear loads not moments) and included fin plates, web angles, partial depth endplates and flush endplates.

2.3 Instrumentation

The applied force transmitted through the furnace bar was calculated by resolving the system of forces measured in the strain gauged link and jack bar; the angles between all the bars were recorded using digital images. Deformation of the specimen within the heated furnace was recorded with digital images taken through a 100x200mm observation hole in the front. Targets placed on the column and beams enabled rotations and displacements to be calculated [2].

3 FIN PLATES

Fourteen fin plate connections were tested as shown in figure 2 and summarised in table 1. α refers to the angle between the axis of the stub beam and the furnace bar (see figure 1), with the subscripts *n*, *i*. and *e* denoting the *nominal* angle, the actual *initial* set-up angle and the final angle at the *end* of the test, which altered due to the deformation of the specimen. The specimen geometry denotes the number of bolt rows, bolt-grade and bolt diameter. In all cases the fin plate was 200mm deep, 100mm wide and 8mm thick in S275 steel with bolts at 60mm vertical pitch. It is clear from table 1 that the failure of fin plate connections is generally governed by the behaviour of the bolts in shear (see figure 3). At elevated temperatures the reduction in overall resistance is closely related to the degradation of bolt strength, which due to the heat treatment undergone in the manufacturing process of bolts, is proportionally greater than that of the steel fin plate. Substituting higher grade bolts (10.9) moved the failure into the beam web which failed in block shear, as shown in figure 3, but only at ambient temperature – at higher temperatures bolt failure remained critical.



Bolt diameter d	20
Bolt hole diameter d_0	22
End distance e_1	40
Edge distance e_2	50
Bolt spacing p_1	60
Fin plate thickness t_p	10
Lever arm B_L	490
Vertical off set D_L	51.7

Figure 2: Fin plate test details.

Table 1: Fin plate test results.

Test	Specimen geometry	°C	α_n	α_i	α_e	Force (kN)	Rotation (Degree)	Failure mode
1	3-8.8-20	20	55	53.9	32.4	145.95	8.107	Bolt shear
2	3-8.8-20	450	55	51.5	41.4	70.5	6.093	Bolt shear
3	3-8.8-20	550	55	53.4	42.7	34.8	6.558	Bolt shear
4	3-8.8-20	650	55	53.1	44.0	18.0	6.255	Bolt shear
5	3-8.8-20	20	35	33.8	34.1	185.1	7.805	Bolt shear
6	3-8.8-20	450	35	39.0	33.5	84.5	6.237	Bolt shear
7	3-8.8-20	550	35	40.9	31.5	37.5	7.121	Bolt shear
8	3-8.8-20	650	35	40.5	30.6	19.3	7.367	Bolt shear
9	3*2-8.8-20	550	35	41.6	32.2	81.1	6.853	Bolt shear
10	3*2-8.8-20	550	55	56.0	46.6	67.0	4.782	Bolt shear
11	3-10.9-20	20	35	36.5	29.8	213.0	10.62	Beam web
12	3-10.9-20	550	35	40.9	23.9	56.8	11.50	Bolt shear
13	3-8.8-24	20	35	37.4	29.7	203.1	8.339	Beam web
14	3-8.8-24	550	35	42.1	29.1	74.0	7.855	Bolt shear
14	3-8.8-24	550	35	42.1	29.1	74.0	7.855	Bolt shear

Figure 3: Deformation of beam web and bolt shear (test 12); beam web block shear (test 11)

As part of the investigation a component based model of the fin plate connection was developed [1]. The behaviour of each bolt row was simulated by three springs arranged in series representing the bearing in the fin plate, bearing on the beam web and shear of the bolt. A parallel spring was used to represent friction at the fin plate/beam web interface. Each spring was represented by a non-liner force-displacement curve including modifications to allow for changes in temperature. Figure 4 shows a comparison of the force-rotation behaviour for connections loaded at 55° at ambient and high temperature (650°C) to illustrate how well the model works. In the simple model, bolts in shear may be assumed to have infinite ductility at their maximum capacity (thus allowing all bolts to reach their peak) or be capable of retaining their peak shear resistance only up to a displacement of ½ the bolt diameter. The circle on the figures denotes the maximum load if the latter assumption is made.

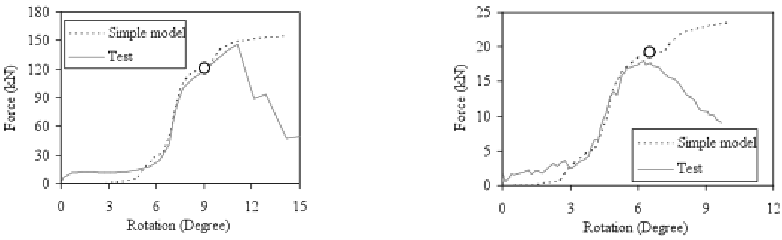
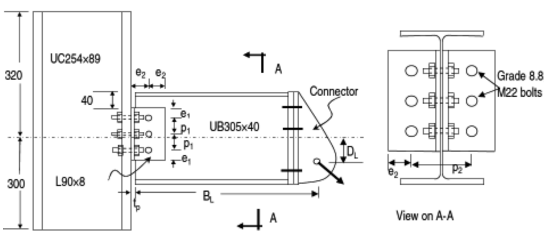


Figure 4: Comparison of simple component model with test results (tests 1 and 4).

4 WEB ANGLES

Web angles (or cleats) were once very popular in the UK but have tended to be less widely used recently as steelwork contractors prefer to fabricate endplate type connections. However, these connections were included the experimental study as they are potentially very ductile. Figure 5 shows details of the test arrangement and table 2 presents the key test results.



Bolt diameter d	20
Bolt hole diameter d_0	22
End distance e_1	60
Edge distance e_2	50
Bolt spacing p_1	60
Bolt spacing p_2	103
Lever arm B_L	490
Vertical off set D_L	51.7

Figure 5: Web angle test details.

The failure mode changed with increasing temperature but not with change of loading angle. For example, at room temperature all tests failed by the column bolt heads first punching through the angle leg and then a block shear fracture of the cleat (see left hand image in figure 6). At 450 and 550°C, the web angles fractured at the heel; at 650°C, the failure mode was double shear of the beam web bolts (right hand images in figure 6). Web angles showed good rotational capacity and developed useful amounts of deformation under tensile forces [3].

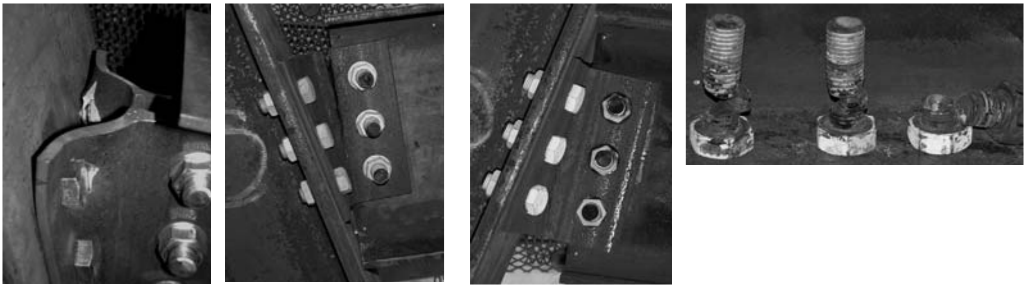


Figure 6: typical failure modes at 20°C, 450-500°C, and 650°C

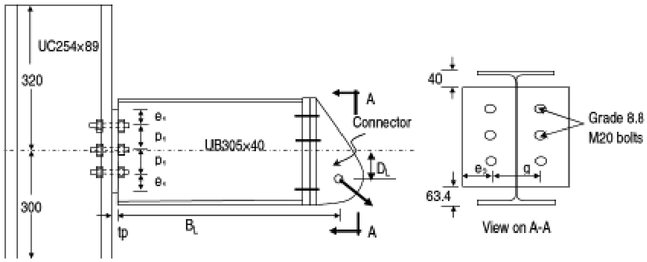
Table 2: Web angle test results.

Test	Specimen geometry	°C	α_n	α_i	α_e	Force (kN)	Rotation (Degree)	Failure mode
1	3-8.8-20	20	55	55.0	34.4	186.34	16.57	Punching shear
2	3-8.8-20	450	55	55.8	43.5	93.74	9.39	Angle fracture
3	3-8.8-20	550	55	56.0	42.2	52.91	10.52	Angle fracture
4	3-8.8-20	650	55	56.5	34.4	25.70	14.15	Bolt shear
5	3-8.8-20	20	45	45.7	32.0	212.54	17.12	Punching shear
6	3-8.8-20	450	45	46.7	37.3	99.42	10.29	Angle fracture
7	3-8.8-20	550	45	47.0	36.8	56.35	11.53	Angle fracture
8	3-8.8-20	650	45	48.1	34.5	28.18	15.94	Bolt shear
9	3-8.8-20	20	35	37.4	21.2	243.17	16.71	Punching shear
10	3-8.8-20	450	35	41.1	29.1	112.85	10.75	Angle fracture
11	3-8.8-20	550	35	41.4	26.6	61.21	12.56	Angle fracture
12	3-8.8-20	650	35	40.9	21.6	31.57	14.86	Bolt shear
13	3*2-8.8-20	550	35	40.2	27.2	85.01	10.95	Angle fracture
14	3*2-8.8-20	550	55	55.7	41.0	66.78	9.19	Angle fracture
14	3-8.8-24	550	35	42.1	29.1	74.0	7.855	Bolt shear

A component model was developed for the web angle connection, the derivation of which is fully described in [4]. The four components modeled by non-linear springs are: column bolts in tension, web angles in bending, beam web bolts in shear and the beam web in bearing. Further work (not reported here) has been conducted to examine the performance of web angles using stainless steel either for the bolts or for the angles. These have shown improvements in high temperature performance.

5 PARTIAL DEPTH ENDPLATES

Partial depth endplates are widely used in the UK. They are simple to fabricate, have good rotational ductility and give good resistance to vertical shear. Their performance in resisting tying forces is quite limited and where resistance to significant tension forces is required designers may opt for a flush endplate instead. A series of 12 tests was conducted to investigate this connection's performance under shear and tension at elevated temperatures – details are shown in figure 7 and table 3 below. All tests failed in a similar manner i.e. fracture of the endplates close to the toe of the weld, as shown for example in the multiple images of test 4 shown in figure 8.



Bolt diameter d	20
Bolt hole diameter d_0	22
End distance e_1	40
Edge distance e_2	30
Bolt spacing p_1	60
End plate thickness t_p	10
Gauge distance g	90
Lever arm B_L	490
Vertical off set D_L	51.7

Figure 7: Partial depth endplate test details.

Table 3: Partial depth endplate test results.

Test	t_p (mm)	Bolt rows	°C	α_n	α_i	α_e	Force (kN)	Rotation (Degree)	Failure mode
1	10	3	20	35	35	43.5	192.0	8.6	Plate fracture
2	10	3	450	35	35	38.7	90.4	4.2	Plate fracture
3	10	3	550	35	35	40.2	68.5	3.9	Plate fracture
4	10	3	650	35	35	39.6	32.6	3.6	Plate fracture
5	10	3	20	45	45	51.1	150.0	8.8	Plate fracture
6	10	3	450	45	45	48.6	64.5	3.5	Plate fracture
7	10	3	550	45	45	47.9	44.1	3.2	Plate fracture
8	10	3	650	45	45	48.6	28.5	3.8	Plate fracture
9	10	3	20	55	55	56.1	179.3	11.2	Plate fracture
10	10	3	450	55	55	55.9	55.6	4.8	Plate fracture
11	10	3	550	55	55	55.1	36.3	3.9	Plate fracture
12	10	3	650	55	55	55.7	22.1	4.5	Plate fracture

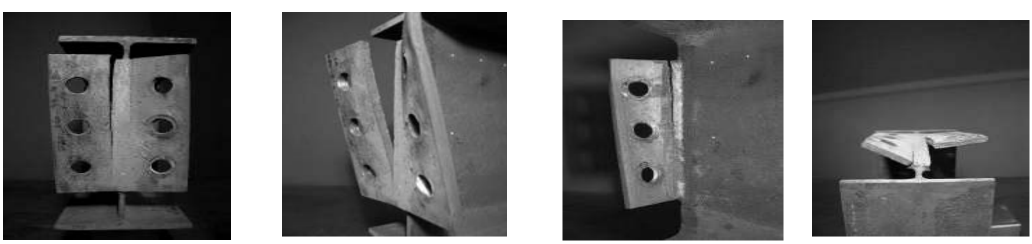
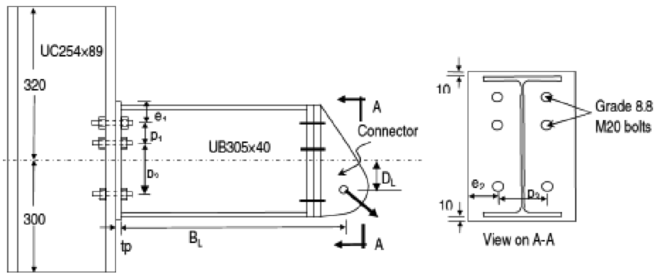


Figure 8: Partial depth endplate (test 4) failure mode.

For this connection type, modeling the behaviour, either by FEA or through a component model, required understanding the failure of the plate. Hu *et al.* [5] reported the development of an ABAQUS model incorporating cohesive elements to represent the behaviour of the plate in the HAZ adjacent to the toe of the weld connecting the endplate to the beam web. The cohesive elements were used to model a fracture as a separation across a surface using a traction-separation constitutive law. Thus tearing through the plate could be modelled. Later, Hu used the test data and FE model to construct a simpler component model building on earlier work by Spyrou [6] and Sarraj [7] but incorporating a new component to simulate the weld behaviour as reported in [8].

6 FLUSH ENDPLATES

Flush endplates, as shown in figure 9, are often used as ‘simple’ connections i.e. designed for vertical shear loads, even though they are capable of taking reasonable amounts of moment and are more accurately categorized as partial-strength semi-rigid connections [9]. Flush endplates are popular because they are easy to fabricate, provide some rotational stiffness, which is useful for temporary stability during the erection phase, and they can resist significant tying forces.



Bolt diameter d	20
Bolt hole diameter d_0	22
End distance e_1	40
Edge distance e_2	55
Bolt spacing p_1	70
Bolt spacing p_2	133.4
Bolt spacing p_3	90
End plate thickness t_p	10
Lever arm B_L	490
Vertical off set D_L	51.7

Figure 9: Flush endplate test details.

The test programme outlined in table 4 shows that the connection is relatively stiff compared with the other connection types investigated. The mode of failure is dependent on the test temperature – at room temperature the critical failure mode was plate fracture close to the weld but under fire conditions the bolts became critical. The ductility of the bolts at high temperature contributes to the rotational capacity of the joint. The use of a thicker endplate increases peak resistance but at the expense of ductility [10,11].

Table 4: Flush endplate test results.

Test	t_p (mm)	Bolt rows	°C	α_n	α_i	α_e	Force (kN)	Rotation (Degree)	Failure mode
1	10	3	20	35			DNF		
2	10	3	450	35	40.5	35.1	180.9	6.1	Plate fracture
3	10	3	550	35	41.2	30.7	105.9	2.2	Bolt fracture
4	10	3	650	35	43.3	32.5	43.6	3.4	Bolt fracture
5	10	3	20	45			DNF		
6	10	3	450	45	46.3	43.7	164.6	6.4	Plate fracture
7	10	3	550	45	44.7	43.1	81.8	1.4	Bolt fracture
8	10	3	650	45	46.6	38.9	36.0	4.9	Bolt fracture
9	10	3	20	55	54.7	43.8	259.0	4.4	Plate fracture
10	10	3	450	55	55.7	46.7	182.5	5.6	Plate fracture
11	10	3	550	55	55.4	47.2	87.8	2.2	Bolt fracture
12	10	3	650	55	55.4	48.1	39.2	3.1	Bolt fracture
13	8	3	20	35	37.8	27.2	258.1	5.3	Plate fracture
14	8	3	550	35	41.8	33.3	101.0	7.5	Bolt fracture
15	15	3	550	35	42.2	37.0	124.6	2.5	Bolt fracture
16	10	2	20	35	36.5	32.8	303.0	6.6	Bolt fracture
17	10	2	550	35	41.7	39.5	79.9	2.6	Bolt fracture

7 DISCUSSION

In the event of a fire in a steel framed structure, thermal expansion of the steel, reduction of material strength and stiffness, and the subsequent regaining of strength and stiffness as the distorted frame cools and contracts, places great demands on the ductility of the beam to column connections. In braced frames, it is desirable to minimise fabrication costs by using relatively simple connections designed to resist vertical shear. However, in fire these simple connections are subjected to significant axial force and enforced rotation. Their ability to maintain integrity of the structure is clearly very important but at the present time it is not possible to guarantee satisfactory performance due to the complexity of the actions (both structural and thermal) during a fire event and the inability of present structural analysis tools to properly account for connection behaviour under extreme conditions. The test programme reported here has provided an insight into the likely response of simple connections to combined shear and axial forces at elevated temperatures. Furthermore, the development of component-based models for each connection type offers the opportunity to incorporate simple yet realistic connection behaviour into structural fire analysis programs. These in turn will permit designers to devise the best connection arrangements for use in structural fire engineering of steel frames.

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